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Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
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The Standardization of Public Buildings.

The enormous expenditure by the Government for public buildings and their sites—$163,085,431 in the last twelve years—emphasizes the need of high efficiency in this class of construction. The necessity of more systematic work in connection with the design and construction of government buildings has long been felt, and it is gratifying to note that the committee which was appointed by Congress to report on the subject of national building construction has recommended the creation of a Federal Bureau of Public Buildings. This committee consisted of Secretary McAdoo, Attorney General McReynolds, Postmaster General Burleson, Senators Swanson and Sutherland, Representatives Clark and Austin, and Sherman Allen, former assistant secretary of the treasury, and the weight of their recommendations probably will result in the creation of such a bureau.

The great saving which should result from the creation of an efficient bureau, in charge of men who possess not only a knowledge of aesthetic building design but also thoroughly understand modern construction methods, materials and equipment, is evident to all. On account of the similarity of the government work in various cities it should be possible to adopt standards of construction which would result in a large ultimate saving. In addition to this it will be possible to create designs for cities of various sizes which will do much to overcome the present inequality of expenditures due to the efforts of those directly interested in buildings in certain localities. Designers of public buildings may rightly be expected to point the way in aesthetic building design, yet it is essential that such buildings be planned for the efficient conduct of federal business and the certain protection of government records. Not only must the buildings be of fireproof construction, but the spread of a fire originating within a building must be rendered impossible. The creation of this bureau will do much toward guaranteeing better public buildings by fixing directly the responsibility for such construction. Moreover, better public buildings will act as an incentive to designers of commercial buildings, the educational value of such a bureau being one of its greatest assets.

The New York Sewerage Report.

In the sewerage section of this issue we publish an article outlining the chief features of the main drainage and sewage disposal works proposed for New York City by the Metropolitan Sewerage Commission. The article is based upon the final report of the Commission. The importance of this report warrants a brief review of it in these columns.

The report contains about 750 pages and is the third bound volume issued by the Commission to describe its investigations, to record its opinions and to state its recommendations. There have also been issued, at various times, seventeen preliminary reports. The final report summarizes the work done from 1906 to 1914, describes the plans for main drainage and sewage disposal works recommended for the protection of the harbor water, contains the reports of experts consulted by the Commission, and contains data relating to the protection of the harbor. The reports by experts include five critical reports and four reports upon special topics, with digests of these reports and the inclusion of explanatory matter by the Commission. Analytical results not previously published, relative to the effect of sewage on the harbor water, are included. There is also a discussion of the present status of the various methods of sewage disposal and numerous practical examples are given, by illustrations and descriptive matter, of the main drainage and sewage disposal works of various large European and American cities. A chapter of special interest relates to the utilization of sewage with particular reference to the possibility of deriving a financial return from the sewage of the city. This chapter contains a very thorough study of the composition of sewage, the value of sewage as a fertilizer, the utilization of sewage in agriculture, and of sludge and its disposal.

The report will prove interesting reading to all sewerage engineers. It is notable for the plan of presentation consistently followed whereby the Commissioners state the reasoning which caused them to advocate certain plans in preference to the various alternatives considered. The advantages and disadvantages of the alternatives are also given in a consistent degree of detail. This feature of the report makes it of high value to the sewerage engineer. The report is conservative in its recommendation of expenditures, of treatment processes and of the degree of purification to be sought.

The outlet island project is given special attention in our article above mentioned. This feature will appeal to the general interest of all our readers for it possesses, at first sight, many of the spectacular elements characteristic of the feat of engineering skill equally attractive to the engineer and the layman. On closer examination, however, one is impressed by the fact that this project possesses no untried or unprecedented feature and really is conservative engineering. Aside from the island project all of the measures proposed are commonplace and, while extensive, are exceedingly simple in principle.

Concerning Experience.

Speaking from the standpoint of the individual there are two kinds of experience, one's own and that of others. Each type, when properly interpreted, classified and recorded, adds to the knowledge of the individual. Having observed a tendency on the part of many engineers to ignore, overlook or to make slight use of the second type named, we desire to suggest its extreme value and to urge its fullest utilization.

The young engineer, impressed by the value of experience, ordinarily will make sacrifices to attain it. Frequently after years of work, he finds it hard to market to advantage the experience he has gained. He is perplexed and begins to doubt its value. It is interesting to examine the cause of his predicament. Very commonly, in such cases, the individual is the victim of a confusion of ideas; he has mistaken service for experience. He has performed the same kind of work from day to day, learning very little new once he had his job well in hand. It should be borne in mind that length of service does not proportionately add to one's fund of experience unless he seeks it constantly.

In the majority of minor engineering positions the duties are largely routine and are soon learned. If he would advance the worker must then turn a part of his attention to the work of others and learn all possible concerning it. He must observe and he must weigh and record what he observes. This is learning from the experience of others. In addition to direct observation much of this type of experience may be gained by the engineer who has self-interest enough to read technical literature, particularly current literature.

These thoughts are suggested at this time by our knowledge of a recent decision by the head of the civil service bureau of one of the largest cities in this country. Engineer candidates for promotion will be tested to determine what they know of recent developments in their field. In other words, they will be examined as to their willingness and ability to learn from the experience of others through reading.

The Increasing Value of Annual Water Reports.

From the viewpoint of those who receive and examine the annual reports of the water departments of our larger and secondary cities the value of such reports is increasing. We have several times urged the advisability of including in such reports specific information of general interest and it is in this respect that their value may fairly be said to be increasing. While recording our conviction on this point we desire once more to point out the wisdom of including in annual reports information of direct value and interest to water works engineers and superintendents generally.

Every department of sufficient size and importance to warrant the publication of an annual report must surely, during the course of a year, learn something which should be made a matter of record available to those engaged in the business in other localities. From the standpoint of local requirements it is doubtless necessary and, in fact, is frequently required by ordinance, that full statements of the departmental finances be published in the report. Similarly, where record keeping systems are incomplete or faulty, it is doubtless advisable to incorporate many pages giving the location of valves and hydrants, stating the length and location of extensions to the pipe lines in the distribution system, and giving somewhat similar data relative to meters and service connections. The compiler of the report should bear in mind, however, that such matter is almost entirely, if not quite, valueless to those on the department's exchange list. Unless something more than information of this character is included in reports their exchange is an unwarranted expense.

We do not wish to discourage the inclusion of whatsoever matter local customs, rules or laws may require, but we desire to reiterate our previous statements that specific information of general interest must be included to make such reports of any value to those in other localities. If only a half dozen pages are utilized for reporting unit costs, construction methods, ingenious methods of making repairs, improved methods of keeping records, or the results of special studies made during the year, the report assumes value at once as an exchange.

It is probably unnecessary to do more than state that each department owes a debt to the water works fraternity which can best be paid by reporting its own progress in methods or the results of its researches. The annual report is a proper vehicle for the dissemination of such information.

To return to local requirements, we take it that all progressive and competent water department officials desire the interest of the water takers in the department's activities. We believe that these very points of specific interest to which we have made reference will also prove of vastly more interest to local people than many pages of tabulated statistics. The subject is a very important one, and is worthy of the attention of all departments issuing annual reports.

BRIDGES

Design and Construction of the St. Lawrence River Bridge near Montreal, Canada, With Particular Reference to the Erection of the 408-Ft. Spans.

The St. Lawrence River bridge of the Canadian Pacific Ry., which crosses the St. Lawrence River about seven miles from Montreal, near the village of Caughnawaga, is a double-track structure consisting of two single-track structures on common piers. The methods used in erecting this bridge are of special interest, due to the fact that the current was swift and it was necessary to maintain traffic. As the old single-track bridge was an exceptional structure some data will also be given relative to it. This article is based on a paper by P. B. Motley, in Proceedings, Canadian Society of Civil Engineers.

THE OLD BRIDGE.

The Caughnawaga crossing was adopted for the site of the old bridge after a thorough investigation of several possible lines, as it gave the most economical location and best suited the requirements of navigation. The bridge was approximately ⅝ mile long, and the navigation requirements were taken care of by using two through spans of 408 ft. each over the deepest portion of the river, a clearance of 60 ft. above high water being maintained. The span lengths for the remainder of the bridge were generally 270 ft. and 240 ft., these lengths being dictated by the judgment of the engineers with respect to ice flow in the river.

The substructure of the bridge was begun in the spring of 1885, and the erection of the steelwork was carried on during the winter of 1886-1887. The superstructure was designed by the late C. Shaler Smith and was fabricated and erected by the Dominion Bridge Co. It was especially designed with a view to quick and simple erection, as it was impossible to place falsework in the deeper portions of the river where the 408-ft. spans were located. To meet the conditions the designer decided upon a peculiar type of bridge. The structure consisted of four spans (two deck and two through), continuous over five supports, which enabled the steelwork on the side or flanking spans to be erected first on falsework and the main channel spans to be erected by the cantilever method, the flanking spans being used as anchors. Some of the steelwork was also cantilevered in both directions from the pier located in the center of the channel. When these several spans were connected they formed a continuous span over five supports, fixed at the center pier and extending in both directions therefrom. The channel spans were made of through design to allow steamers to pass, and Mr. Smith solved the problem of combining the deck and through spans by a very beautiful and interesting design, in which the ends of both the upper and lower chords of the 408-ft. through spans were curved.

This procedure is open to criticism from a mathematical point of view, but, considering that the pier supports were founded on rock and in addition had adjusting screws so that the ideal conditions upon which the calculations had been made could be at all times maintained (if necessary), the design was considered justifiable. The engine loads for which the old structure was designed were equal approximately to Cooper's E-35 loading, followed by a train load of 2,500 lbs. per lineal foot. The material in the structure was steel, except the stringers, counters and wind bracing which were iron. The design lent itself admirably to rapid erection, which was borne out by the fact that the steel took only twelve months to erect.

THE NEW BRIDGE—GENERAL REQUIREMENTS.

By 1910 the requirements of traffic had necessitated the use of much heavier engines than were considered in the original design, and the increasing volume of traffic made it advisable to double track the line from Montreal eastwards. Bids were accordingly called for on designs prepared by the railway company's engineers, and a contract was subsequently entered into with the Dominion Bridge Co. for the removal of the old spans and the erection of the new ones. A contract was also made with The Foundation Co. for the extension of the substructure to accommodate the extra steelwork.

In the old structure there was no traffic to be taken care of during construction, but in the new one it was not allowable to interfere with the regularity of passing trains. This considerably complicated the problem, and it was decided that the only possible way of meeting all the requirements was to build two independent single-track bridges, and to remove the old bridge in sections, transferring traffic from side to side, as will be described later.

SUBSTRUCTURE OF NEW BRIDGE.

From observations made during the life of the old bridge it was noted that the ice of Lake St. Louis generally grounded on the Lachine side of the river in shallow water and after breaking up floated under the Lachine end of the bridge in small pieces in a manner which did not seem to justify the existence of four 240-ft. spans between piers Nos. 3 and 7 (see Fig. 1). It was therefore decided to bisect these spans by building new intermediate piers (Nos. 3A, 4A, 5A and 6A), and to use eight 120-ft. spans instead of four 240-ft. ones. This resulted in considerable economy in cost. Between piers Nos. 7 and 11, it was not considered advisable to make a change. With these exceptions, the structure was renewed in span lengths similar to those which originally existed; but it was decided to us simple spans of ordinary deck and through types instead of the continuous spans between piers Nos. 11 and 15. The new second track was placed on the downstream side of the existing bridge. The added masonry was bonded into the old above the water line, while below the water line open caissons were sunk to the same hard bottom to which the original masonry was carried, except in the case of piers Nos. 9 to 13, where pneumatic caissons were found necessary. Caisson No. 13 was square ended where it butted against the old masonry, and was pointed at the downstream end. It was carried about 7 ft. lower than the bottom of the old pier, because it was found that the shale immediately under the pier was of such character as to make it advisable to go deeper in order to avoid any possibility of settlement, which would result in serious cracks in the bond above the water line. The work at piers Nos. 8 to 14 was carried on in still water, which was obtained by the use of wing dams composed of rock-filled cribs sunk a short distance upstream at an angle of about 45°. Pneumatic caissons were used for constructing pier No. 13. Masonry work was started in June, 1910, and was finished in November, 1911, except the upstream end of pier No. 13, which was left until the removal of the old steelwork, after which it was built up by the railway company's forces.

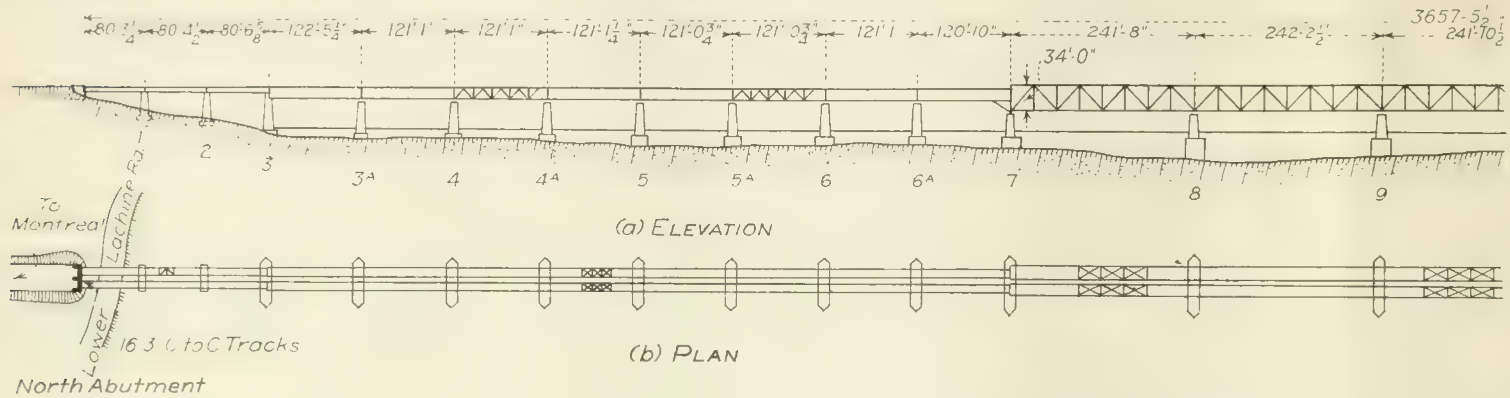


Fig. 1. Elevation and Plan of St. Lawrence River Bridge Near Montreal, Can.,

SUPERSTRUCTURE OF NEW BRIDGE.

The 80-ft. deck plate girders at the Lachine end of the bridge (see Fig. 1) are the Canadian Pacific Ry. standard design, and are single-track spans placed alongside of each other. The 120-ft. spans are Warren deck truss spans with riveted connections, their ties resting upon the top chords. These spans are also simple single-track spans laid abreast of each other. The 240-ft. and the 270-ft. spans are Warren trusses with riveted connections, and have the usual floor system of stringers and floorbeams (two stringers per track) riveted against the vertical posts directly under the top chord. The upper laterals are also riveted directly below the top chord, and they are connected to the top flanges of the stringers, where they intersect the same.

The general dimensions and the type of truss used for the 408-ft. spans are shown in Fig. 1. These spans are of the sub-paneled Pratt truss type, and the top chords are curved to an approximate parabola. In the web members, solid web plates have been largely used instead of lattice bars. The vertical posts are all of I-sections, composed in most cases of bulb angles and web plates, and where necessary there are stiffeners on the webs, especially on the longer vertical posts. The top and bottom chords are very stiff sections, partly to allow them to be cantilevered out during erection. The 408-ft. spans were also calculated for the stresses caused by the special method of cantilevering and launching, which will be described later. The portal and other subsidiary bracing is generally of a stiff design, consistent with the main trusses to which it is attached. The 270-ft. spans were also calculated for the concentrated weight of one end of the 408-ft. spans, which was to be carried upon them during the process of launching.

The alignment on the bridge is ruled by the overall width of the 408-ft. spans. At pier No. 11 and at the south abutment the two single-track bridges are 27 ft. 0 in., center to center, and from pier No. 11 to the north abutment the spans come closer together until they are 16 ft. 4 ins. at the north abutment. This slight "kink" in the alignment is quite immaterial from an operating point of view, and permitted a great saving in masonry from pier No. 11 northward.

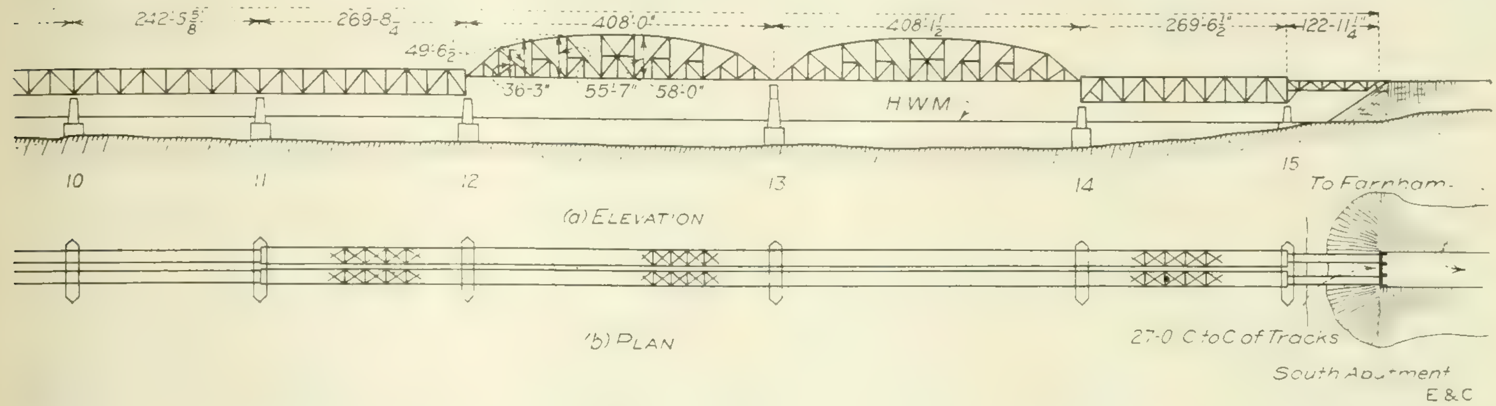
The following table gives the actual weights

south abutment were then erected. After these were finished the 408-ft. downstream spans were erected on top of the 270-ft. spans.

The method used in connection with the erection of the 408-ft. spans constitute one of the most interesting features of the work. The scheme was to launch the span endwise with its rear end supported upon an ingenious truck, or buggy, while the forward end was supported on a large scow of special design. In order to avoid overstressing the adjacent 270-ft. span by the concentrated loads of the sliding gear, an ingenious framed structure was devised by which it was possible so to distribute the end reaction over the floor system of the carrying span that no part would be overstressed. A diagram of this truck or buggy is shown in Fig. 2, from which it will be noted that the efficiency of the construction is due to the fact that there is no vertical

The 270-ft. spans were erected by means of these same 120-ft. temporary trusses supported on a temporary tower about 30 ft. wide. The temporary towers used for erecting the 240-ft. and the 270-ft. spans constituted the only falsework permitted. The railway company further specified that no traffic was to be borne by these towers.

The four 408-ft. spans are alike, but two methods differing somewhat in detail were used in launching them. The two downstream spans were launched on the same set of carrying scows, but with a pilot scow upstream to take up the slack in the cables and to control the movement of the spans during launching. The experience gained in launching the two downstreams 408-ft. spans led to the abandonment of the pilot scow for the placing of the upstream ones. Under this arrangement the new spans were allowed, while traveling, to



Showing Type of Construction and General Dimensions.

of the various spans and the total weight of the superstructure:

Double-track spans	Wt., lbs.
3 80-ft. deck plate girder spans	613,672
8 120-ft. deck truss spans	3,914,781
4 240-ft. deck truss spans	7,710,077
2 270-ft. deck truss spans	5,414,681
2 408-ft. through truss spans	10,291,135
2 122-ft. deck truss spans	518,585
Total weight of superstructure	28,462,931

ERECTION FEATURES.

One of the most important problems in the work was the maintenance of traffic during the erection of the steelwork. There were on an average ten trains between the hours of 8 a. m. and noon, and sometimes an average of eight trains in the afternoon during the usual working hours. In order to carry out the work without interfering with traffic it was decided to erect first all of the spans on the downstream side from the north abutment to pier No. 7 (see Fig. 1). After this was done the two new 120-ft. spans between piers Nos. 6 and 7 were moved bodily into the location of the old 240-span, and the latter was moved upstream upon timber towers prepared for it. The new downstream spans between pier Nos. 6 and 7 were then erected. Traffic was then diverted over the four new spans between piers Nos. 6 and 7 by means of a cross-over laid on suitable wooden ties bridging from span to span, all the old spans between pier No. 7 and the north abutment being thus released. These old spans then were taken down and new spans were erected. Next, the new spans on the downstream side between piers Nos. 7 and 11 were erected.

In order to release the other upstream spans it was only necessary to slew over the 240-ft. span between piers Nos. 10 and 11, all new spans between the north abutment and pier No. 11 now being under traffic and all old spans between these points being released. After the new spans on the upstream side between the north abutment and pier No. 10 had been erected it was only necessary to pull into alignment the span between piers Nos. 10 and 11, and thereby put the traffic on all new spans between the north abutment and pier No. 11, the old spans between pier No. 11 and the south abutment being still under traffic. The spans between piers Nos. 10 and 11, Nos. 11 and 12, Nos. 14 and 15, and No. 15 and the

tie inside the triangulation. In place of such a tie there is an exterior strut which, by reason of the proportions of the members, carries a reaction which is equal to that at each of the outer ends of the triangulation. Thus, there is a three-point bearing, with equal reactions. The skidways consisted of eight 100-lb. greased rails (2 sets of 4), on which cast-steel skids or slippers were placed. The scheme was to move the span forward until it came to the last panel of the 270-ft. span, where the front bearing of the three would naturally tend to

rub along a specially prepared vertical skidway bolted to the lower chords of the downstream spans already in place. Details of the actual operation of launching are shown in the Figs. 2 and 3.

The carrying scow was really composed of two independent scows (see Fig. 2), with two 100-ft. deck plate girder spans (four girders) placed on them to equalize the load over the two scows. On these equalizing girder spans was erected a stiff timber tower, upon which the spans themselves were carried.

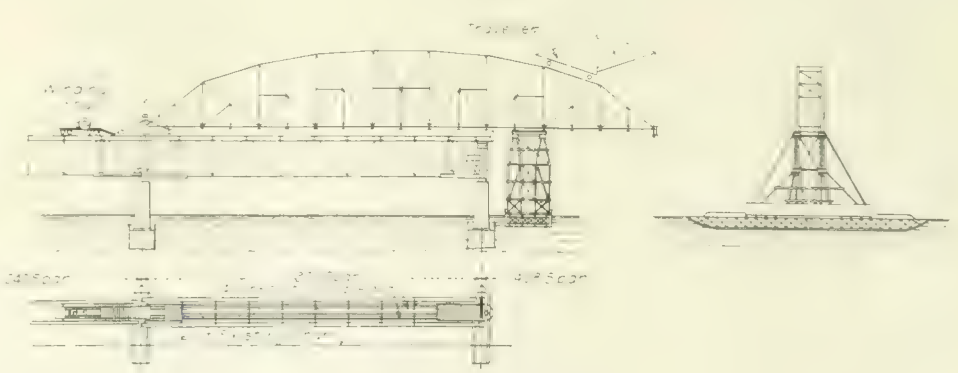


Fig. 2. Diagram Showing Manner of Erecting and Launching 408-ft. Channel Spans of St. Lawrence River Bridge.

pass overboard. In order to satisfy all conditions the two outer bearings were here abandoned, and the span was wedged up on the center bearing only. As the whole reaction was thus concentrated on one point, it was necessary to reinforce the end panel of stringers in the 270-ft. spans. Details of this floating operation will be referred to later.

The 120-ft. spans were erected by means of a temporary 120-ft. deck span, which was swung from the abutments in such a manner that its top chord was practically in line with the top of the abutments.

The 240-ft. spans were erected by means of two 120-ft. temporary trusses, which were supported upon the regular piers and upon a temporary wooden pier placed midway between these piers.

The following weights and reactions were determined before the floating operations were carried out:

Item—	Wt., lbs.
Steel in each 408-ft. span	2,550,000
Floor at 250 lbs. per foot	100,000
Total	2,650,000
Traveler	90,000
Loading truss and sundries	99,000
Scow reaction-span	1,765,000
Traveler cantilever	90,000
Total	1,855,000
Loading truss reaction-span	885,000
Truss and sundries	99,000
Total	984,000
Floor and track on 270-ft. span:	
9 panels at 660 lbs. per foot	165,000
1 panel at 800 lbs. per foot	20,000
Total	185,000

The following computations were made to determine the displacement of the scow:

Weight of span, pounds	1,855,000
Tare of scow, pounds	700,000
Falsework	400,000
Engines, borders, pumps, etc.	30,000
Total weight, pounds	2,985,000

$$\text{Draft of scow} = \frac{2,985,000}{180 \times 37 \times 62.5} = 7.15 \text{ ft., or } 84 \text{ ins.}$$

$$\text{Displacement per inch of immersion} = \frac{2,985,000}{84} = 35,400 \text{ lbs.}$$

Anchors, each composed of concrete blocks securely strung together, weighing approximately 76 tons out of water or 52 under water, were placed about 1,500 ft. upstream from the bridge. These anchors were generally in line with piers Nos. 12 and 13, respectively, but on the land above pier No. 14 a "dead man," consisting of an I-beam embedded in the rock was used.

The current in the river varied between five and eight miles an hour, according to the location where the meter was used.

In estimating the wind pressure against the span (and the required anchorage) the area of two trusses was computed as 13,000 sq. ft. The area resisted by the scow was ⅔ × 13,000 = 8,700 sq. ft. The wind pressure for a velocity of 25 miles per hour = 2.5 × 8,700 = 21,800 lbs.; for a velocity of 35 miles per hour = 5.0 × 8,700 = 43,500 lbs.; and for 50 miles per hour = 10 × 8,700 = 87,000 lbs. The design of all anchorage was based on the scow resistance of 100,000 lbs.

On the main carrying scow a dynamometer was inserted in the reaving of a 14-rope tackle attached to the main anchor line, in order to record the pull on the anchor ropes. The readings taken from this dynamometer fully corroborated the experiments which had previously been made regarding the resistance of floating bodies to be current in the river. They also agreed generally with Froude's formula,

$$R = fs\left(\frac{V}{6.9}\right)^2$$

where

R = resistance in pounds;
s = wetted surface in square feet;
v = current velocity in miles per hour;
f = constant (by experiment = 0.925).

For the erection barges in the Lachine current,

s = 10,800 (for 8-ft. 6-in. draft);
v = 9.5 (7.5 actual);
R = 19,000 lbs.

A dynamometer placed in 1 part of a 14-part tackle showed, during the launching of the span, a maximum reading of 1,200 lbs.; representing a total pull of 16,800, and an average reading of 1,000 lbs., representing a total pull of 14,000 lbs.

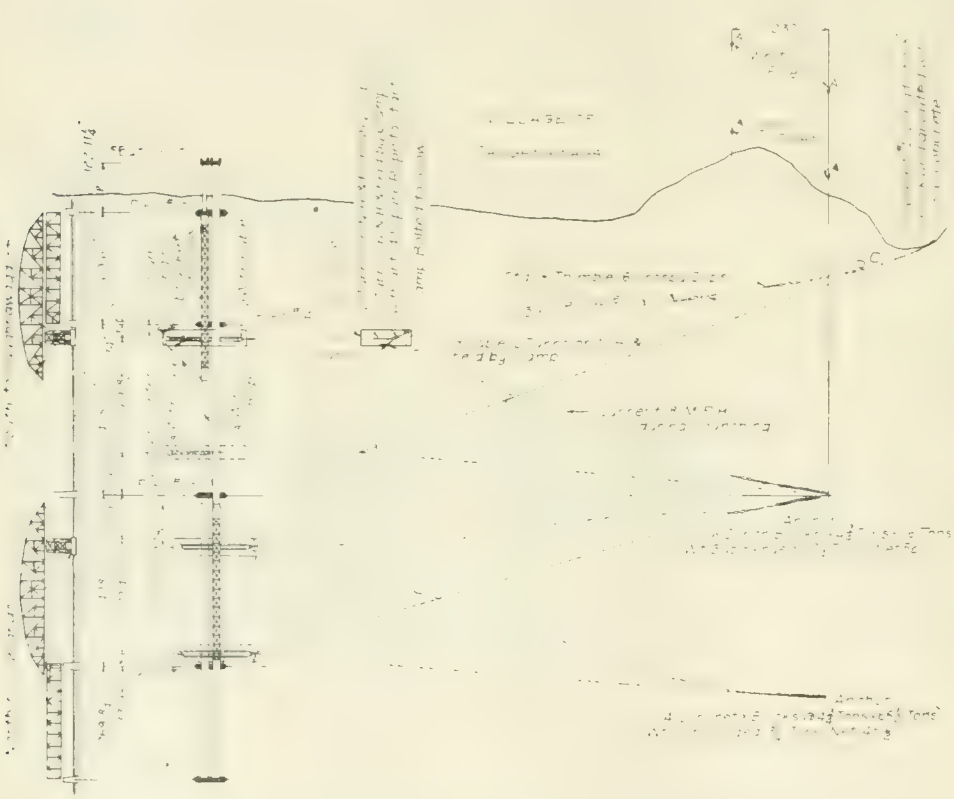


Fig. 3. Diagram Showing Equipment Used and Manner of Anchoring Scows and Launching 408-ft. Spans of St. Lawrence River Bridge.

DETAILS OF LAUNCHING OPERATIONS.

After all the preliminary arrangements had been completed an ordinary Lidgerwood unloader, such as is used on railway work, was located and strutted in a position where a direct pull could be made from the drum of the engine.

Assuming friction at a maximum of 18½ per cent the pull was 180,000 lbs. The unloader capacity with a two-part purchase was 240,000 lbs., and the load on the 1½-in. cable was 90,000 lbs. The compression in the rails, per rail, was 3,600 lbs.; or 3,670 lbs. per square inch; and the pressure on rails per inch (assuming a shoe 6 ft. long) was 570 lbs.

Communication was maintained between the man in charge of the Lidgerwood engine, those in charge on the scows, and the man in full charge of the operations, by means of a system of flag signals. The movement of the span was started by a number of jacks, after which the Lidgerwood engine controlled the whole of the movement; at no time was there any unexpected trouble. While the span moved forward the anchor cables allowed the anchor scow to float across the current with a radial motion. This made it necessary for the cable which connected the anchor scow with the main scows to be constantly shortened in order to maintain the true alignment of the span. As has already been stated, these anchor scows were used only in connection with the floating of the two downstream 408-ft. spans. They were omitted when floating the upstream 408-ft. spans, which were allowed to rub against the neighboring spans already in position. The adjustment of the anchor cables was made as required.

The time occupied in the floating operations is of considerable interest, these data being shown in Table I.

TABLE I.—DATA ON FLOATING OF 408-FT. SPANS.

Span.	Between piers Nos.	Date floated into position.	Elapsed time. Hrs.	Mins.	Net time occupied in moving. Hrs.	Mins.	Total distance floated, ft.
Downstream—408-ft. span	13 and 14	Nov. 4, 1912.	3	0	1	0	275
Downstream 408-ft. span..	12 and 13	Nov. 22, 1912.	2	30	0	28½	275
Upstream—408-ft. span........	12 and 13	Sept. 18, 1913.	1	30	0	22½	275
Upstream 408-ft. span	13 and 14	Oct. 6, 1913.	2	0	0	16	275

TABLE II.—TIME REQUIRED TO ERECT THE STEELWORK.

Date.	Operation.
March, 1911....	Erection started at north end.
May 28, 1911...	New 120-ft. spans moved into upstream alignment, replacing old 240-ft. span between piers Nos. 6 and 7.
July 6, 1911....	Traffic diverted to downstream track between piers Nos. 6 and 7.
Nov. 12, 1911...	New downstream span between piers Nos. 10 and 11 slewed over, thereby releasing old spans between piers Nos. 7 and 11.
Mar. 31, 1912...	Spans between piers Nos. 10 and 11 moved into upstream alignment, allowing erection to proceed on downstream spans between piers Nos. 10 and 11.
June 18, 1912...	Spans between piers Nos. 14 and 15 and between pier 15 and south abutment erected.
July 13, 1912...	Downstream spans between piers Nos. 10 and 11 and between piers Nos. 11 and 12 erected.
Nov. 4, 1912....	Downstream 408-ft. span between piers Nos 13 and 14 floated.
Nov. 22, 1912...	Downstream 408-ft. span between piers Nos. 12 and 13 floated.
April, 1913.....	Started taking down old spans between pier No. 11 and south abutment.
June 10, 1913...	All old steel dismantled.
Sept. 18, 1913...	Upstream 408-ft. span between piers Nos. 12 and 13 floated.
Oct. 6, 1913.....	Upstream 408-ft. span between piers Nos. 13 and 14 floated.
Nov. 4, 1913....	All new steel erected and double track put into service.

The total weight of each 408-ft. span while launching was 1,300 tons.

During each of these operations all of the regular trains of the railway company were allowed to pass on the adjoining span, which necessitated stopping the floating operations because the work of signaling and superintendence was interfered with. The difference between the net time and the gross was occupied in overhauling cables, taking up slack, and in dismantling some of the steelwork connected with the special truck when it reached the last panel of the 270-ft. span. At this point it was necessary to remove certain steelwork which became no longer necessary on account of the load being shifted from a three-point to a one-point bearing. A diagram of this buggy is shown in Fig. 2. The perfection of the control, under which the span was at all times, is exemplified by an incident which occurred during one of the floating operations. The span had reached a point 3 ins. short of its correct location, and after the necessary signaling over the intervening distance of nearly 800 ft. the Lidgerwood engine driver made exactly the 3-in. movement called for. This is remarkable considering the tonnage which was handled. As has already been stated, the 408-ft. spans were skidded upon the deck of the adjacent 270-ft. deck spans, and after each pair of the large spans (on one track) was floated into its correct location they were at an elevation approximately 12 ft higher than their permanent levels. This required that they be jacked down to their correct bridge seat levels, which was done by means of 150-ton jacks and blocking, the end floorbeams having been designed for this purpose, as were also the end cross-beams of the 270-ft. spans. The 240-ft. spans were also provided with special end bracing to enable them to be jacked up on the piers (if necessary) during erection.

SUMMARY.

The time occupied in erecting the steelwork for the various spans is shown in Table II.

The total weight of steelwork in the old single-track bridge was about 4,100 tons; in the new double-track structure it was 14,231 tons.

The total quantity of masonry and concrete in the original piers and abutments was approximately 12,400 cu. yds., while that in the additions to old piers and in new piers was 13,300 cu. yds.

The total length of the bridge and the height above watermarks were not changed.

The number of rivets in the new bridge is approximately 3,500,000.

As far as can be ascertained from the records, about 3,500 cars were used in transporting stone and steel to the structure.

One of the noteworthy performances during erection was the speed with which the five old spans between pier No. 11 and the south abutment were dismantled and the new spans erected. This work was done during the period April 22-Oct. 31, 1913, i. e., 4,100 tons of steel were erected in six months, or 683 tons per month, without interrupting the railway company's traffic.

The cost of the work was slightly under $2,000,000.

PERSONNEL.

In charge for the Foundation Company were John W. Doty, chief engineer, and W. B. Taylor, superintendent. Representing the Dominion Bridge Company were F. P. Shearwood, James Finley and David Bell, in charge of the design, superintendence and erection, respectively, under the direction of G. H. Duggan, chief engineer. C. C. Schneider was connected with the work as consulting engineer. Representing the railway company were J. M. R. Fairbairn, assistant chief engineer, and the writer, who, as engineer of bridges, was responsible for the design and approval of all detail plans.

Fundamental Principles of Aesthetic Bridge Design.

As our bridges are becoming more permanent in character, and as their minor features are becoming standardized, engineers can well afford to give increased attention to the aesthetic design of such structures. The engineer should not consider his problem solved when he has provided a structure which will economically carry the required loads; he should not authorize its construction until he has carefully studied the structure with a view of improving its appearance. In many cases the increased cost of an aesthetic design is relatively very small. The following article, which was abstracted from a paper by Charles Evan Fowler, in Proceedings, Pacific Northwest Society of Engineers, considers some basic principles applicable to the aesthetic design of bridges.

The designing of a bridge is not merely a matter of figuring the stresses, fixing the sizes of members, and making a set of drawings. This may all be completed and a design may result, but the design may be one which is not suitable for the location and is not such as it should be architecturally.

The true design to adopt must come very largely as an inspiration to the designer, who must have the necessary talent or imagination to conceive a bridge that will be the most appropriate and harmonious for a given location, and for the existing or future surroundings.

DISCUSSION OF SOME IMPORTANT BRIDGES.

The Tower Bridge in London, considered by many writers to be a monstrosity, was conceived by the engineer Sir John Wolf-Barry to harmonize with the surroundings, and particularly with the old Tower of London in the immediate vicinity. When considered in this light it is an appropriate and harmonious design; the mediaeval towers are monumental and the steelwork graceful.

The great suspension bridges of New York are of the same class and are mostly in harmony with the towering buildings in the adjacent territory. The details, however, have not always been carried out in the proper spirit, the towers of the Roebling Bridge, never having been completed, and are thus lacking as truly monumental or architectural features. The towers of the nearby Manhattan Bridge are too light to harmonize with those of the Roebling Bridge or with the nearby buildings, although considered alone it is a complete and pleasing design. The Williamsburg Bridge is entirely lacking in architectural features, it is out of harmony with the present or probable future surroundings, and is only notable in design by reason of its magnitude and the graceful sweep of its cables.

The great arch at Hell Gate, over East River in New York, is, with its 1,000-ft. span and carefully designed abutment towers, a truly monumental structure, and the abutment towers are well designed and appropriate, the great arch demanding this mass for backing to satisfy the impression of a great thrust properly cared for or resisted.

The Washington Bridge across the Harlem, one of the great bridges of the world, is wonderful in its architectural detail, but is lacking in the great essentials for a work of architecture. The main structure consists of two spans, instead of three, thus giving a pier in the middle; and the approaches are not balanced, thus causing the structure to lack in symmetry. Some of the other designs for this structure, while more simple in detail, would have made a much better structure and a more pleasing one, as they possessed the fundamental features that are necessary to any real work of architcture—simplicity, symmetry, harmony and proportion.

This is well illustrated by the Eads Bridge, at St. Louis, with its three great arch spans and appropriate approaches. Although the details of the structure are very simple, it is one of the most pleasing and dignified of the world's bridges.

GOVERNING CONSIDERATIONS.

The conception of a design depends upon no rules, but upon the inherent ability of the designer, limited as we shall see hereafter by certain theoretical and practical requirements. There may be conceived a number of designs for a given location of different types, any one of which would be appropriate and beautiful, but the final decision as to which one of these to adopt must be made on the basis of relative cost, the most economical being usually selected. On the other hand the most economical should not always be selected, but the best design architecturally should be adopted, especially where the difference in cost is not very large.

The cost, for example, of a four-span structure for a given crossing might be found to be somewhat less than for a three-span bridge, yet it would be wise from any architectural standpoint to adopt the three-span design. The four-span design would have to be very much cheaper than three spans to call for its adoption. The reason for adopting a three-span design in place of a two-span bridge might, on the other hand, be based entirely on the architectural features, as the cost and risk of the three-span structure might be much the greater.

There are, then, two considerations that govern in making a decision as to what design to adopt for any location—the architectural features and the economy of construction, which latter may well be discussed in full before considering the architecture of bridges, and entirely aside from the aesthetics of the problem.

The loads on the piers for various lengths of spans must first be determined in order that the foundations, abutments, and piers may be designed and the costs determined as factors in the cost of complete structures with different numbers of spans. The load on the pier and the weight of the pier having been determined, the size of the base of the pier may be arrived at by using the formula for the allowable pressure on the foundation bed, as given in the writer's treatise on "Sub-Aqueous Foundations," the character of the foundation bed having been determined previously by careful core borings. Several trials may be necessary before the proper size of a pier is determined, and then careful calculations as to the stability of the pier must be made. Should the size be found deficient, as is often the case with deep trusses, or where bascule or lift spans are employed, then it must be increased in length of base until the maximum allowable stresses and pressures are not exceeded. Then, by means of the cost of abutments, piers and spans, the relative costs of various designs may be determined, the weight of the spans being calculated from reliable formulas or from actual stress and section diagrams. The whole process may be somewhat shortened by using the "Ottewell" formula given in the writer's treatise, or that given in "Merriman's" Vol. III. The design of the approaches may be a factor in the relative costs, and in such cases must be included in making the comparisons.

TYPE OF SUPERSTRUCTURE AND DETAILS OF DESIGN.

The type and design of the superstructure are such a large factor in its cost that they must be fully decided upon before beginning any of the above investigations, and then all designs can be compared upon a common basis. Where the grade is high above the stream, with plenty of clearance for floods and navigation, deck spans of some type are of course the most economical to employ. Where the bridge is high or where the falsework is expensive for other reasons, it may be best and cheapest to use an arch or a cantilever design. Where very long spans are necessary either the cantilever or suspension bridge must be used. Where the clearance for high water or navigation is limited through trusses must be used, although it will often be best to use cantilever spans instead of simple trusses. Through arches or half-through arches may often be employed with good results, both architecturally and economically.

The economical design of the superstructure of a bridge requires careful consideration as to the style of trussing, panel length and truss depths. Longer panels and deeper trusses are more economical for modern heavy loads, but for plate and riveted lattice girders the depths are usually much less than for regular trusses. The span lengths for cantilever bridges should always be decided by careful mathematical analysis, as well as the lengths of the cantilever arms, suspended spans and depths. The height of towers and the depth of stiffening trusses for suspension bridges must also be carefully analyzed.

The design of movable spans should be carefully considered, not only with regard to first cost, but as to cost of operation and maintenance as well; but one should not get the idea that revolving draw spans are out of date, for in many locations they are the cheapest and the best to employ. Then there are locations where bascule spans are the best from every point of view, but where used they should harmonize with the remainder of the structure if there are additional spans; but in any case those forms must be abandoned that have no pretensions to architecture or beauty, and the same thing may be said with regard to other types of movable spans.

The details for ordinary spans have reached a practically permanent basis, so that standards, at least as to type, are usual. Where the structure is of unusual size like the Blackwells Island Bridge, the Forth Bridge, or the Hell Gate Arch, each member must be the subject of critical analysis and study. Lack of such study and analysis was the cause of the failure of the first Quebec Bridge. Great suspension bridges are of necessity special problems throughout and must be studied with greater care than any other type.

The floors of railway spans are usually of a standard type, either having steel stringers and ties or a steel trough floor with ties laid on the ballast; but the types of floor and paving for highway bridges are so numerous that it is often a grave question what type to adopt. The cheapest floor that should in any case be used on a good bridge is one having steel joist with spiking strips bolted to them, to which the floor plank is spiked. The very best floor is undoubtedly a reinforced concrete slab, with from 1½ to 2 ins. of sheet asphalt surface, although it may be wise to use a creosoted block surface, the thin blocks being set directly in asphalt on the concrete and filled with sand or grouted.

Having discussed those things which have to do directly with the economy of bridges, with masonry piers and steel superstructures, we may well discuss structures built entirely of stone, of concrete, or of reinforced concrete. Such structures must be fully designed and a careful estimate of cost made in order to make any reliable comparisons. The cost of stone bridges is, of course, the greatest of any type which might be considered, and in the case of the Knoxville Bridge where the cost of the arched cantilever was only $250,000 the bids for the stone arch design were about $1,500,000, thus making it out of the question to use stone where low first cost was a necessity. The same is true to a very great degree with concrete arches, but when reinforced concrete arches are considered, they may be designed to cost but slightly more in many instances than steel bridges, and should be carefully considered where a permanent and artistic structure is demanded.

BASIC PRINCIPLES OF AESTHETIC DESIGNING.

There are, it is true, no orders of architecture for bridges as for buildings, nor is there possible any classification of styles for particular ages, as in the case of building architecture, but basic rules must be observed, whether the design be for a building or for a bridge.

Simplicity, harmony, symmetry, and proportion must be regarded in any design that would have any pretensions to beauty or to architectural effect. They are the fundamentals of true architecture, no matter what the structure may be to which thought is to be applied in its design.

Simplicity means first a truth telling structure, no subterfuges about the lines of stress, no covering up of a concrete structure with a stone facing, no frivolous or inappropriate details, but a strict adherence to the necessary features, whether they are to carry the loads or to ornament the bridge.

Harmony is essential to a pleasing design, for without it the structure would be distasteful. Harmony not only as between substructure and superstructure, between the various spans, between the spans and the approaches, between the utilitarian features and the ornamental details, but also with the surroundings.

Symmetry may or may not be essential to a pleasing design, although it usually is necessary if a truly architectural structure is to result. Where the bridge has a great length, unsymmetrical features are not so noticeable as in a shorter one which may be all taken in at a glance. As is often true in a landscape, balance may frequently be secured by including unsymmetrical features. For example, in a bridge which contains a draw span or other unbalanced feature, some feature may well be introduced in another part which will tend to restore the balance, even though symmetry does not result.

Proportion is necessary in order that the three preceding principles may be realized; and usually when the economical proportions have been determined, they are pleasing. However, in many cases modifications must be made to reach the point where economy and beauty can both be satisfied. Proportion of details employed for ornamentation is quite another thing, and to be harmonious bridges must have the proper proportion.

Examples may be seen in very structure of the proper application of some of these principles, but more often we find one or the other flagrantly violated, so that the remark of C. Shaler Smith, that Dean of American bridge engineers, to one of his assistants should be well remembered by him who would reach high rank in his profession—"most bridges are examples of what not to do." Seldom if ever do we find a structure that complies with all of the fundamentals, although many bridges approach so nearly to the ideal that careful consideration and analysis is necessary to determine just where improvement could be effected.

Simplicity is best illustrated in its pleasantest features by suspension bridges and arches with no decorations or embellishments of any kind. Harmony is best exhibited where no part of the structure seems to be extraneous, and where the structure seems part of the surroundings. Symmetry in its simplest form is where one-half of the structure is exactly like the other. Proportion is most nearly reached when the structure is most pleasing and the truth expressed most accurately.

The basis for a real architectural system for bridges must come of course from building architecture, and on studying the columned or arched facades of buildings we find an uneven number of openings or arches are employed in the great majority of the world's notable and best pieces of architecture. Where there is an entrance it is nearly always in the center, with one or more arches symmetrically disposed on each side.

Careful study and analysis of the examples of Egyptian, Roman and mediaeval buildings discloses the fact that such an arrangement is most pleasing to the senses, and where it has been violated the design is unpleasant. This, then, we may take as the starting point of any design, an opening instead of a pier at the center, with the remainder of the structure arranged symmetrically on each side. Carried to its logical conclusion, where there is an approach it should have an uneven number of openings, and where there are spandrel arches employed in an arch bridge, they should be of an uneven number.

DISCUSSION OF SOME EXISTING BRIDGES.

The Knoxville arched cantilever, designed by the writer, was of five main spans and two anchor arms, thus giving an opening at the center and a perfectly symmetrical structure, except that one abutment was longer than the other, but this is not apparent to the eye in a structure a third of a mile long. Economy was violated in the depth of the anchor spans in order to make the bridge harmonious and the basic rules were all as nearly adhered to as is often the case.

The Market St. arch, at Youngstown, Ohio, designed by the writer, was a very difficult problem to solve, both on account of the side spans having to clear the railway tracks and on account of the 4 per cent grade of the roadway. The design, however, is symmetrical with the exception of the grade and the approaches, and it was made quite harmonious by carrying the sub-trussing of the side spans through at the same elevation as the lattice truss over the arch.

Comparing the Memphis Bridge, by Morrison, with the Thebes Bridge, by Modjeski, we can readily see how much is gained in architectural appearance by the symmetrical arrangement of the spans in the Thebes Bridge. Comparing the approaches of the Thebes Bridge with the approaches of the Forth Bridge, we see how much more in harmony with the main structure are the approaches of the Forth Bridge than those of the Thebes Bridge, although considered alone the latter are of the best design architecturally.

The Grosvenor Dee Bridge, at Derby, England, with its 200-ft. masonry span, is one of the great bridges of the world, but the paneling of the abutments and spandrels, and indeed all the decorations, are so out of harmony with the great span that they dwarf it and ruin the design. Compared with similar details of the Eden Park reinforced concrete arch, Cincinnati, Ohio, we find such ornamentation entirely appropriate and harmonious for the smaller span.

European bridges are more often well designed architecturally than those of other countries and the great bridge over the Rhine, at Bonn, Germany, is an example where simplicity, harmony, symmetry and proportion are all as fully met and satisfied as has ever been the case in any bridge structure.

The Camelback Bridge in the Imperial Palace grounds at Pekin, China, is also one of the most perfect of the world's bridges from the architectural point of view and satisfies the cardinal requirements of design.

The best design in the United States, in the writer's opinion, is the Connecticut Ave. Bridge, in Washington, D. C., with its five great concrete arches. Very little fault can be found with the design, except the inappropriate decoration of the wing walls of the abutments. The designing of harmonious and appropriate details in the proper proportion is a study in itself and is entirely beyond the scope of this paper. However, should designers carefully observe the cardinal principles herein laid down much more pleasing structures would result and a great stride forward be made in bridge engineering and architecture.

The necessity for good construction, especially of foundations, must not be overlooked, and no matter how chaste and beautiful the design, it is as nothing if not of first-class engineering and construction.

Laying Out the 5,327-Ft. Lethbridge Viaduct in Alberta, Canada.

The laying out of a long viaduct is an important factor in its construction, yet engineers seldom record the methods and equipment used for this part of the work. It is necessary that the pedestals and abutments be very accurately located, and to secure this high degree of accuracy requires the greatest care and the use of special equipment for each project. The following article on the procedure adopted in connection with the laying out of the Lethbridge Viaduct, in Alberta, Can., is based on data taken from a paper, by Mr. B. Ripley, engineer of grade separation, Canadian Pacific Ry., in a recent issue of the Canadian Engineer. This viaduct is a steel structure having a total length of 5,327 ft. 6 ins. and a maximum height of about 320 ft. above the bed of the stream. It consists of a series of 67-ft. deck girder tower spans and 100-ft. intermediate spans. An outline drawing of this structure is given in the May 20, 1914 issue of Engineering and Contracting.

LOCATING HUBS AND RUNNING BASE LINE.

On the center line, which was tangent throughout, permanent hubs were established at *W* and *F* (see Fig. 1). These points were near enough to the ends of the viaduct to be of value throughout the building of the bridge, but were far enough from the structure so that they would not be disturbed during construction. Holes, similar to post holes, were dug to a depth of between 4 and 5 ft., and pieces of 8x8-in. timbers were inserted in these, so that about 2 ins. projected above the surface of the ground, which had been carefully leveled off. These were firmly tamped around with concrete, and after being centered they constituted the hubs from which the true center line was established.

These hubs were securely fenced, leaving a sufficient space within the enclosure to permit the setting up of a transit. By means of a special construction a sighting rod was always left standing over the center of each hub, the former being established on a small brass brad. This sighting arrangement saved considerable time, as otherwise it would have been necessary to have sent a picket man to the hubs every time they were used. As it was, the sighting rods were inspected from time to time to see that they were in true position, and were always carefully replaced after having been removed for a transit set-up.

After having carefully adjusted the transit, it was set up on point *F* (see Fig. 1). Hubs *E*, *C* and *X* were established by using point *W* as a foresight, then *T*, *S* and *R* were established from the west end of the line, using *F* as the foresight. All hubs were established in a manner similar to that used in placing hubs *W* and *F*.

In centering these points there was often found a great difference between their elevations; therefore, in addition to having the lining hair of the transit truly vertical and the standards carefully adjusted (so that a vertical plane would be followed in depressing or elevating the telescope), double centering was used to eliminate errors.

After having these main hubs or stations set up at the most commanding or useful points, station X was chosen as the starting point for all measurements. There were several reasons for this. From X there was an excellent opportunity to get a sufficiently long base line for triangulation. More of the center line could be seen from this station than from any other point. At it, also, the triangulation base could be laid out at right angles to the center line of the bridge, making it equally valuable for work in the direction of W as in the direction of F. The ground was fairly level, affording a good opportunity for measuring and checking the base line; and as it was of the most importance to have the river work started first, hub X would be close at hand and no delay would be occasioned in laying out excavations for the river piers, the contractors for the substructure being already on the ground.

Although the center line of the bridge had been measured both by the locating engineer and by the engineer making the preliminary survey from which the bridge was designed, no measurement across the gorge was made with sufficient accuracy for the construction of the foundations. It is of interest to note that, during the winter of 1906-1907, a special party was put into the field to make a preliminary survey of the site just referred to, and, even though extreme care was used, the final measurement, made with a steel tape, was slightly over 3 ft. in error, as was found after the final measurement had been made.

The banks of the gorge were very steep and irregular in places, and it was decided to start measurements in both directions from X, so that any error in the measurement of the center line would be distributed on both banks —instead of being thrown upon one.

The actual work in connection with the laying out of the substructure was commenced during September, 1907, but at that time no detailed plans of the structure were available. Therefore, it was not safe to figure on any small errors in the total length of the viaduct being taken up by the erection of steel. It was therefore decided that nothing would be left undone in the matter of securing a correct chainage throughout the whole length of the bridge, and that nothing in the way of laying out the different piers or bents should be done until this length was finally decided upon.

Various systems of measuring were considered. An unsupported steel tape was considered of little value, on account of the prevalent high winds. (It should be mentioned here that the laying out of the work was commenced during a windy period, which lasted until late in the following April.) Neither was it considered advisable to use a supported tape. Therefore, both on account of the wind and the fluctuations in temperature, something less sensitive to both was required; and a graduated wooden rod was chosen.

STANDARDS OF MEASUREMENT.

A well-seasoned piece of cedar, 16 ft. long and 2x4 ins. in section, was obtained, and this was dressed to 1⅛x3 ins. A supply of brass plates 2½ ins. long, 1 in. wide, and about 3/32 in. in thickness, beveled on one long side, was made and set into the rod to receive graduations. About 3 ins. from the end of the rod, the first one of these was placed so that the surface of the plate was flush with the surface of the rod. This was marked with a very fine scratched line terminating at the beveled edge and designated zero. A plate was then put on at the 12-ft. mark, and from this point to the end of the rod plates were put on at intervals so as to receive graduations at every tenth of a foot from the 12-ft. mark to the end of the rod.

A 100-ft. steel tape was obtained, and this was compared with the standard of the company. When supported throughout its entire length, at a temperature of 70° F., with a 10-lb. pull, it was found to be 100.025 ft. in length. With this it was decided to establish a 90-ft. base, from which to graduate the measuring rod for 15-ft. measurements. A suitable piece of ground was found, and a heavy 12x12-in. post, about 6 ft. long, was set about 5 ft. into the ground, so that the top projected 12 ins. above the ground surface. A similar post was then placed at the 90-ft. mark. Between these, other smaller stakes were firmly set and on these 2x4-in. scantlings were nailed so that a straight line could be run on the level surface. The scantlings were left free from the 12x12-in. end posts, so that any contraction or expansion caused by differences of humidity would not affect the main hubs on which the 90-ft. standard base was laid out. A fine brass tack or brad, similar to those used by a shoemaker, was placed in the first post and a fine scratch was made across its face. This was the zero of the standard 90-ft. base. The tack at the 90-ft. point was set after the tape was stretched with the proper tension, and after calculations were made for any error in its length and for differences of temperature. It was marked, as was the zero end, by making a very fine scratch across the tack. Tacks were then lined in at 15-ft. intervals on the scantling edge which had previously been used to support the tape.

The 15-ft. point was marked very lightly on the proper plate on the measuring rod, as accurately as it could be done with the steel tape. The measurement of the 90-ft. base was then attempted with the rod, and the 15-ft. graduation was changed until six lengths of the rod exactly reached the 90-ft. mark on the standard base. When this was accomplished, the rod was considered correct at its 15-ft. length. The other graduations were then put on by using the common engineers' boxwood scale. The intention in measuring the length of the bridge was to place stakes at 15-ft. intervals, thereby using the 15-ft. graduation, as it was considered the most accurate of the graduations. Stakes were placed at other intervals from 12 ft. to 15.5 ft. only when it was impossible to place the regular 15-ft. stakes. The rod was also fitted with a reliable level bubble.

MEASUREMENT OF BASE LINE.

The measurement of the distance X to C was then attempted, and as the ground was fairly level it was possible to measure horizontally and without the use of a plumb-bob, with one or two exceptions. Stakes were carefully lined in, being stationed at 15-ft. centers and leveled at the same time. After about 500 ft. of the 15-ft. stations were set, it was found necessary to start a new level of stakes about 2 ft. lower than the first set, as the ground was found to be falling slightly in the direction of C. It was also found necessary, after this level of stations had been set for about 600 ft., to raise the remainder of the distance to a higher level. Measurement was carried from one level to another by using a fine plumb-bob protected by a canvas wind shield.

The method of measurement was as follows: Hub X, the initial point, was set in concrete, being deeply planted in the ground as a safeguard against its being shifted, and a fine tack with a scratch across its face was made to indicate its chainage. The first set of level stations was measured after two transit points had been taken on the head of each stake and these joined by a pencil line. This was done in order that the transit need not be used while the measuring was being done—the transitman could therefore keep notes carefully and could watch the work. With the zero end of the rod at the starting point, a tack was placed in the first station so that the 15-ft. distance would come upon its head, a fine scratch in the form of a sharp arrow was made on the tack with the small blade of a pocket knife, and this was checked to see that the 15-ft. graduation on the rod exactly coincided with the point of the arrow on the tack. To avoid confusion, no two marks were made on the same tack. The rod was then moved ahead to the second pair of stakes, and so on, until a portion of the line was measured; but in no case was this distance too great to be checked three times during the same day. The measurements were recorded by the transitman. The man at the zero end of the rod was then moved to the front end, the front-end man was moved back to take the rear end, and the measurement was checked carefully throughout. If any discrepancy was found the recorder took one end of the rod and both rodmen jointly took the other end. When any differences were found new tacks were used to receive the scratches. After a chainage was decided upon the measuring rod was again checked on the standard 90-ft. base, and, if found to be correct, the chainage previously agreed upon was allowed to stand.

After a final measurement had been agreed upon as far as hub C, a triangulation base, X-A, was laid off south of and at an angle of 90° with the center line of hub X, and this was carefully measured and was found to be 759.270 ft. in length. By triangulation, X-C was found to be 1639.115 ft. long as against 1639.110 ft. as measured with the measuring rod (see Fig. 1).

From C to E a great deal of steep and irregular ground was encountered, and for several reasons it was decided that no horizontal measurements should be taken. Some of the ground was as steep as 1 to 1½, and it would not have been practicable to have shielded the plumb line each time a measurement was taken. Slope measurements were taken in all cases, careful levels being taken over the stations and the horizontal measurements calculated. Wherever it was possible with a little grading to get two or more stations on the same slope, this was done. This procedure simplified calculations and also reduced the liability of error. On some slopes, which were somewhat uniform, it was possible, with a little grading, to get as many as five or six stations on the same slope. The tops of these stakes were carefully lined in to the uniform slope by using a level and a target rod. Wherever a change was made from one slope to another, the station where the change took place was called a "change," and this word was written on the stake so that when levels were

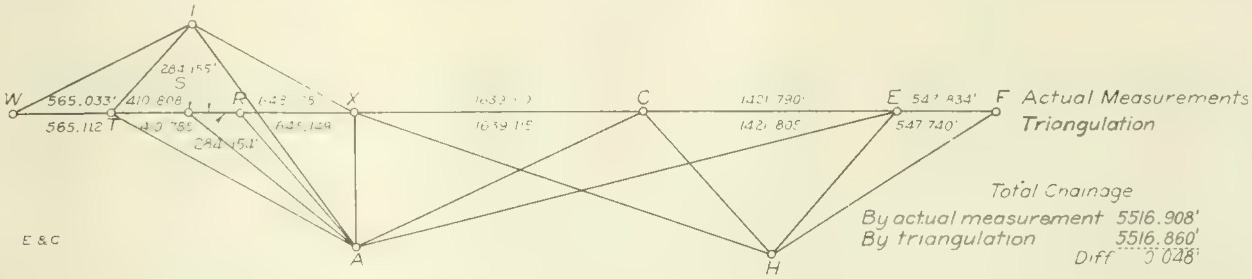


Fig. 1. Triangulation Chart Showing Measured Distances and Those Obtained by Triangulation for Lethbridge Viaduct, Alberta, Canada

taken for calculation of the horizontal measurements the respective elevations of the changes only were required. The measured chainage of this portion of the bridge length was 1421.790 ft., as compared with 1421.805 ft. which was computed from triangulation.

From E to F, the distance measured with the measuring rod was 547.834 ft., while that obtained by triangulation was 547.740 ft. The west end of the work, which was in the direction of X-W, was measured and triangulated in a similar manner, the only difference being that point W was not visible from either X or A. For this reason, a secondary base line had to be established by triangulation. From this secondary base, the distance from T to W was obtained. The fact that W was not accessible from X will perhaps explain the discrepancy of 0.079 ft. between the actual and triangulated distances between T and W, as a very slight error in the alignment of T (which was accessible) would account for this difference.

Actual measurements were relied upon for laying out the work, and triangulation was used as a means of detecting any large errors which might possibly have come into the actual measurements due to the shortness of the measuring standard used. The actual distance measured with the rod was 5516.908 ft., which compared favorably with the triangulated distance, 5516.860 ft., the difference being only 0.048 ft. The measuring of the center line was considered the most difficult part of the work, and the actual measuring of the length of the bridge (slightly over a mile in length) took about six weeks of very trying time, the high winds proving especially troublesome.

LAYING OUT THE FOUNDATIONS.

As it was necessary to proceed at once with the excavations, a great many of these were laid out immediately. The "bent" hubs were established by laying off the proper distances from the "change" stakes, the chainage of which had already been established. Owing to the number of haulage roads that were to have been opened and to the number of sightseers who were constantly walking about it was decided to have all "bent" hubs protected by a railing supported on four good posts set firmly into the ground. This enclosed a space sufficiently large to allow the setting up of a transit. All intermediate stakes were pulled out, both to avoid confusion and to give the workmen, teams, etc., an opportunity of working without disturbing the stakes. The preservation of permanent stakes and hubs was an important factor on this work, and a great deal of pains was taken to protect them. The "bent" hubs were also referenced by means of other hubs set out at considerable distances and approximately at right angles to the center line.

In laying out the foundations it was only necessary to turn off an angle of 90° from the center line, and to measure the given distance to the center of the particular foundation. This was done with the tape, on fairly level ground, but the measuring rod and plumbing device were used on irregular ground. This work was generally done before the ground had been disturbed. By setting up the transit over the center of the foundations and turning off other angles of 90°, four hubs were set which were sufficient for the contractors to carry out the work. When the ice was out of the river, the river piers were located by triangulation. When it was possible to lay out piers from the ice, this work was done in a similar manner to that used for the land piers.

LEVELING PROCEDURE.

The initial bench mark was established from preliminary location bench marks, and inside the enclosure at hub X a hole (similar to one which would have been dug for a telephone post), was made. Into this a ⅝-in. steel rod was driven so that its top would be slightly below the original ground. Then concrete was dumped into the hole around the steel rod. The hole was then filled with earth, and the bar was carefully referenced, so that no unnecessary digging would have to be done in finding the rod. After the concrete had firmly set the rod was examined to see that it had a well defined top, and a bench mark was then established on it. Other bench marks were established at convenient points, from which, with one "turn," elevations could be given for the finishing of a pedestal near them. It was so arranged that it was never necessary to make more than one turn from any bench mark in setting elevations for the finishing of the tops of any of the pedestals.

In leveling to establish a system of benches, difficulty was had in getting fine calm days, and it was rarely possible to get two good days in succession. A "Gurley" level and a sliding target rod were used. The levelman, after reading the rod, directed the rodman to set the target. This was then read again by the leveler, and, if correct, both rodmen were instructed to read the target as a check on the leveler himself. As another precaution, after taking the final reading on the rod, the leveler was instructed to see that the bubble of the level was in the center and that in focusing no jarring was done to the level. The level was always tested for adjustment every time before using—twice a day.

The levels were checked, from one side of the gorge to the other, several times, and the results were found to be entirely satisfactory. The writer considers, for similar work, that a one-piece target rod is superior to the two-piece one which was used. With the former there would be no danger of the slipping which has to be guarded against with the two-piece rod. Constant use also has a tendency to wear the mechanism of the two-piece rod, thereby making it less accurate for careful work. As the sun was fairly dull at this time of the season, no sun shade was used. Precautions were taken to keep the leveling rod in a place where the humidity was practically constant.

Owing to the design of the steelwork it was necessary to have a precision of ⅛ in. in the finishing top of the piers. Points were set for the forms, and after considerable concrete had been put in points were set with fine nails for the finishing of the top of the pier.

The transit used was a "Gurley" light mountain transit. The horizontal limb and verniers were graduated to read to 20 seconds, and attached to the standards were special magnifiers for reading both limbs. These were attached to the standards by means of universal three-joint arms, thus allowing the lens to be placed over any point on either vernier. To facilitate further the measuring of the angles required in triangulation, the process of "repetition" was used, so as to distribute any inaccuracies of the graduations of the horizontal limbs over several readings.

WATER WORKS

Design of New Steam Turbine Driven Centrifugal Pumping Machinery of the St. Louis Water Works.

The Chain of Rocks pumping station, located about seven miles north of the Merchants Bridge, is the low service station for the city of St. Louis. The water is pumped from the river at this station to the clarification and filtration works, thence flows by gravity to three high service pumping stations, where it is delivered into city mains under pressures varying from 80 to 125 lbs. per square inch.

The pumping equipment at the Chain of Rocks station originally consisted of two Worthington direct acting compound pumping engines, each having a capacity of 20,000,000 gals. in 24 hours, and four Allis-Chalmers crank and fly wheel compound pumping engines, each having a capacity of 30,000,000 gals. in 24 hours.

The safe working capacity of this station having been reached, it was decided to replace the 20,000,000 gal. Worthington pumps with pumps of greater capacity. The design of the new steam turbine driven centrifugal pumps installed is here described from information taken from a paper by Mr. L. A. Day before the Engineers' Club of St. Louis as published in the Journal of the Association of Engineering Societies for May, 1914.

It is only in recent years that the attention of water works engineers has been directed toward centrifugal pumps. Their low first cost, small maintenance charges, simplicity and compactness, compel instant recognition.

In considering relative values, not only first costs, but operative, maintenance and interest charges, as well, must be taken into account. Progress in the design of centrifugal pumping units has reached a point where serious doubt is created as to the wisdom of invariably installing enormous piston pumping engines, for in installing large pumping engines entailing a considerable expenditure, it seems wise to consider not only the possible life of the machinery, but its probable duration in view of present developments. It is true that the high duty steam pumping engines, with capacities for pumping large quantities of water, are superior to turbine driven centrifugal pumps in economy. However, a careful comparison between the two types of pumping units led us to choose the turbine driven type of pump.

Two 40,000,000-gal. reciprocating pumps for this service would have cost approximately $230,000, or $115,000 each. The duty in foot-pounds of work per 1,000 lbs. of steam of the reciprocating type of pump would have been approximately 150,000,000. Two 40,000,000 gal. turbine driven centrifugal pumps cost $55,000, or $27,500 each, with an average duty of 94,000,000. The average maintenance costs of reciprocating units, according to station records as kept on the Allis-Chalmers 30,000,000 gal. pumps is $780 each per year. It is safe to assume a maintenance cost not exceeding 2 per cent of cost of turbine pumps per year, or $550 each per year. The operating charges are considered the same for each type of pump.

In capitalizing the investment the following formula was derived:

$$\frac{A \times W \times H \times P}{D} + F(i + d) + L + M = C,$$

in which

A = Total number of gallons pumped per year.
W = Weight of a gallon of water.
H = Average total head in feet pumped against.
P = Cost of steam per 1,000 lbs. (13.4 cts.).
D = Average duty in foot-pounds per 1,000 lbs. of steam.
F = Total investment.
i = Rate of interest on investment.
d = Rate of depreciation.
L = Yearly cost of operating labor.
M = Yearly cost of miscellaneous expenses of operation.
C = Total cost per year.

Solving for C with both types of pumps we obtained a difference of $13,000 per year in favor of the turbine driven pump, which means that in a little more than four years the centrifugal pumps will have paid for themselves. It is proposed to install a third turbine driven pump of 40 or 50 million gallon capacity in this station in the next two years, in order to bring the safe working capacity up to 150,000,000 gals. per day, and if reciprocating pumps were necessary it would necessitate the erection of an additional building owing to the space they require. Needle ice suspended in the river water during the winter months and sand throughout the year are additional important reasons for installing centrifugal pumps at this station, as this type of pump is admirably adapted to handle water under these conditions with practically no trouble.

In July, 1911, a contract was awarded to the Dravo-Doyle Co., of Pittsburgh, for two De Lavel 525 HP. Steam Turbine Driven Centrifugal Pumping Units, each to have a capacity of 42,000,000 gals. in 24 hours, working under a total head, including friction in the suction and discharge pipes, of 46 ft.; a capacity of 40,000,000 gals. under a total head of 56 ft., and a capacity of 30,000,000 gals. under a total head of 63 ft. The 46 and 62 ft. heads are the minimum and maximum heads at this station due to the different stages of the river throughout the year.

STEAM TURBINE.

The De Lavel turbine is fundamentally the turbine invented by Branca, an Italian, in 1629, designed and constructed to suit the prevailing conditions, and consists essentially of a series of diverging nozzles from which the steam jets impinge upon blades fixed in the periphery of a wheel. The velocity of steam upon leaving the nozzles is very high, the speed of the wheel on these machines being approximately 10,250 r. p. m. In order to give the pumps the desired speed of approximately 610 r. p. m., the two shafts are connected by spur gears. These are of the double helical type, with the teeth cut at an angle of 45°, the pitch being small in order to produce quiet running and prevent vibration. The double helical type of gear eliminates end thrust, and the small pitch of the gears brings a large number of teeth in contact at one time, which very considerably reduces the fiber stress on the teeth as well as the pressure on their surfaces. The pressure is extremely light, and does not tend to squeeze out the film of oil, so there is no tendency to abrade the metal. The gears are set in a cast iron case with a tight cover. The noise is very much less than would be supposed, as the gears are cut and ground with the greatest care and accuracy.

The turbine wheel, the weight of which is 275 lbs., and diameter 31⅜ ins., is mounted on a very small flexible shaft (1-9/16 in.). The flexibility of the shaft is obtained by so proportioning the diameter to the length that the shaft will stand considerable bending without straining the metal beyond its elastic limit. The shaft is rigidly supported in the two reduction gear pinion bearings, but is without support from the inner pinion bearing to the outboard ball-seated bearing, and on this section of shaft the turbine wheel is mounted. The construction of the wheel is such that it is practically of uniform strength to resist centrifugal force. Directly under the rim of the wheel a small groove is turned, which makes this section the weakest part in order to prevent the wheel bursting in case of any accident which would allow the wheel to run away. This safety groove protects the wheel case and all parts of the machine from damage, so that the wheel in running away and bursting at the rim can cause no greater damage than the destruction of the rim and buckets. The buckets are held in the wheel by driving them in from one side. The buckets are drop forgings of high carbon steel. Monel metal buckets were tried in these machines but were found entirely unsuitable for St. Louis conditions, no less than six sets of buckets having been ruined due to the cutting action of the steam. The steam at the throttle of these machines is commercially dry, but not superheated, but when it is considered that approximately 20½ per cent of steam when expanding adiabatically in a perfect nozzle from 140 lbs., absolute pressure down to a 27-in. vacuum is converted into moisture, it at once appears very evident that the selection of a proper bucket metal for a single impulse wheel is very important. The steel buckets which are now in use seem to have solved the difficulty. The probable life of a steel bucket running continuously is about two years and the cost of rebucketing the same is approximately $85.

The speed regulation of these turbines is accomplished by a governor valve which throttles the steam supply. The governor valve is a double disc balanced valve and is actuated by a bell crank lever from the governor shaft. The governor itself consists of two small weights which are pivoted on knife edges and held in position by a spiral spring. When the speed rises above normal these weights spread apart and push forward the governor pin, which acts on the bell crank lever, closing the governor valve. An automatic safety stop is also provided in case of failure or damage to the main governor valve. It consists of a butterfly valve in the exhaust pipe controlled by the governor, so that in case of accident to governor valve, the butterfly valve is automatically closed, which confines the steam in the wheel case, creates a back pressure, and checks the speed of the turbine.

CENTRIFUGAL PUMPS.

Each pumping unit consists of two single stage double suction 24-in. volute pumps. The centrifugal pumps are of the simple type without diffusion vanes. The impellers are of the closed type. The water enters at the center and discharges at the periphery directly into the spiral casing which terminates in the delivery pipe. The volute or spiral case is given a variable cross section, increasing gradually toward the discharge opening, so that the lineal velocity of flow will remain constant, in spite of the increments of volume added opposite each point in the periphery of the impeller. While the double suction impeller practically eliminates end thrust a marine thrust bearing is provided at the end of the pump shaft to take care of any thrust which might occur, if for instance something should become lodged in one side of the impeller and prevent the proper filling of pump on that side. The material used for these impellers is manganese bronze and this metal has proved to be very satisfactory. However, one of the original set of impellers was cast of Government bronze and owing to the fact that a much better casting can be obtained with Government bronze than with manganese bronze, it is very likely that Government bronze will be used when the impellers require renewal. So far as wear is concerned both metals show practically no effects of the cutting action of sand after having been in service about 18 months. The impellers are finished on all surfaces, and the vanes are filed and polished by hand to templets. They are balanced for static and running balance. The pump case is horizontally divided through the center of the main shaft, and when the pump case cover is removed an examination of all working parts can be made without disturbing them, or any of the pipe connections. The only moving parts are the shaft and impeller, and the only wearing parts are the wearing rings, bearings, and shaft protecting sleeves. The pump shafts are made from hammer forged steel carefully finished. Leakage of water out of the pump or air into the pump is prevented by stuffing boxes on the shafts. The boxes contain soft packing, with a hollow skeleton ring in the middle of the packing to which clear water under pressure is admitted, forming an air tight seal and preventing a loss of suction. The shaft in the stuffing boxes is protected by a bronze sleeve which forms a bearing for the packing and can be easily removed when worn or scored, which means that the shaft proper is subject to no wear.

In order to prevent leakage from the discharge chamber back past the impeller to the suction chamber a water tight joint is provided which consists of a removable wearing ring attached to the pump case and another removable ring screwed on to the revolving impeller, the inter-meshing grooves form a labyrinth type of joint. The path of the water through the ring is tortuous and the loss of head at each turn prevents rapid flow, besides the water in the ring is set in rotation creating a counter centrifugal force opposing the leakage. The rings are made of Government bronze.

The pump shaft bearings are of the horizontally split, babbitt lined, ring oiled type. The babbitt lined bearings are not integral with the pump frame or shaft pedestal, but are formed in separate shells resting in the brackets or pedestals. The latter are separate and distinct from the pump case stuffing boxes, which prevents any possibility of water leaking past the stuffing boxes, and finding its way into the bearings and oil reservoirs.

Since it is practically impossible to maintain three or more bearings in alignment with a rapidly turning shaft, a flexible coupling is provided which divides the shaft into two parts, each part being supported by two separate bearings. The coupling consists of two separate steel flanges, one of which is furnished with a number of steel studs extending into holes in the other, the driving force being transmitted through the medium of steel lined rubber cushions.

TEST DATA.

Both of the pumping units were tested at the contractor's shops. The shop tests were conducted with the suction and discharge of each pump arranged independent of each other and the quantity of water was measured by the use of calibrated nozzles and Pitot tubes. The first unit when pumping at the rate of 31,420 gals. per minute or 45,250,000 gals. per 24 hours under a head of 57.19 ft. developed a duty of 111,600,000 ft.-lbs. The second unit when pumping at the rate of 24,735 gals. per minute, or 37,050,000 gals. per 24 hours, under a head of 61.45 ft. developed a duty of 101,000,000 ft.-lbs. A test on the final foundations was run on the first pumping unit on April 3, 1913, and the following results were obtained: head 53.06 ft., delivery 42,900,000 (29,800 gals. per min.), duty 96,800,000. On April 4, 1913, the following results were obtained: head 55.46 ft., delivery 39,040,000 (27,100 gals. per min.), duty 93,800,000. The delivery was measured by a Venturi meter which had been standardized by basin measurement. The original impellers, i. e., the impellers that were in the pumps when tested in the contractor's shop, were designed to work with a suction lift of 10 to 12 ft., and were designed to show maximum efficiency under this suction condition. The pumps, however, when placed on their foundations operated under a minimum suction head of 2 ft. This necessitated a change in the design of impeller, which consisted in slightly changing the vane angles and reducing the number of vanes to six instead of eight, also increasing the diameter of the impellers slightly. After changing impellers a test was run on July 21, 1913, and the following results were obtained: head 58.86 ft., delivery 38,480,000 (26,720 gals. per min.), duty 104,100,000. The head was then changed to 62.15 ft., resulting in a delivery of 36,458,333 (25,300 gals. per min.), and a duty of 101,600,000.

Water Coagulation and Its Effect on Character of Mechanical Filter Effluents.

One of the earliest objections to the rapid sand filtration of public water supplies arose from the preparatory treatment of the raw water by a coagulating chemical before filtration. Laymen generally, some sanitarians and many physicians viewed with apprehension the use of alum and iron salts in connection with the purification of water, fearing that some of the chemical so introduced finally reached the consumer to his detriment. This view is still held by many, particularly, of course, in communities using unfiltered supplies. This objection is very commonly raised when the filtration of previously untreated surface supplies is under consideration. In addition to the hygienic objection above stated it has been claimed that the use of coagulating chemicals impairs the water supply for steaming purposes. More recently "red water" troubles have been discussed at considerable length. Red water, so-called, is water to which a reddish brown color is given by suspended particles of iron oxide. Some claim that most complaints concerning red water come from cities using a coagulated and filtered supply. The present article discusses the actual effect of coagulating chemicals upon the quality of the filtered water. This is an exceptionally clear exposition of the subject and the discussion and data will be very useful to water engineers and superintendents who find it expedient to recommend the rapid sand filtration of public water supplies. The matter here given is taken from the report on the Improved Water Supply for the city of Wheeling, W. Va., by Messrs. George A. Johnson, J. Richard Kommer and C. B. Cooke, members of the filtration commission for that city.

Definition of Coagulation.—By coagulation is meant the use of certain harmless chemicals which, upon being added in minute quantities to water, produce a gelatinous precipitate that envelopes the mud, clay, organic matter and bacteria, and causes them to form into aggregates of such size that they can be much more easily removed, either by sedimentation or by filtration, than is the case with uncoagulated water. This treatment makes possible the adequate preparation of muddy water for final filtration, without the construction of large sedimentation reservoirs, wherein the removal of the mud in the water takes place very slowly and more or less incompletely.

Coagulating Chemicals.—The chemicals most commonly used for the coagulation of water are compounds of aluminum and iron, and of these potash alum, sulphate of alumina, and sulphate of iron are the most extensively employed.

The manufacture of alum is of great antiquity, and for many centuries this chemical has been used for coagulating water, as an aid to speedy clarification. The manufacture of alumina sulphate from bauxite and lime-clay is of more recent orgin. The sulphate of iron used in water coagulation is, for the most part, a by-product of iron and steel industries.

The choice between the different coagulating chemicals is partly based upon their efficiency as coagulants, and this refers directly to the percentage of available alumina or iron which they contain. In a general way it may be said that potash alum, and sulphate of alumina cost about 1 ct. a pound, while sulphate of iron costs about ½ ct. a pound. In this country the sulphates of alumina and iron are the most widely employed in water purification.

TABLE I.—APPROXIMATE PERCENTAGE COMPOSITION OF COAGULATING CHEMICALS.

Constituent.	Pure potash alum.	Sulphate of alumina.	Sulphate of iron.
Matter insoluble in water	.....	0.30	0.50
Alumina (Al_2O_3)	10.77	17.00	.....
Iron (Fe_2O_3 and FeO)	.....	0.25	57.50
Potash (K_2O)	9.93	.....	.....
Sulphuric acid (SO_3)	33.76	38.70	28.80
Water (H_2O)	45.54	43.75	13.20

In composition these chemicals show considerable variation, but they may be bought on a basis of a guaranteed percentage of available alumina or iron. The essential feature is that the chemical shall be basic, namely, that it shall contain more available alumina or iron than is required to combine with the dissolved sulphuric acid, and that it shall contain no free sulphuric acid. The approximate composition of these chemicals is as shown in Table I.

ACTION FOLLOWING ADDITION OF COAGULATING CHEMICALS TO WATER.

When either potash alum or sulphate of alumina is applied to water the chemical is rapidly and completely decomposed by the alkaline compounds naturally present in the water. Ordinarily this alkalinity is due to carbonates and bicarbonates of lime and magnesia. The sulphuric acid portion of the coagulating chemical displaces the weak carbonic acid of the alkaline compounds above mentioned. As a result soluble sulphates of lime and magnesia are formed and equivalent amounts of carbonic acid and alumina are liberated. The latter unites with the water and forms the white, insoluble and gelatinous precipitate, known as aluminum hydrate, and which has the property of massing together various impurities as mentioned above.

The amount of coagulating chemical that it is necessary to use depends upon the character of the water to be treated, especially the turbidity or color of such water. Very turbid or very highly colored waters frequently require several grains of coagulating chemical per gallon of water. At most plants, however, the annual average is not greatly in excess of one grain per gallon. This quantity is equivalent to about 17 parts per million by weight, or 143 lbs. per 1,000,000 gals. of water treated.

One grain per gallon of sulphate of alumina requires for its decomposition about 7 p.p.m.of alkalinity, depending upon the precise strength of the chemical used. This means, as explained above, that 7 p.p.m. of the carbonates and bicarbonates of lime and magnesia naturally present in the untreated water are converted into the sulphates of lime and magnesia.

The total hardness of the water is unaffected by the application of coagulating chemicals, such as sulphate of alumina, when decomposed by the natural alkaline compounds of the water to be treated.

The amount of carbonic acid which is liberated for each grain of applied sulphate of alumina is equal to about 3.5 p.p.m., or, roughly, one-fifth of a grain per gallon.

Under some circumstances the liberation of carbonic acid seems to increase somewhat the rate at which clear filtered and oxygenated water corrodes certain forms of uncoated (unprotected) metal. It is not a factor to be regarded with apprehension because the conditions are no more conducive to corrosion than are found in some of the best ground-water supplies of the country. If thought advisable, this carbonic acid may be removed by the addition of lime.

The average consumer cannot ordinarily detect any effect due to the addition of coagulating chemicals as they are bound to be decomposed in order to become effective. Neither can the ordinary consumer detect any changes as regards hardness, because the total hardness of the water is unchanged.

Where water is boiled or is used for steam-raising purposes, a water treated with a coagulating chemical is a little less satisfactory than one not so treated, but the effect is merely nominal when consideration is given to the large diminution in the accumulations formed within the boiler due to the elimination of mud where treated water is used.

As a general proposition all surface waters naturally contain a sufficient quantity of alkalinity to completely decompose all of the chemical which is applied for coagulating purposes. In some waters, however, the natural alkalinity is so low, particularly at times of floods, that it is necessary to make up the deficiency by applying soda ash or lime to the water.

Sulphate of iron is the ordinary commercial by-product in iron and steel industries. A somewhat higher grade of sulphate of iron is manufactured by a vacuum crystallizing process.

The use of sulphate of iron in water purification ordinarily requires for the precipitation of the iron the addition of lime. When added to a natural water, the chemical is decomposed in a manner somewhat similar to alum, except that the resulting bicarbonate of iron is partly soluble and more or less granular. By adding lime bicarbonate of iron is changed to the gelatinous ferrous hydrate, which in turn is oxidized into ferric hydrate. The latter is insoluble and gelatinous, serving well in the massing together of impurities. To obtain satisfactory results from the use of lime and iron as coagulants it is necessary to add sufficient lime to neutralize and precipitate the iron. This must be done carefully. The use of too little lime results in poor coagulation, caused by the incomplete precipitation of the iron, some of which is usually left in solution, appearing in the effluent of the coagulating basin. The use of too much lime results in the formation of lime incrustants, which are liable to cause trouble through deposition in the sand bed and in pipes and valves.

COAGULATION IN THE PREPARATORY TREATMENT OF WATER FOR FILTRATION.

Where waters in their raw state are normally clear and colorless, as is true in some parts of northeastern United States, practically no prepatory treatment is required before filtration. It is in connection with such waters as these, which more nearly approach in character the waters of western Europe, that purification by slow sand filtration, whose birthplace was in England, may be successfully practiced.

Waters which are comparatively clear, but highly stained by decaying vegetation, and which require treatment for the removal of bacterial life, may be purified by slow sand filtration. Such treatment, however, will remove only a relatively small part of the color dissolved in the water, and if it is desired to remove all of this color a coagulating chemical must be used. To use coagulants effectively and economically in connection with sand filters the period of preliminary coagulation and sedimentation should be at least 18 hours, and preferably 24 hours.

Many waters in the South and Central West are almost always more or less muddy. The waters of the Missouri, Mississippi and lower Ohio rivers are good examples of this class of water. The suspended matters which cause these waters to be muddy are largely mineral, but vary greatly in the size of the particles. Some are comparatively coarse and settle out readily when the water is allowed to stand. Others are of exceeding fineness, many of them less than 1/100,000 of an inch in average diameter, which is smaller than the ordinary bacterium. Turbid waters such as these, applied to filters without preliminary treatment, cannot be satisfactorily purified to the degree now demanded in this country.

It is in the purification of these muddy waters that some of the most difficult problems are found. The turbidity of many river waters show abrupt changes from that of comparative clearness following long periods of drought, when the amount of suspended matter it contains may be less than 50 p.p.m. (430 lbs. of dry mud per 1,000,000 gals.) to periods of great muddiness during freshets, when it may contain as much as 2,000 or more p.p.m. (17,000 lbs. or more per 1,000,000 gals.). At Pittsburgh the highest amount of suspended matter recorded in 1909 was 1,300 p.p.m., equal to 11,000 lbs. of dry mud per 1,000,000 gals. Changes from comparatively clear to very muddy water occur very suddenly at times, and the character of the suspended particles is subject to great variation.

To remove the great bulk of this suspended matter prior to filtration and to do it economically is not a simple problem. If it is done by plain sedimentation, then the basins in which subsidence takes place must be large enough to deal satisfactorily with the water when in its worst condition. If sedimentation is to be aided by preliminary coagulation, then the basins must be large enough to permit of adequate subsidence of the coagulated matters before the water reaches the filters. Otherwise the surface of the sand becomes clogged too quickly, requiring cleaning at prohibitively frequent intervals. Well baffled settling basins should have a capacity of 18 to 24 hours' flow, and if the basins are not well baffled their required capacity may be several days' flow in order to provide satisfactory results.

At Cincinnati and New Orleans careful investigation with a view of ascertaining the feasibility of using coagulants in connection with the clarification of water passed through quite large settling basins before application to sand filters showed the process to be entirely feasible, although its expense was substantially greater than for mechanical filters. It was found necessary to have a settling capacity following the application of the coagulating chemical equal at least to an average flow of 18 hours and preferably 24 hours, otherwise some of the gelatinous precipitate would remain in the water as it entered the sand filters and would clog the layer immediately at the surface so frequently as to make the cost of cleaning burdensome and abnormally expensive, as already stated.

The use of coagulants in connection with the preparatory treatment of water prior to its application to slow sand filters is quite successfully practiced in this country at Springfield, Mass., Ferncliff and Poughkeepsie, N. Y., Indianapolis, Ind., Washington, D. C., and elsewhere.

Exclusive of filters of any kind and with sedimentation alone, coagulation is used for the clarification and purification of quite a number of water supplies of considerable size in the United States, among which may be mentioned Omaha, Neb., Leavenworth, Kan., Kansas City, Mo., and Nashville, Tenn. In connection with rapid sand filter plants coagulants are always used, and at the present time there are some 450 such works in actual operation.

In the regular slow sand filtration processes, as practiced at Lawrence, Mass., New Haven, Conn., and a few other smaller places, no coagulants are made use of, and there should not be caused by such filtration any change in the hardness of the water. In slow sand filtration as practiced at Washington, D. C., Indianapolis, Ind., and other places, where coagulants are used in the preparation of the water for slow sand filtration the effect of the added coagulants is, or should be, the same as that noted in rapid sand filtration processes. In any event the total hardness, and hence the soap-consuming quality of the water, is neither increased or decreased, but the incrustants are slightly increased. Such increase in incrustants could hardly be detected by steam-raisers, judged by incrustations in steam boilers. In fact, it is decidedly probable that the elimination of the fine clay and silt obtained by coagulation would make the filtered water a more desirable boiler water than unclarified water.

The data in Table II are given as illustrative of the actual changes in hardness which takes place at some of the filtration plants.

The data in Table II show practically all there is to the proposition that coagulation seriously affects the water for steam raising purposes. The data show positively that this cannot possibly be so. The only detrimental change which takes place is the formation in the water of some 6 or 8 p.p.m. of incrustants for every grain per gallon of coagulant used. Such small increase could not be measured detrimentally by the steam user, and the undesirable features surrounding their presence in the filtered water are more than offset by the advantages inherent in a boiler water which is free from silt and clay.

THE RED WATER QUESTION.

Although the entire question of corrosion of metals by water has been given considerable attention by chemists and engineers ever since the general introduction of public water supplies in this country, of late years there has been considerable active consideration of the red water question and a tendency has been shown to attach a large measure of responsibility for the appearance of iron-rust in water to alleged increases in the corrosive properties of water caused by the use of certain chemicals in the course of its purification. Thus more or less recently there has been a great deal of discussion of the so-called "red water plague."

It is a well established fact that pure iron is in some considerable measure soluble in pure water; also that the presence of oxygen enhances the corrosive action of water on iron. The oxygen of course oxidizes the iron dissolved in water and forms iron-rust, and this imparts a reddish color to the water.

All surface waters in the absence of gross pollution or complete stagnation, contain some dissolved oxygen, in amounts ranging up to 8 to 10 p.p.m. In ground waters the amount of dissolved oxygen is generally less than in surface waters. Pure soft waters will corrode metal pipes, as will even pure rain water. Neutral salts in water will also cause corrosion, but are of considerable significance from the standpoint of the aid they offer in the formation of protective coatings in pipe.

Free carbonic acid increases the corrosive action of water on metals, and is usually present in all ground waters and in the majority of surface waters. The ground water supplies of Lowell, Framingham and Malden, Mass., contain from 7 to 25 p.p.m. of carbonic acid; and the surface supplies of other cities in the same state contain as high as 15 parts at times. At Pittsburgh, in 1912, the free carbonic acid in the Allegheny River water was at times as high as 18 p.p.m., and the average for the year was about 7 parts. At Springfield, Mass., in 1912, at the West Parish filter plant, the river water contained from 2 to 3 parts. The Scioto River water at Columbus, Ohio, contains about the same amount. At Little Falls, N. J., the colored Passaic River water in 1912 contained on an average 2 p.p.m. of free carbonic acid. On some days it contained as much as 15 parts.

So far as filtration processes are concerned, where slow sand filters are used without the aid of a coagulant, and where the water contains substantial quantities of carbonaceous organic matter, the bacterial processes active in these filters cause an oxidation of such matter. An increase in the amount of carbonic acid in the water is the result. In the case of rapid sand filters, or slow sand filters, where sulphates of iron or alumina are used in the preparation of the water for filtration, and where lime or soda are not also used, there is an increase in the amount of carbonic acid to the extent of some 3 to 4 p.p.m. for each grain of coagulant used per gallon of water treated. At Little Falls, N. J., in 1912, the river water contained on an average 8 p.p.m. of free carbonic acid, and due to the use of 1.5 grains of sulphate of alumina per gallon of water the free carbonic acid in the filtered water as it left the plant averaged 15 parts for the year, an increase of something less than 5 p.p.m. for each grain of sulphate of alumina used per gallon of water treated.

Within the limits of ordinary practice in the use of coagulating chemicals in water purification in this country, the assumption that the increase in free carbonic acid caused by such chemical treatment is responsible for the more or less common run of red-water troubles cannot be substantiated. Such troubles are encountered in cities having pure ground water supplies, as well as in cities using the old style slow sand filtration. The cause is not one common to all cases or localities, but is due as much, and in many instances more, to the natural character of the water supply itself, regardless of the way in which it is purified.

TABLE II HARDNESS OF RIVER WATERS AT VARIOUS PLACES BEFORE AND AFTER COAGULATION AND FILTRATION.

City.	Average for year	Kind of filters.	Amount of coagulant used grain per gal.	Parts per million: Total hardness. River water.	Total hardness. Filtered water.	Incrustants. River.	Incrustants. Filtered.	Increase of incrustant in filtered water due to use of coagulants.
Springfield, Mass...	1912	Slow sand	(d) .24	11	11	2	4	2
Little Falls, N. J...	1903	Rapid sand	(d) 1.38	31	30	7	14	7
Louisville, Ky.	1912	Rapid sand	(d) 1.73	95	91	29	38	9
Cincinnati, Ohio ...	1910	Rapid sand	(a) 0.84 (b) 1.79	76	89	32	41	9
New Orleans, La...	1912	Rapid sand	(a) 4.41 (b) 0.33	111	60	21	25	4
Columbus, Ohio ...	1910	Rapid sand	(a) 7.5 (c) 4.3 (d) 1.57	270	[illegible]	111	35	Softened.

(a) Lime. (b) Iron. (c) Soda ash. (d) Sulphate of Alumina.

Rapid Installation of Auxiliary Booster Pumping Plant and Pipe Lines at San Diego, Cal.

In the early part of May, 1913, it became apparent that a shortage of water in the city of San Diego was imminent. By the middle of the month the water level in the distributing reservoir was falling rapidly. There was plenty of water in the impounding reservoirs but the pipe line capacity under gravity flow was not sufficient to bring the water into the city. The normal difference in elevation between the water levels of the lower impounding reservoir and the distributing reservoir is 53 ft. This gives a hydraulic grade of 1 in 2,000 which produced a flow of 7,200,000 gals. per day. This quantity was somewhat below the consumption so the installation of a booster pump was determined upon. The rapid installation of the pump and pipe lines is here described from information taken from the latest annual report of Mr. H. A. Whitney, assistant superintendent and hydraulic engineer of the department of water of San Diego.

The problem was to go back far enough along the 24-in. wood stave gravity supply line to be sure that suction would not cause a vacuum on the line which would result in its collapse. It was figured that to go back 12,000 ft. would give sufficient static head to insure a pressure at all times on the pump suction. Another 200 ft. were added for safety, as it was of the utmost importance that nothing should happen to this only supply line to the city. A 10-in. wrought iron pipe was laid on the surface of the ground parallel with the wood stave pipe. It connected at its upper end with the latter pipe as above stated and connected at its lower end with the high pressure distribution pipes leading from the distributing reservoir.

It took exactly one week to get the machinery, install it, connect the suction with the 24-in. wood-stave pipe, lay 12,200 ft. of 10-in. wrought-iron pipe, lay 350 ft. of 12-in. cast-iron pipe, and connect it with a 30-in. main. The pipe for the work was ordered, hauled and laid within this period by six gangs of 15 men each, working two shifts per day. A loan of 8,000 ft. of 10-in. wrought-iron pipe, intended for gas mains, was obtained from the San Diego Consolidated Gas & Electric Co., 4,175 ft. were purchased by wire and shipped from Los Angeles, and the remainder was on hand.

On Saturday, May 17, an order was given to the owner of a 5-ton truck to haul a 150 HP. motor and pump from a point five miles away, which was loaned by the Cuyamaca Water Co. Through some misunderstanding the truck driver failed to locate the pump and the water department took over the job and loaded the equipment on a 3-ton Packard truck and had it on the site with the suction connected by Sunday morning. During this time it was found that the electric company had no transformer on hand to step the 2,200-volt current down to the 440-volt motor, so another trip was made and a 75 HP. motor using current at 2,200 volts was obtained. About 3,500 ft. of transmission line was laid while the pipe was being installed.

Connection with the wood pipe was made by a 10-in. saddle cast at the Standard Iron Works on the same day it was ordered. The specials used to reduce to the 6-in. pump suction and to increase again to the 10-in. wrought iron pipe were made up of material on hand. The final joint to the 10-in. pipe, which had been laid previously, was made by slipping a piece of 12-in. cast-iron pipe over the outside of the 10-in. main and calking the space between the two.

Five gulleys, some of them very steep, were crossed by the pipe line. In order to have the pipe conform to the contour of the ground it had to be heated and bent in position. There were very few pipe tongs of a size necessary for assembling this pipe, the consequence was the city had to borrow from the Sweetwater Water Co. and the gas company and lengthen its 8-in. size pipe tongs so they accommodated the additional diameter.

Sharp bends were made by a number of short lengths of bell-and-spigot pipe. Where the sections being laid by the different gangs met cast iron pipes were used as sleeves. These acted as contraction and expansion joints. In fact, there was so much contraction when the water was turned in that one of the joints pulled apart about 2 ins. and was repaired, without taking out all of the water, by using a 12-in. pipe as a sleeve. To put this on it was necessary to jack one section out of line. Kerosene was poured in and as much lead was poured as possible. Lead wool was then calked in on the lower side to effect absolute tightness.

The starting of the pump was felt immediately; the consumers who had previously been without water, obtained a sufficient supply for all practical purposes. Before starting the pump the pressure in the pipe line was 19½ lbs. at the site of the pump and 11½ lbs. at the far end. Following the installation the reservoir began to fill, showing a gain of 750,000 gals., which settled the difficulty until the permanent new booster pump was installed.

The Dual System of Water Supply and Distribution.

The dual system of water supply and distribution has frequently been discussed in technical literature. Whenever the subject has been brought up the more or less obvious disadvantages of the system have been pointed out. The subject is an attractive one apparently, however, and water works men show a disposition to revert to it from time to time in their discussions before the several water works associations. The latest discussion of the subject was by Mr. Henry C. Hodgkins of Syracuse, N. Y., in his paper before the recent annual convention of the American Water Works Association. The following matter is taken from his paper and from the discussion brought out by it.

The daily average consumption per capita for cities in this country will run from 100 to as high as 200 gals. Very few large places will average less than 100 gals. The maximum per capita consumption will run from two to four times the average. The average daily consumption per capita for domestic purposes will amount to from 25 to 30 per cent of the total average, and the amount required for culinary and drinking purposes will amount to from 3 to 5 per cent of the total. The amount of potable water thus required is very nearly a constant quantity from day to day. Filtration plants cost from 12 to 20 per cent of the total cost of the remaining portion of the water works in the case of mechanical filters and 20 to 30 per cent in the case of slow sand filters. In Pittsburgh, the estimated cost was 30 per cent and in Toronto 20 per cent of the previous cost of the works.

In a city of 100,000, Mr. Leonard Metcalf estimates the cost of that portion of a distribution system due to domestic consumption at 21 per cent, that due to industrial consumption at 32 per cent and that due to fire protection at 47 per cent. If the division is carried farther that portion due to culinary and drinking purposes would be about 4 per cent.

If the distribution system could be separated into distinct systems and at the above proportionate costs the problem of dual systems would be easy. But such is not the case. The cost of a dual system in terms for a complete system would vary widely, but as a general proposition I think it a fair statement that a separate system to supply the inhabitants of a city with potable water would cost from 25 to 30 per cent of the distribution system as generally constructed. In other words, that the cost of the dual system would be from 70 to 150 per cent of the cost of the filtration plant, varying widely according to conditions and according to which system of filtration is used.

There is a wide range in the cost of filtering water. Making due allowance for maintenance of plant, interest and depreciation, the cost would probably fall between 1 ct. and 1.5 cts. per 1,000 gals. Assuming a cost of 1.2 cts. per 1,000 gals., the cost of filtering

1,000,000 gals. per day would be	$12.00
40,000 gals. per day would be	.48
Difference per day due to filtration	$11.52

That is, if instead of filtering all of the water only the portion furnished through the dual system was filtered, the saving in the cost of filtration would amount to $11.52 per million. From this would have to be deducted the interest, depreciation and maintenance of the dual system which in a medium sized city would require not to exceed half of this saving, leaving the remainder for the additional cost of administration and saving to the department.

The pure water should sell at a much higher rate than the raw water for two reasons: first, to provide ample revenue to cover all charges due to the dual system, including service pipes and meters; and second, the price should be high enough to prevent the use of pure water for other purposes than those intended.

These figures are merely illustrative, but they are sufficient to show that the dual system is not impossible from a cost point of view, but that it is entirely feasible and in many instances the solution of a very difficult situation.

It is not claimed that the dual system should be universally adopted, but in places where there is a supply of naturally pure water sufficient to meet the culinary and drinking requirements, it is urged that it be supplied through a separate system, leaving the larger requirements to be furnished from the nearest and cheapest source of reasonably clean water.

There are many instances where towns have outgrown their water supply. The present supply being excellent in quality, but insufficient in quantity. Just such an instance exists in the writer's practice. The demands have increased beyond the supply and the capacity of the distribution mains. It has been recommended and considered with favor, that a duplicate system covering nearly half of the town be installed, furnishing a water more suitable for manufacturing purposes, but not fit for drinking. The original distribution to supply all of the needs of a portion of the residential section from the original source, and with the smaller mains extended to all parts of the town, supplying all the inhabitants with the excellent drinking water, it being the expectation that as the city grows the dual system would be extended until the entire city is covered.

Because the industrial growth of the town has caused a demand beyond the capacity of the supply, but which is still sufficient for all house supplies for even a much larger town, should the best of nature's supplies be discarded and replaced by a filtered supply?

The difficulties of supplying the ever increasing demand with a potable water are such that all naturally pure waters should be conserved, and it would seem that it is time that the scientific planning of dual systems should be commenced. It is not the purpose of this paper to criticise work that has been done or that is under way; but to emphasize a new departure in water works construction and management. There is no reason why the bottled purveyors should have the best of the business. Pure water, like other food products, should be conserved and delivered in the best possible manner.

The great argument against the dual supply has been the difficulty of controlling its use. It has been argued that poor people would use the cheaper water, and that many, especially new arrivals in the city, from ignorance would use the contaminated water. Every precaution should, of course, be taken to prevent the wrong use of either water. The drinking faucets should be of one handsome, ornamental design, properly labeled. All other faucets should be of the same and an entirely different and distinctive design. Greater care and inspection of all fixtures would be necessary, which would certainly be progress in the right direction.

In discussing the paper, Mr. Frank C. Kimball related the classic Lawrence, Mass., case as an argument against the dual system. In that case there was a filtered water supply for drinking and culinary purposes. The polluted river water was piped through the mills for ordinary use in washing and similar purposes. The polluted river water was piped through the mills for ordinary use in washing and similar purposes. The pipes carrying the polluted water were of one uniform color and were labeled. A penalty was imposed on the use of this water for drinking. Despite these precautions the mill operatives drew water from the polluted supply whenever it was more convenient to do so than to draw on the pure supply. This resulted in considerable sickness and several deaths and the polluted supply was abandoned by the mill owners.

Mr. F. W. Cappelen, city engineer of Minneapolis, said that it is the duty of every city to furnish a pure water supply to all its inhabitants. The quality of this supply must be so good that a child can drink it; so that anyone can open a faucet anywhere and be safe in doing so. Every inhabitant must be protected against his ignorance or carelessness. This cannot be done with a dual system.

Use of Electric Light for Detecting Leak in Casing of Deep Well at Galva, Ill.

The following description of the detection of a leak in the casing of a deep well at Galva, Ill., by means of electric light is taken from the paper on that subject by Loyd Z. Jones, city engineer, as published in the Proceedings of the Illinois Water Supply Association for 1914:

The city of Galva is situated on the divide between the Illinois and the Mississippi Rivers. It draws its water supply from the St. Peter sandstone which is found at a depth of 1,381 ft., and the wells are drilled to depths of from 1,477 to 1,525 ft. The log of the wells is as follows:

Materials traversed.	Depth in ft.
Soil and clay	62
Coal measures	155
Niagara limestone	882
Utica shale	1,030
Hudson River shale	1,055
Trenton limestone	1,381
St. Peter sandstone	1,475-1,525

A study of the logs of wells at Rockford shows that the St. Peter stratum falls 15 ft. per mile from Rockford to Galva.

When the well was first drilled, 20 years ago, the water rose to within 150 ft. of the surface, but for some time it has stood at 240-246 ft. below the surface. The pump cylinders are 300 ft. below the surface, and are always covered with water. The well is cased 110 ft. with 12-in. tubing, and below this level it is cased with 9-in. tubing to bottom. The joint between the casings is of lead.

In 1906 the quality of the water in the wells seemed to have changed, and it was thought that a leak had developed in the casing. Accordingly, the pump was taken out, and a cluster of three electric light bulbs was lowered into the well. The lamps were connected by a long wire to the lighting circuit, and were provided with a shade above. The lowering of this light into the well was followed by the aid of a field glass. It was found that water was entering through a leak in the casing at the lead packed reducing joint, 110 ft. below the surface of the ground. This was repaired.

In 1911 there were indications of another leak in the casing. The above process was repeated, and it was found that a leak had developed again in the lead joint.

The cause of the failure in the casing at this point is explained by the continual vibration of the earth, which is brought about by the running of heavy trains on the main line of the Chicago, Burlington & Quincy Railroad only 100 ft. away, and by the jar of our own pumps. The upper strata are of soft water soaked material. The pump and heavy masonry base are fastened to the top of the casing and the result is a rather top heavy structure somewhat analogous to a top heavy flag pole. The vibrations consequently tend to break the casing at its weakest point.

ROADS AND STREETS

Methods and Cost of Constructing Tamped Earth Base Bituminous Macadam Pavements in Southern California.

Contributed by Allen Hoar, C. E., Long Beach, California.

The development of bituminous macadam pavements in Southern California has been extensive. This is possibly due to the facts that here the climatic conditions make difficult the maintenance of water-bound macadam and here the most suitable material for this type of pavement is found. The long, hot and dry season robs the ordinary macadam road of its essential factors of life, drying it out and causing an excessive dust nuisance accompanied by rapid disintegration.

Fig. 1. Tamping Roller in Operation Consolidating Subgrade.

PREPARATION OF SUB-GRADE.

A permanent pavement should never be laid upon a new fill. No amount of rolling and watering will give the settlement which will take place naturally in the course of time. In making fills, all organic or vegetable matters should be thrown out, and as nearly a uniform material used as can be economically obtained, in order to get an even settlement.

The best method of preparing the subgrade is to plough to a depth of 6 or 8 ins., and compact by alternately sprinkling and rolling with a tamping roller, Fig. 1, until the soil is compacted to within an inch of the surface. The action of the tamping roller is to pack the earth into a hard crust, forming a solid earth arch, upon which the load is borne. The surface should be kept to cross-section by dragging, while tamping is going on. After tamping within an inch of the top, smooth by rolling with a 10-ton roller.

STONE FOUNDATION.

The foundation stone should be tested for hardness and abrasion. It must be clean and free from all clayey and organic matters that might prevent the uniform penetration of the binder. The size of the aggregate should range from 1½ in. to 3 ins. Care must be used while spreading to prevent the segregation of sizes. A uniformity of size of stone used makes impossible a good mechanical locking of the aggregate. The best results have been obtained by the use of the crusher run of rock with all the ¾-in. stone screened out.

The necessary thickness of the foundation depends entirely upon the traffic conditions to which the road or street will be subjected. After the proper thickness of stone has been spread (allowing for compression) it is rolled dry. Rolled dry, the stones are well locked together mechanically, and the interstices are left open to permit a deep and even penetration of the bituminous material into the foundation, forming a perfect film of binder over all parts of the aggregate and thoroughly binding them together.

BITUMINOUS BINDER.

Properties.—To obtain the best results only liquid asphalts (the residuum of natural asphaltic petroleums, having the lighter volatile matters removed by distillation) should be used as binders. These products should contain from 90 to 98 per cent of solid asphalt, but should not be made by fluxing solid asphalt with a lighter oil. They must have high ductile and viscol qualities and become solid when cold, and must not contain to any appreciable extent, highly volatile matters or free carbon. The presence of free carbon denotes an unstable composition, due to its chemical and physical characteristics, and gives to the bitumen a false consistency. Any great percentage of highly volatile matter causes the binder to remain soft and sticky, and prevent the "set" taking place. The liquid asphalt, when exposed to the air, should begin to set, or harden, but it must also retain its adhesiveness and resiliency. These are the three essential qualities of a successful bituminous binder: hardness, giving the durability and wearing quality; great adhesive powers, in order to hold the road metals together; and reailiency, promoting the general satisfaction and success of the finished pavement.

Application.—Liquid asphalt should be delivered for application at a temperature of not less than 350° F. The danger of burning has been found to be small by those experienced in this class of work, and it has been established that only at a high temperature can these heavier grades of asphaltic oils be liquefied sufficiently to allow proper penetration. A bituminous material injured by the application of heat at a temperature of from 50 to 75 degrees F. below its flash point, should be discarded as unfit for use, as it denotes unstable physical properties and the presence of free carbon. The formation of carbenes as a result of heating, is looked upon with suspicion. It is certain that when contained to any appreciable extent, they cause a shortness or brittleness which affects the durability of the pavement.

enough liquid asphalt should be used for perfect penetration and to cover every particle of the aggregate with a film sufficient to bind them together. Too much binder results in a soft doughy mat, in which the stones are only loosely held, owing to the fact that the thickness of the bituminous mat allows only a small part of its bulk to come in contact with the air and thus take its set; the bulk of the binder remaining soft and spongy, allowing the stones to rock under traffic, and eventually causing the road to ravel and rapidly disintegrate. Practice has shown that for the first application ½ gal. of liquid asphalt to the square yard gives the best results.

TABLE I.—UNIT COSTS OF 14,000 SQ. YDS. OF BITUMINOUS SURFACE IN GLENDALE, CALIFORNIA.

Item.	Cost per sq yd.	
Bottom course—		
Broken stone, 0.128 cu. yd. at $1.55	$0.1975	
Cost of hauling, spreading and rolling	.043	
	$0.2405	
		$0.2405
Oiling—		
First application—		
½ gal. oil at $0.023	$0.0115	
Hauling	.0056	
Applying	.0014	
¾-in. crushed stone	.0510	
Laying	.0243	
	$0.0938	
		$0.0938
Second application—		
¼ gal. oil	$0.0058	
Hauling	.0028	
Applying	.0014	
⅜-in. stone	.0270	
Spreading	.0170	
	$0.0540	
		$0.0540
Surface treatment—		
Oil	$.0058	
Hauling	.0028	
Applying	.0014	
Screenings	.0210	
Spreading	.0096	
	$0.0406	
		$0.0406
Total cost per square yard of surface		$0.4289

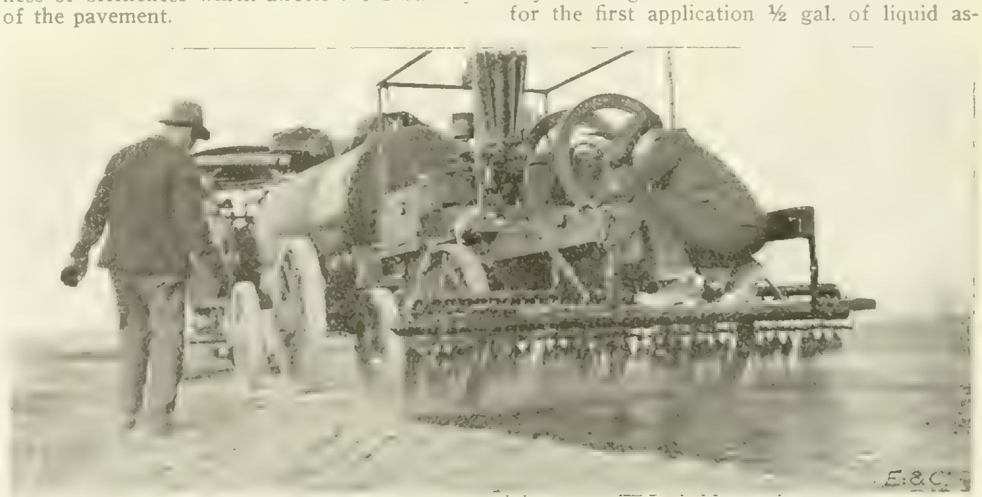


Fig. 2. Applying Bituminous Binder With an Atomizing Machine.

To secure the most satisfactory results, the binder must be applied under pressure, in a veritable spray. Although there are several pressure distributors on the market, the writer has secured the greatest satisfaction by the use of the Ward Atomizer. The liquid asphalt is applied by this machine, Fig. 2, under a pressure of 45 lbs. per square inch, in a very finely atomized condition which insures the greatest possible penetration and an even distribution. The binder must be distributed in an absolutely even coat. Just

WEARING SURFACE.

After the foundation has received its first application of binder, just enough crushed rock, ranging from ¾ to ½ in. in size, is spread to lightly cover the entire surface. This course should be well broomed into the interstices of the foundation and then rolled. Next apply the second coat of binder, using ¼ gal. to the square yard, and cover with just enough ⅜-in. stone to prevent the roller from picking up the surface as it passes over it. The seal coat should now be applied using ¼ gal. to the square yard and covering with enough ¼-in. screenings to absorb all excess binder. All spots which show evidence of bleeding after being rolled should be covered with more screenings and the road thrown open to travel.

Care must be taken the first month or two to watch for any evidence of bleeding, and, if it appears, to immediately cover the spot with fresh screenings before the traffic has a chance to pick it up. The writer has seen the destruction of the intersection of a new street by the neglect of those in charge to cover up a very small patch where bleeding had occurred. The tires of an automobile picked up a few of the sticky stones, leaving a small hole which gradually became larger as more vehicles passed over it. This was not cared for, and in less than three months after the street had been finished the whole intersection was very badly in need of resurfacing.

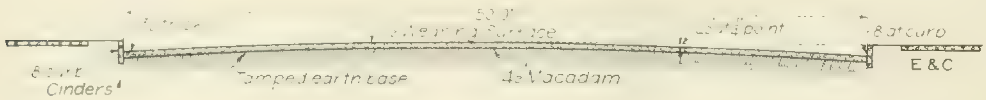


Fig. 3. Cross Section of Street in Glendale, California.

COST OF CONSTRUCTION.

In Table I is given the cost of constructing about 14,000 sq. yds. of this pavement in Glendale, Calif. The roadway is 50 ft. wide between curbs, Fig. 3, and has a macadam base 4½ ins. at the crown and 3½ ins. at the curb. One gallon of 93 per cent liquid asphalt was used in three applications. Crushed stone ranging from ¾ to ⅜ in. in size was used for the wearing coat, and the top covering was of ¼-in. screenings. The contract price for this work was $0.047 per square foot. A motor truck distributor was used for hauling and applying the binder. The average haul for the oil was six miles, and for the stone one mile. Labor was $2.50 per 9 hr. day and teams $4.50. The work was done under the supervision of Mr. Harry Lynch, city engineer.

TABLE II.—COST OF CONSTRUCTING ONE MILE OF THE FOOTHILL BOULEVARD IN LOS ANGELES COUNTY, CALIFORNIA.

Item.	Total.	Cost per sq. yd.
Grading	$ 534.89	$0.057
Stone, 2,253 cu. yds. at $1.65	3,717.12	.397
Hauling and spreading	691.42	.074
Rolling	118.24	.0126
Oil, 9,384 gals. at $0.0225	211.16	.0225
Hauling and distributing	52.55	.0056
Screenings, 97 cu. yds. at $1.60	154.61	.016
Spreading	49.74	.0053
Rolling	28.15	.003
Totals	$5,560.88	$0.536

Note.—Labor at $2.50 per day of 9 hrs., and teams at 4.50 per day.

Table II gives the cost of constructing one mile of the Foothill Boulevard, Figs. 4 and 5, in Los Angeles County. This road is one of the main traveled automobile roads in Southern California, and has been down four years. Nothing has been spent on it for repairs and it is in excellent condition.

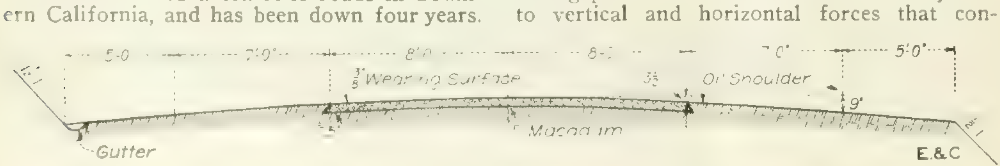


Fig. 4. Cross Section of Foothill Boulevard, Los Angeles County, California.

Nine million square yards of concrete roads and streets were constructed in 1913 and were widely distributed, the Northwest Central States leading with 922,522 sq. yds. and the Northeast Central division coming next with 538,000 sq. yds. The three Pacific Coast States constructed 423,000 sq. yds. Among the cities, Kansas City, Missouri, led with 250,000; Fort Smith, Arkansas, 163,000, and Sioux City, Iowa, 120,000 sq. yds.

An Analysis of Worn Out and Ravelled Macadam Surfaces with Suggestions as to Treatment.

The treatment of wornout and ravelled macadam surfaces is engaging the attention of many engineers at this season. Surfaces in many cases have become rough and irregular to such an extent that traveling over them is very disagreeable. Yet with their apparent defects they undoubtedly have a definite value if properly treated. An analysis of the causes of this condition was made by E. A. Stevens, State Highway Commissioner of New Jersey, in a paper before the 3rd American Road Congress which is short and as complete as information available warrants.

The word ravelling is used rather loosely. I shall consider it as the loosening of the bond of a road surface until the macadam stone lies loose and free on the road. By macadam stone, is meant, not the small stone used to fill voids and give a smooth finish to the surface, but the stone that constitutes the body of the road's surface. In macadam work this stone when compressed to its final form occupies about 60 per cent of the volume of the road surface. The 40 per cent of voids is filled in varying proportion with surface stone, screenings, stone dust, sand, earthy materials and any chemical binder used to "hold the road." The mixture of fine stone, sand and earth filling the voids has no appreciable tensile strength. Its duty is to wedge the macadam stone in place and prevent internal movement. Such a structure is called on to carry loads, to receive and absorb propelling thrusts. The road should be of sufficient depth to transfer the stresses thus imposed to the sub-base without serious internal movement and at unit pressure less than the resisting power of the soil. It is thus subjected to vertical and horizontal forces that contribute largely to ravelling. Let us briefly consider these.

A draft horse weighing 1,200 lbs. will have all his weight at one time on two feet. He will exert say 1 HP. at a speed of 4 miles. The vertical forces at the foot is 600 lbs., the

horizontal $\dfrac{23{,}000}{2 \times 4 \times 88} = 47$ lbs. These forces

are or may well be concentrated on a toe calk 2 ins. in width; the resultant force slightly exceeding 300 lbs. per linear inch.

A motor truck weighing, loaded, 16,000 lbs. and exerting at the wheel rims say 30 H. P. at 10 miles an hour, will with 60 per cent of load on rear axle, exert a vertical force of 4,800 pounds and a horizontal of 562.5 lbs. at each rear rim, the resultant being about 800 lbs. per lineal inch for 6-in. tire.

For a pleasure car weighing 4,000 lbs. with 60 per cent of weight on rear axle and exerting 40 HP. at the wheel rims with a speed of 35 miles, the vertical force at each wheel is 1,200 and the horizontal 214. The resultant is about 400 per linear inch for a bearing width of 3 ins.

At curves with high speed cars, the horizontal force is considerably increased, for it is impossible to so "bank" a curve as to suit the speed of all classes of traffic. In the case of wheels trasmitting vertical loads only, observation indicates but little dust raising from a road not overlaid with loose dust. Such a wheel will at the point of mathematical tangency have no velocity relative to the road; a vertical velocity is imparted to it and as any section leaves the surface it will raise with it any of the lighter particles that are loose and may come into contact with it. At the driving wheel there is a slight slip which in addition to lifting will throw particles backwards.

These are the forces tearing at the road surface. In some cases they are more than the road can stand. The horse's calk acts somewhat like a chisel. It will pry out the binding material between the stones as well as the latter themselves when the bond is

Fig. 5. View Showing Surface of Foothill Boulevard After Four Years of Service.

weakened. The driving wheels of motor vehicles push or suck any material thus loosened out of place. The binder loses weight as it parts with moisture; besides this, without moisture it also loses, not only its property of recementing itself under pressure but to a great extent, its binding power as well. The real work of transmitting the stresses due to traffic must be borne by the stone. These are merely held in place by the binder. The latter is gouged out by the horse, drawn or pushed out by the automobile tire, blown away by the wind, or washed off by the rain and a loosening of the upper stone results.

Even on a well consolidated road climate stresses impose a heavy duty. If, however, there be voids the risk of failure increases. Voids may be due to deficient rolling or to the rise of too much clay in the earthy binder. Lumps of this material will bridge the spaces between stones even under hard rolling. The bridge will break down eventually either from pressure or moisture or a combination of them. A void forms where the bridge was and continues rising until near the surface when ravelling results. The same may be true in the cases of too light rolling.

An ordinary water bound macadam may lose material by wear up to a thickness of about a half an inch a year without being overloaded. The thickness that can thus be lost will vary with conditions, one of which probably is the ratio of the maximum wheel load to the total tonnage borne by the road.

Whatever the rate of waste it must be replaced if the road is to be maintained.

To summarize the foregoing, roads ravel

from: (1) Improper construction, (2) overload, (3) neglect. In all cases these affect the binder.

Poor workmanship in construction can only be cured by remedying the original defects. These generally show themselves by small local depressions in the surface from which ravelling spreads, in certain cases at an almost incredible speed. In bituminous surface faulty material and thin spots generally show up clearly. In both cases the only remedy is to rebuild the work properly, if necessary, from the sub-base up. This is not strictly repair work.

In considering the effect of overloading and neglect it must be borne in mind that any given type of construction may be so maintained as to vastly increase its carrying capacity. The problem is largely one of economics and administration. As an illustration, assume in two cases the same foundation—let one road have a water bound macadam surface dressed with a heavy bitumen forming a sheet about ½-in. in thickness and the other a bituminous concrete (mixed method) surface of say 2 ins. The former will cost about $0.40 for stone and $0.15 for dressing, or $0.55 per square yard, and the latter about $1 (both exclusive of the foundations). With proper care the higher surface will last about three years under fairly heavy traffic, the heavier one an unknown period, but let us assume twelve years. The total yearly charges against the two may then be stated about as follows:

	Cts
Dressed Macadam—	
Interest on cost, 55 cts. at 4 per cent	2.2
Depreciation, one-third of 15 cts	5.0
Labor (patrol system)	1.9
Materials:	
Stone, ⅜ cu. ft. at 9 cts	3.4
Bitumen, ⅛ gal. at 12 cts	1.5
Total	14.0
Bituminous Concrete—	
Interest on cost, $1 at 4 per cent	4.0
Depreciation, 1/12	[illegible]
Labor	0.9
Material	0.8
Total	14.0

The fixed charges are 7.2 cts. against 12.3 cts.

The operating charges 6.8 cts. against 1.7 cts.

I do not claim that these figures are more than illustrations of the principle involved. They show a great saving in operating charges, those that show up in the yearly tax bill. The fixed charges, however, are just as real and must be met at some date.

For a road with 9,500 sq. yds. per mile the costs as shown in yearly tax charges, where depreciation and interest are not visible, would in this case be,

For the dressed macadam, per mile	$636.50
For the bituminous concrete	161.50

yet as shown above the real cost of the two roads is the same. This case affects our subject because the treatment of a ravelled road surface must depend on the system of maintenance.

In most communities the great consideration is the next yearly tax bill. If that can be kept down for a period the ultimate economy of such a policy receives but slight attention. It is generally easier to get money for a new road than for repairs. A road requiring a large yearly repair charge is condemned without a hearing. The road calling for heavy interest and depreciation charges may be an equally or even more expensive solution; but the interest charge is not so apparent and the depreciation charge is not made. This is simply putting off the day of reckoning which is sure to come. However, the troubles of those in charge ten years hence are usually lightly borne by the officials of today.

If we consider the structure of the road surface we can easily see that the 40 per cent of voids in the macadam stone will be filled somewhat as follows:

	Per cent
Surface stone passing 1-in. ring and caught on ½-in. ring	[illegible]
Screenings, passing ½-in. ring	15 to 10

When dry the clay is driven off to a greater or less extent as dust, washed away or splashed off as mud. Its place is supplied to some extent by detritus, the result of the wear of the larger and heavier materials. These also blow or wash away and the road loses its bond. If our road is not overloaded we can retain its usefulness by making good its losses, with proper materials in their needed proportion. It is here that the trained road man is most needed. Nothing can replace his experienced judgment.

In the case of a ravelled road having first determined that the road was well built we must decide whether the traffic is too great for the type of surface, or whether the failure was due to neglect. If the former we must resurface with some better type. If the latter we can repair the old surface.

Resurfacing should always be preceded by scarifying and by bringing the road up to the necessary depth of stone. For water bound macadam needing greater surface strength several classes of chemical binder may be used of which I shall discuss two—bitumen and lignin.

The bitumen may be applied either by penetration or mixing methods. The former is the cheaper, the latter the more trustworthy. Which method is to be used will, after consideration of the care the road will receive, depend on the estimate of the overload, as will also the depth of the bituminous sheet. My own observation leads me to question the wisdom of attempting to get any thickness exceeding one-half inch by penetration methods. It also leads me for this class of work to prefer tar to asphalt. The former material appears less sensitive to injury by dirt and to yield better results in repair work.

The lignin binders are derived from the waste products of wood pulp or tanin manufacture. They are cements being also used to bind core sand in foundry work and impart a tensile strength to the binding materials. They will not act on materials soaked in the so-called road oils. The action of some of these materials on slag and red shale is quite remarkable. They are slightly soluble in water and will therefore need renewal. Their application, however, is easy and inexpensive and the effect of successive applications seems cumulative, increasing not only the depth of penetration but the strength of the bond. The water proofing of lignin bound roads with bituminous tops has been carried out in Connecticut but I am unable to give any facts as to the results.

For water bound macadam roads that have failed through neglect a thin coat of gravel carrying some earthy matter or of screenings or coarse sand mixed with earth will usually cure cases that have not gone too far. In some of the counties of New Jersey it is usual to fill all ruts, etc., with fine stone and to give the middle of the road a coat of the same mixed with a little clay. Much of this material is wasted by being thrown off the road by the traffic, and old ruts almost invariably reappear. This method, however, is very successful in preventing ravelling. It seems a false economy to omit rolling and wet rolling, at that. The same is true of patching holes with anything but macadam size stone. The roads treated with small stone are yearly losing depth. The moisture for wet rolling is usually bountifully supplied by nature in the early spring. It can also be had by the use of hygroscopic salts where water is hard to get. Traffic will usually throw off from the road surface enough stone to pay for rolling.

A treatment of clay, without rolling, will often give astonishing results. Such treatment, however, seems only a palliative not a cure. Roads thus treated become dusty or muddy according to the weather, show a tendency to ravel again and are hard to properly treat with bitumens. They will, however, take the lignin binders with good results, if the dose of clay has not been excessive, or if the excess is swept off before treatment.

A Spike Tooth Harrow for Use in Gravel Road Construction.

(Staff Article.)

In gravel road construction the quality of surface obtained may be materially improved by harrowing the loose gravel thoroughly with a heavy spike tooth harrow. This is especially true where the gravel used is not uniform in quality. Thorough harrowing gives a uniform surface and is helpful in preventing pockets of one-sized material which are liable to occur. In case it is necessary to incorporate clay binder in gravel of a sandy nature this tool will be found especially useful.

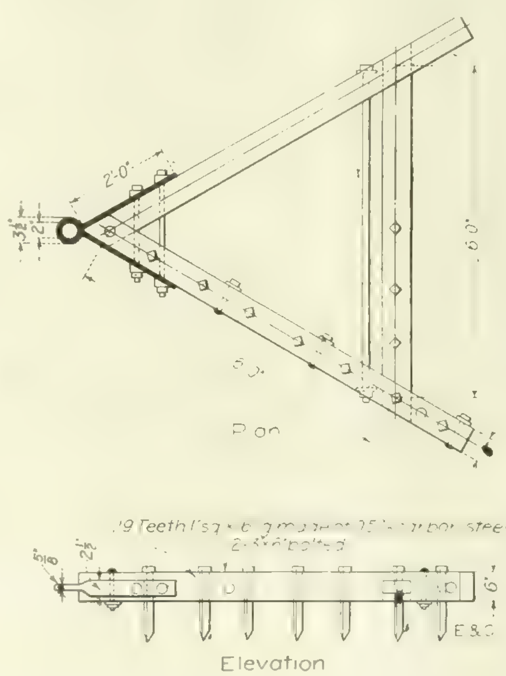


Fig. 1. Spike-tooth Harrow for Use in Gravel Road Construction.

The ordinary farm harrow is too weak in construction to use for this purpose. The tool should be so heavy that the spikes will be effective to a depth of several inches and strong enough to withstand the rough usage to which it is subjected.

The figure illustrates a type of harrow recommended by the Utah highway commission. Two, four and sometimes six horses are necessary for operation, depending on the depth of penetration desired and the degree of compactness of the gravel. Additional penetration may be secured by weighting. The weight of the harrow alone is about 350 lbs.

The method of harrowing ordinarily used is, briefly, as follows: After the gravel is dumped in the sub-grade it is spread with shovels and stone rakes having the tines about 1¼ ins. apart or with a road machine. Stones which the rake collects are pulled forward onto the earth sub-grade. After spreading the harrow is dragged over the loose gravel until a uniform mixture is obtained. As a rule, four or five trips are sufficient to accomplish this. If the road is narrow teams may be hitched so that they can walk on the earth shoulders, which is an advantage. When gravel is laid in two courses the top course should receive the more thorough harrowing. Great care should be used to rake all over-size stones out of this course. This is best accomplished while the harrow is in operation.

In incorporating clay or loam binder in a sandy gravel great care should be observed to see that the binder is in a dry and thoroughly pulverized condition. Damp clay cannot be harrowed in without leaving lumps to cause trouble in the future. Binder is best spread with shovels from a wagon or piles at the side of the road. It should never be dumped on the gravel. An even coat from ¼ to ½ in. thick should be spread uniformly and the spike-tooth harrow used to mix it with gravel below. The quantity of binder to use is dependent upon the condition of the gravel and is best determined by trial on a short section of road.

Method of Marking Through Roads in South Dakota.

In South Dakota many of the roads are marked by highway association of various kinds under the supervision of the State Highway Commission. The following abstract from the first report of the commission describes the method used:

We do not approve of boards with arrow points painted on them, or signs fastened on posts or fences. The experience with these in other states has been that vandals destroy them by shooting to pieces or reverse them and send the traveler in the wrong direction. We have but one rule, and that is to paint the emblems adopted by any particular road association on telegraph, telephone or fence posts, where such posts exist. Where nothing exists that can be plainly marked with paint, then the setting of posts with the upper part painted with the sign adopted, is the only solution. The commission has requested all highway associations in the state to mark by painting.

On a straight trail (Fig. 1a) where the driver travels beyond the crossroads to continue straight ahead, mark with two posts each 100 ft. from the corners on one side or the other and parallel with the trail to be traveled.

When the main trail (Fig. 1b) turns at a corner mark one post 100 ft. before reaching the corner, one at the corner, and another 100 ft. on the road to be followed and on the same side of the road as post on the corner. Where telephone, telegraph or fence posts exist paint the one standing nearest to point described above but not nearer than 25 ft. to the corner or more than 300 ft.

In case of a straight road with a fork leading off, Fig. 1c marked with one post not in the fork but on main line opposite the fork along the unbroken side of the road to be traveled, 100 ft. beyond the acute angle of the fork.

Posts must be set outside the fence on ground belonging to the road in such a manner that the view is not obstructed.

These road markers are to be placed every mile unless at some cross road which has been legally vacated. This gives the public confidence for they are constantly on the watch for markings. Also the traveler knows when he strikes the trail from a cross section road. We believe this system of marking will cover every road condition in our state.

Recent Specifications for Corrugated Metal Culverts in Minnesota.

A recent bulletin of the Minnesota highway commission, Geo. W. Cooley, state engineer, gives detailed specifications as to dimensions, materials and fabrication of corrugated metal culverts to be used on roads within the state. Their type of culvert is not permitted on state aided roads but may be used on other roads.

The bulletin states that ordinarily metal culverts should be laid on a compacted foundation with a fall of not less than 3 ins. in 24 ft. Backfilling must be free from organic matter and tamped to the height of the culvert.

Mill inspection is required, or a test piece with a certificate from the mill laboratory may be submitted by the contractor. Either pure iron or steel may be used, two tenths of one per cent impurities being allowed in iron and seven-tenths in steel. Metal must be manufactured by the basic open hearth process and sheets after proper annealing must have a tensile strength of not less than 40,000 lbs. per square inch and an elastic limit not less than 25,000 lbs.

Galvanizing.—All material entering into the construction of the culvert shall be galvanized in such a manner that the galvanizing forms a continuous, impervious, pure zinc coating, uniform in thickness. It shall be so applied that it will adhere firmly to the metal. Plates having blisters or other imperfections in the galvanizing after corrugating shall be rejected. The galvanizing coating shall contain not less than one ounce of zinc per square foot of surface so that two ounces of zinc are required for the two sides of 1 sq. ft. of metal. The amount of spelter shall be determined by test conducted in an approved manner by the inspection bureau.

Rivets and Riveting.—All rivets shall be of the same material as specified for the culvert. They shall be well galvanized. Rivets shall be of the following dimensions:

No. 16 gauge material, 5/16 in. diameter x ⅝ in. long.
No. 14 gauge material (two thicknesses of sheets), 5/16x⅝ in.

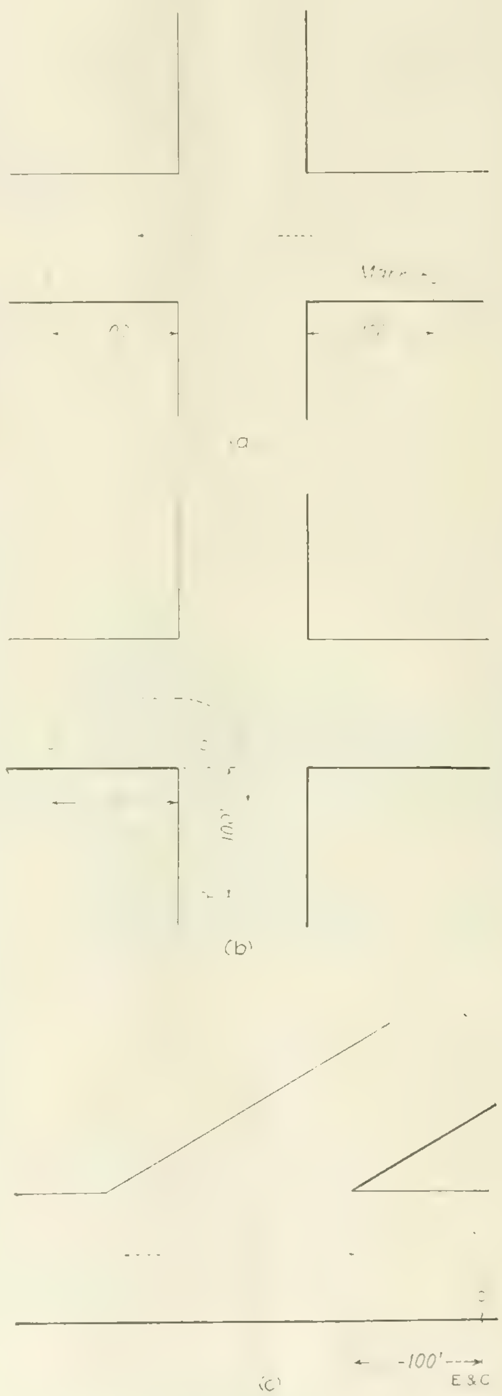


Fig. 1. Method of Marking Through Roads.
(a) Continue past cross-road, (b) turn, (c) continue past intersecting road.

No. 14 gauge material (three thicknesses of sheets), 5/16x¾ in.
No. 12 gauge material (two thicknesses of sheets), ⅜x¾ in.
No. 12 gauge material (three thicknesses of sheets), ⅜x⅞ in.
No. 10 gauge material (two thicknesses of sheets), ⅜x1 in.
No. 10 gauge material (three thicknesses of sheets), ⅜x1⅛ in.

All rivets shall be driven cold in such a manner that the plates shall be drawn tight together throughout the width of the seam. No rivet shall be closer than 1 in. from the edge of the metal. All rivets shall have neat, workmanlike and full size heads. They must be driven without bending and must fully fill the hole.

Corrugations.—All corrugations shall be not less than 2½ ins., center to center, nor more than 3 ins. The corrugations shall have a depth of not less than ½ in. for 2½ in. and not less than ⅝ in. for 3 in. corrugations.

End Reinforcement.—The ends of all corrugated metal culverts shall be reinforced with a galvanized iron band, riveted to the culvert at intervals of 10 ins. or less. This band shall be equivalent in cross section to ⅜x1 in. for No. 16 metal, ⅜x1½ in. for No. 14 and No. 12 metal, and ½x2 ins. for No. 10 metal. Any other style of reinforcement may be used that will furnish equal strength with the above, and that may be approved by the State Engineer.

Workmanship.—All material and workmanship shall be first class in every particular. Culverts shall have a truly circular cross section. They shall be truly straight throughout their entire length and free from all imperfections such as bends, dents, or crimps in the metal.

Thickness of Metal.—Culverts shall be made of sheets of the following thickness before galvanizing:

Clear inside diameter.	U. S. gauge.	Minimum weight per sq. ft. of corrugated sheet.
18-in. and smaller	No. 16	2.71 lbs.
Over 18-in. up to and including 30-in.	No. 14	3.39 lbs.
Over 30-in. up to and including 48-in.	No. 12	4.74 lbs.
Over 48-in. up to 60-in.	No. 10	5.80 lbs.

Metal culverts over 48 ins. will not be used without special strengthening, the design for which must be approved by the engineer of the State Highway Commission.

Seams.—All joints shall be lap joints. Longitudinal seams shall lap not less than 2 ins. They shall be riveted with one rivet in each outward projecting corrugation. Circumferential shop riveted seams shall lap at least one full corrugation. Rivets in circumferential seams shall be not more than 10 ins. apart.

Field Joints.—Field joints shall be made with bands of the same material as the culvert and shall be not less than 8 ins. wide for culverts up to 30 ins. in diameter, and 11 ins. wide for larger sizes—such bands shall be connected at the ends by angles or straps having a cross-section equal to 1x¼ in. and fastened by bolts not less than ½ in. in diameter. All such connections shall be made of galvanized iron.

Arrangement of a Temporary Convict Camp.

The living arrangements in a temporary convict camp are described by J. H. Pratt in a paper before the American Road Congress.

The camp in mind is located near the Broad River in Henderson County, N. C. It consists of a bunk house, or cell house as it is sometimes called, 30 ft. by 60 ft. in outside dimensions. In the center of this house is a double decked platform called the cell upon which are arranged the beds of the convicts. There is a clear space of 12 ft. between each end of the building and double platform, and 6 or 8 ft. clear between the cell and the side walls. The space between the two platforms is approximately 5 ft. Each man is allowed a single mattress, so that he has plenty of room for sleeping purposes. Four chains run the length of the platform cell—one each side for the lower tier, and one each side for the upper tier. To these chains the convict is fastened at night by a light weight ankle chain. This is so arranged that there is little or no weight on the ankle and he can turn in any position he wishes while sleeping. The construction of such a bunk house depends on the time of the year and length of time it is to be occupied; but is is always built so that there is plenty of air circulation through the building and that it may be kept warm and comfortable in cold weather. Guards are on duty in this building at night, one at each end.

Near to this building is the dining hall, kitchen and store house. Surrounding these two buildings and enclosing an erea of about one-fifth of an acre is a six-strand barbed wire fence. Just outside of this fence at opposite corners armed guards are stationed during the day. At night the only guards are within the bunk house. The sleeping houses for the superintendent, steward and guards are a little distant from the enclosed area. The food supplied to the prisoners is the same quality as that supplied the guards and the steward. It is necessary that pure, wholesome food, clean and well-cooked should be furnished to prisoners, and that is what this camp tries to do.

In a camp of this sort the men have free run of the building and of the area within the fence during the daytime. At the present time one week out of every four is remitted from the prison sentence for good work. In Virginia and in certain parts of North Carolina men who formerly worked on the roads are now foremen in charge of free labor in building roads. Likewise, men are now employed by farmers who watched them at work on public roads, kept track of them when their sentence was over and offered them work on the farm because they knew they were good laborers. The idea is to build up men capable of becoming citizens of the community after their sentence has expired, which is one function of the state with regard to the treatment of its convicts.

SEWERAGE

The Main Drainage and Sewage Disposal Works Proposed for New York City.

The final report of the Metropolitan Sewerage Commission of the City of New York, dated April 30, 1914, has been printed and some copies have been distributed. The report describes the plans for main drainage and sewage disposal works which the Commission recommends the city to construct for the protection of the harbor. The present article, based on the report, describes the general nature of the proposed works and has special reference to the outlet and disposal island which is by far the most interesting feature of the project.

The plans outlined in the report are sufficiently definite to show the nature, extent and approximate cost of the recommended works. Detailed studies of design will be worked out by those entrusted with the construction. The Metropolitan Commission did not seek authority to carry its recommendations into effect.

THE SYSTEM RECOMMENDED

The system recommended consists largely of intercepting sewers, running approximately parallel to the water front, to collect the sewage from the local sewerage systems, to a number of centrally situated disposal plants where sufficient of the impurities can be removed to permit the effluent to be discharged into the waters without danger or offense. To facilitate the diffusion and assimilation of the sewage materials by the water, it is considered desirable to place the outlets at the bottom of the deep and swiftly-flowing channels.

The system of main drainage which the Commission recommends is presented for adoption by the city both as a plan and policy for future construction and should be carried out in successive stages and not as one undertaking. The immediate construction of the whole scheme is not necessary from a sanitary standpoint. Such parts of the system as are needed for the immediate future should be taken in hand at once and the remainder built as required. The plans are sufficiently flexible to permit of the adoption of any discoveries or improvements in the art of sewage disposal which may be made in the future.

When complete, the works will constitute a systematic and well co-ordinated scheme of main drainage for the whole city which will utilize the absorptive capacity of the harbor waters to the greatest extent consistent with public health requirements and with a view to the prevention of nuisance.

THE FOUR MAIN DRAINAGE DIVISIONS

The four main drainage districts, illustrated in Fig. 1, are as follows:

The territory on Manhattan Island and in Brooklyn which naturally drains into the Lower East and Lower Hudson Rivers and Upper New York Bay constitutes the Lower East River, Hudson and Bay Division.

The areas in the Boroughs of Queens and the Bronx which naturally drain into the Upper East River, and those parts of the Boroughs of Manhattan and the Bronx which naturally drain into the Harlem River constitute the Upper East River and Harlem Division.

The territory whose drainage flows or can readily be made to flow into Jamaica Bay is called the Jamaica Bay Division.

The territory in the Borough of Richmond, or, as it is more generally termed, Staten Island, constitutes the Richmond Division.

SELECTION OF POINTS FOR DISPOSAL

Having tentatively settled upon the main divisions, the selection of the central points to which the sewage should be collected for treatment and disposal became a matter of principal importance. Upon the choice of these points depends not only the cost of collecting the sewage, but the method of treating it and the facility with which the effluent can be disposed of after treatment.

It was considered by the Commission that as far as practicable, the collection point should be near the ocean or Long Island Sound or close to deep tidal channels. Points of outlet for untreated sewage, if any sewage was to be discharged in crude condition, should never be situated in shallow, stagnant, land-locked or remote parts of the harbor. Favorable conditions for a prompt dispersion and digestion of the sewage matters should be sought. Where facilities were lacking for the disposal of the sewage through dilution by large volumes of freely flowing tidal water, compensation for this lack should be made in the degree of treatment given to the sewage

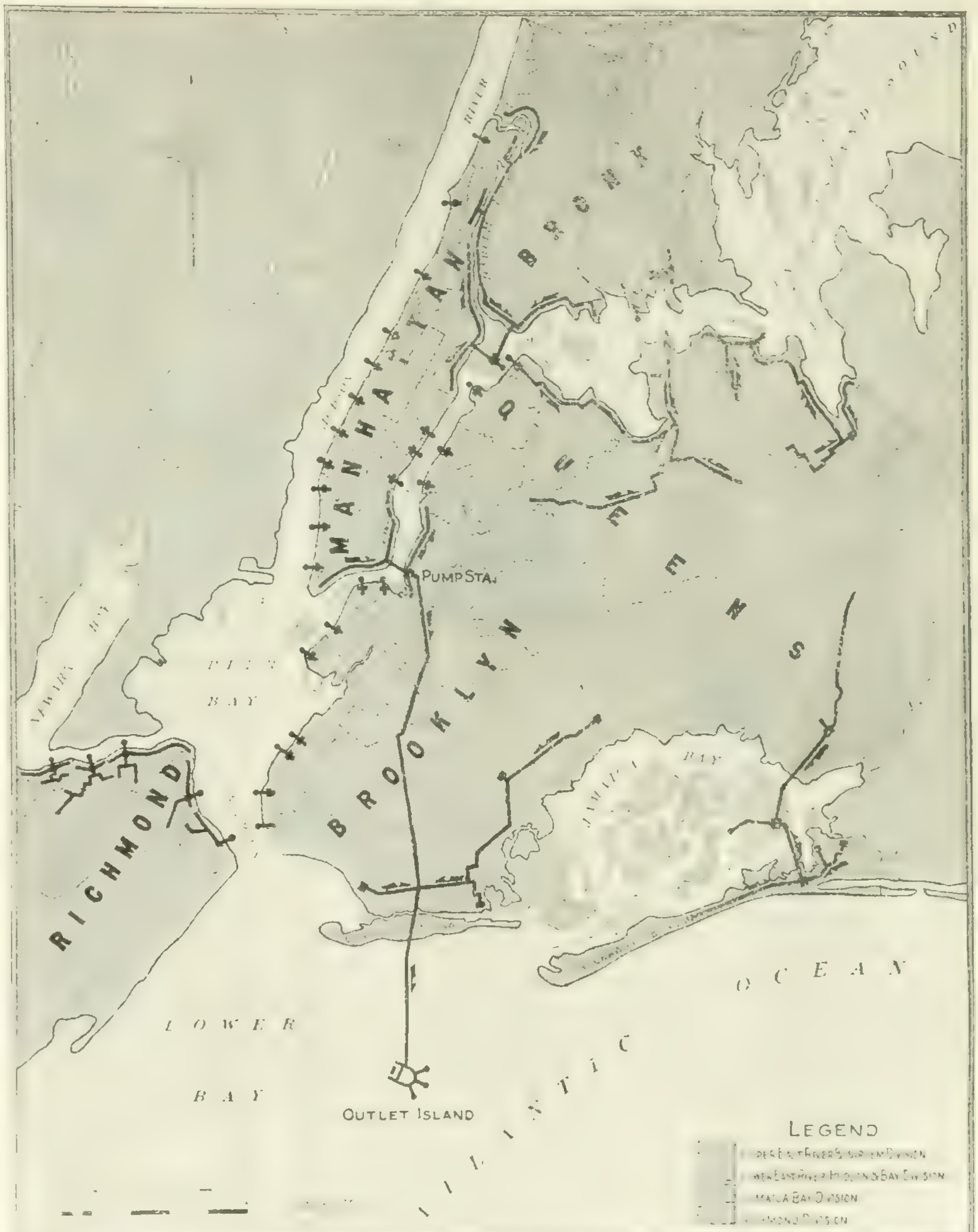


Fig. 1. General Plan Showing Main Drainage and Sewage Disposal Works Proposed for New York City by the Metropolitan Sewerage Commission, 1914.

for the removal of its organic ingredients before the discharge.

It was considered desirable to make the number of central points as small as practicable in order to simplify the problem of administration and to facilitate the ultimate disposal of the sewage, due attention being given to the probability that pumping would have to be employed to some extent and to the fact that for purposes of economy of operation the works should be as compact and concentrated as is consistent with due regard to the first cost.

The exact degree of purification required for the sewage could not be stated when this method of designing the works was decided upon, but it was the opinion of the Commission that elaborate processes and those which required much land, extensive apparatus, patents and untried or experimental features should be avoided as far as practicable. As between large first cost and low running expenses, or small first cost and high maintenance charges, the Commission favored the former as likely to lead to more satisfactory results, since the expenditure represented by the large investment would be made up chiefly of interest charges which could be reckoned with in a definite manner and so free from the uncertainties of such elements of expense as are involved where large quantities of supplies and labor are concerned. So far as methods of purifying the sewage are concerned, the object was to make good use of the absorptive capacity of the harbor waters and purify the sewage no more completely than was necessary in order to satisfy a reasonable standard of cleanness. By good use is here meant such use as would do no material harm to the public health and welfare either through the production of disease or nuisance. To a large extent, screens, grit chambers and similar methods of so-called preliminary treatment should be employed. Only in exceptional cases would the utmost degree of purification be required.

THE UPPER EAST RIVER AND HARLEM DIVISION.

The location of this main drainage division is shown in Fig. 1, which also shows the location of interceptors and outfalls. The division will have five outlets. After a careful study of the question of the form of treatment required for the sewage of this division, due regard being had to the needs of each of the five outlets, the conclusion has been reached that fine screening or coarse screening and sedimentation will, for some years, give an effluent of satisfactory character for discharge into the water of the East River.

Where sedimentation tanks are to be used, coarse screens will be employed to protect the pumping machinery and to keep large floating matters from causing trouble in the tanks and from passing out through the outfall.

In all cases grit chambers will be placed on the lines of the main trunk sewers at or near the treatment works. In these chambers the sewage will be given a settling period of from one to two minutes, the velocity being reduced sufficiently to allow the heavy mineral detritus borne by the sewage to be deposited, but not enough to permit much organic matter to settle.

The grit chambers will afford protection to the pumps and will keep the proposed long and deep outfall pipes clear of gritty deposits wherever sedimentation tanks are not used. Where sedimentation tanks are planned, the grit chambers will first rid the sewage of suspended matter of a kind which would cause trouble and be difficult to handle if allowed to settle in the tanks. Grit chambers are especially useful where combined sewers are intercepted or form a part of the collecting system, as will largely be the case with the main drainage systems as here proposed for New York City.

The sewage will be discharged in every case at a distance from the shore, the position of the outfall depending upon the nearest point at which water of suitable depth can be found. It is proposd always to have the sewage discharged at depths of from 30 to 50 ft., and in such manner as to give a favorable opportunity for its admixture with the water of the river. In order to facilitate diffusion, it will be desirable to discharge the sewage from each point at more than one outlet.

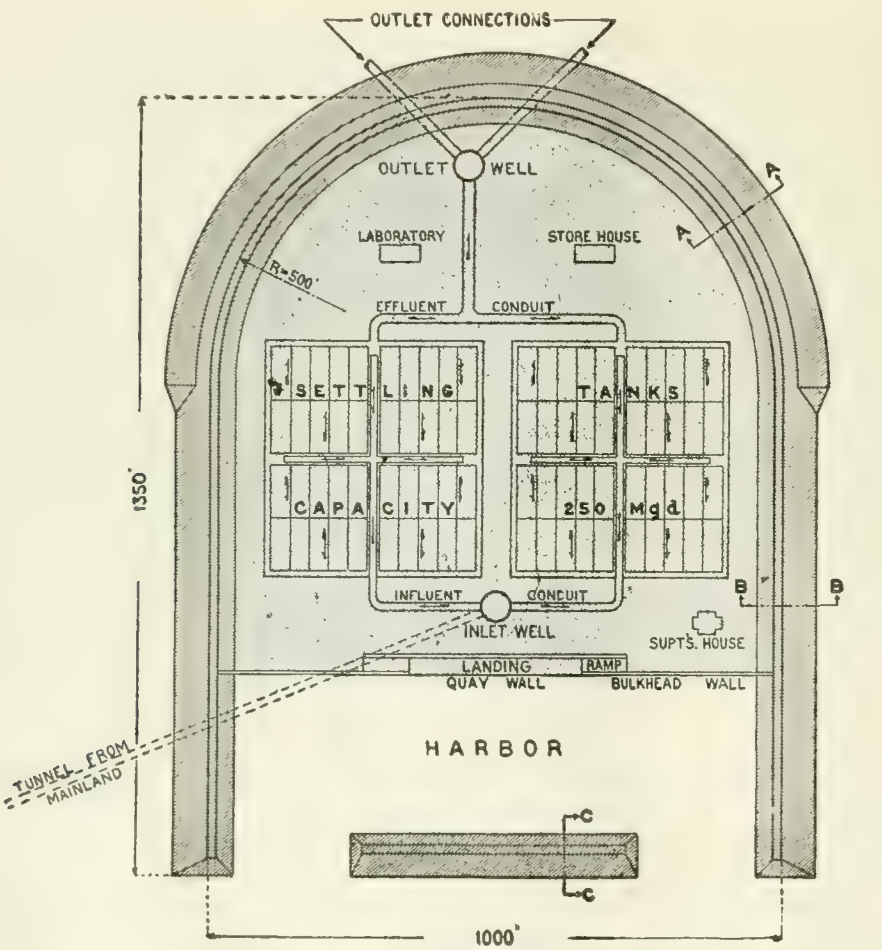


Fig. 2. Plan of Proposed Outlet Island for Sewage Disposal Works to Treat a Flow of 200,000,000 Gals. Per Day from Lower East River, Hudson and Bay Division.

THE RICHMOND DIVISION.

The location of the Richmond main drainage division and of the interceptors for the division are shown in Fig. 1. The division is further subdivided into five parts. In one of the subdivisions the intercepted sewage will be passed through coarse screens, grit chambers and fine screens. In the other four subdivisions the intercepted sewage will be passed through coarse screens and settling tanks and then discharged into water 30 ft. deep. A period of about two hours will be allowed for settlement. The settling period in the grit chambers will be from one to two minutes. The treatment works in the district are so placed that the sludge can be transported by water or rail. It would be possible, therefore, to dispose of it either at sea or on land. The presence of two garbage incinerators, both almost directly on railroad lines, would make it feasible to burn the sludge and garbage together. Centrifugal dryers might be placed at each disposal plant to dry the sludge before transportation, or it might be more economical to install such dryers at one or both incinerators and transport the larger volume of wet sludge to the incinerators in proper cars.

THE JAMAICA BAY DIVISION.

This division, shown in Fig. 1, is further divided into two parts—Eastern Jamaica and the Western Jamaica subdivisions. Each will have a distinct system of collection and disposal. The sewage from the latter subdivision will probably be conveyed to the outlet island in connection with that from portions of the Lower East River, Hudson and Bay Division as described more fully later. The sewage from the Eastern Jamaica subdivision will be pumped through submerged mains to a treatment plant on Jo Cos marsh. This plant will consist of settling tanks, sprinkling filters and settling basins.

THE LOWER EAST RIVER, HUDSON AND BAY DIVISION.

Briefly, the works which the Commission's studies indicate should be constructed to protect the waters of the Lower East River, Hudson and Bay Division consist of intercepting sewers to collect the sewage, screening plants to remove the coarser solids, and submerged outlets to discharge the effluent into the main tidal channels at a distance from shore. Should this form of treatment prove insufficient for the Lower East River Section, it will be desirable to remove a large part of the sewage tributary to the Lower East River by means of a tunnel discharging at an artificial island at sea about three miles off the Coney Island shore. If the sewage is to be carried to sea, Dortmund tanks should be constructed upon the island and the sewage subjected to sedimentation for a period of about two hours. After this short and inoffensive treatment the sewage can be discharged into the ocean with confidence that the putrescible matters will be promptly rendered inert.

This division contains the largest population, and the most densely settled sections of any of the four divisions into which the Commission has divided New York for the purpose of planning the main drainage and sewage disposal works which will be required. Within it lies the major part of the Boroughs of Manhattan and Brooklyn.

The Plan Recommended.—The plan recommended by the Commission contemplates the collection of the Manhattan sewage tributary to the East River below 26th St. at Corlears Hook, and that part of the sewage of Brooklyn which is tributary to the East River from Classon Ave. to Newtown Creek at South 5th St. At Corlears Hook and South 5th St. the sewage will pass through grit chambers and fine screens and will be discharged into the East River through multiple submerged outlets. This is the first stage in the larger project of taking the sewage of the Lower East River section directly to the sea, which is the plan the Commission recommends for ultimate construction.

Advantages of Disposal at Sea.—In accordance with the ultimate plan proposed for the Lower East River section, the sewage will be tributary to a general central station, to which point will be gathered such part of the sewage as needs to be carried to a distance. Pumps will be located at the central station

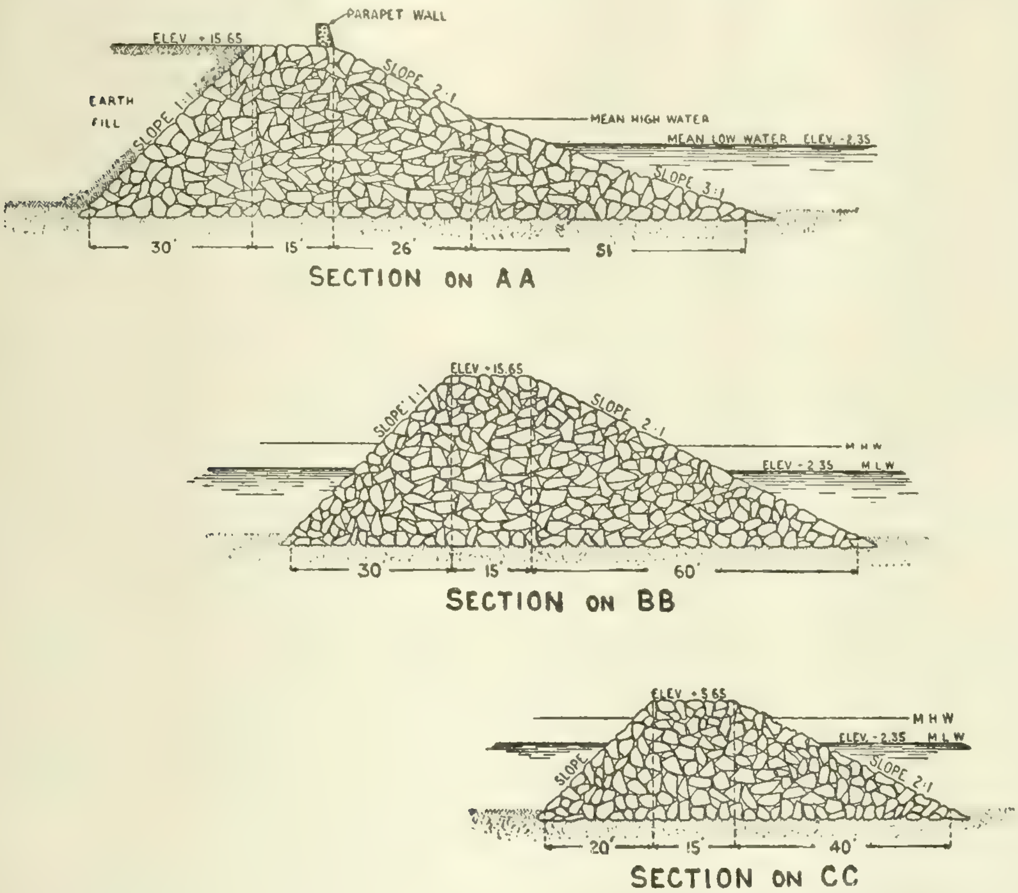


Fig. 3. Sections of Retaining Walls for the New York Artificial Sewage Disposal Island. Sections Indicated on Fig. 2.

and from it a main will run directly to an outfall island to be built about three miles from land on a sandy reef. This reef is one of a series of shallow areas, interspersed with channels, which once formed the bar to New York harbor. The outfall will be about 13 miles from the New York City Hall, 6 miles from the Narrows, over 4 miles from Sandy Hook and about 3 miles from Coney Island. The point selected for this island is shown in Fig. 1.

As much sewage as it is necessary to carry to a distance from the Lower East River, Hudson and Bay Division can be taken to the island for disposal. As time proceeds and the quantity of sewage increases, the main sewer can be duplicated and the provisions for treating and discharging the sewage at the island can be enlarged.

A large part of the sewage of this division is now discharged into the waters of the Lower East River. If this sewage is taken away for disposal, it will for some time, at least, be possible to discharge the sewage from the rest of this division with no other treatment than screening and passage through grit chambers. The water of the Hudson will be capable of assimilating the sewage produced on the west side of Manhattan Island and the water of the Upper Bay could take the sewage produced along the Brooklyn water front from Governor's Island southward.

All the sewage from this division, except that part which is taken away, should be passed through grit chambers and screens and the dry-weather flow discharged through submerged outlets. In course of time, if the quantity of sewage from this division, as well as from that part of the Upper River and Harlem Division which would be concentrated at Wards Island, increases to such an extent as again to place an excessive burden upon the waters of the East River, the sewers can be extended and finally the Wards Island works can be connected with a main sewer to the artificial island for disposal.

The plan of relieving the harbor of its heaviest burden by taking to sea a large part of the sewage which flows to the Lower East River and increasing the scope and magnitude of the work as necessity arises, appears to this Commission to be a necessary and sufficient solution of the problem. In no other way can the sewage be disposed of with so little chance of danger or offense. The project has the advantages that it will afford, at minimum expense, all the relief that is needed for the near future and is capable of expansion.

There are no shellfish industries in the vicinity of the proposed island and no currents which would carry any of the sewage to a bathing beach. The sewage will not be exposed long enough to the air to cause annoying odors to be given off and there will be no opportunity for flies to breed.

The plan is in accordance with the best engineering precedent. There is no feature connected with it which is untried or experimental. It avoids offensive, complicated and uncertain processes of purification. It is based upon a careful consideration of the needs of the whole harbor. It leaves the waters of the inner harbor in a sufficiently improved condition for the assimilation of such sewage as cannot be kept out of the waters without wellnigh prohibitive expense.

At first the form of treatment needed at the island will be settlement in tanks, perhaps aided, at times, by precipitants. In addition, it may be practicable to disinfect the sewage and produce a considerable amount of oxidation by addition of bleach or electrolytically produced hypochlorite. If at any time in the future it becomes desirable to completely purify the sewage, no such favorable location for the necessary works can be found in the metropolitan district than this artificial island. Owing to the shallowness of the water and the ease with which filling can be obtained, land can be made here for less money than an equal area can be bought on shore at any point not more distant from the New York City Hall.

The Inverted Siphon from Manhattan to Brooklyn.—In the completed installation an inverted siphon will be required to carry the sewage from Manhattan to Brooklyn beneath the Lower East River. The point selected for the crossing is at a narrow part of the river where solid rock may be anticipated. The siphon which will be required to take the sewage produced in 1915 will have a diameter of 7 ft. 8 ins. The depth will be 110 ft. beneath the surface of mean low water. The siphon will be 2,400 ft. long and extend from Corlears Hook to South 5th St. The velocities in this siphon will range between 2 and 5 ft. per second.

The General Pumping Station.—The sewage collected at the general pumping station, amounting to about 200,000,000 gals. a day, will have been passed through grit chambers and screens and will be in reasonably fresh condition. The pumps will be required to raise the sewage at times of mean flow from an elevation of about 5 ft. below mean tide and pump it under a head of about 29 ft. to the artificial island at sea. The distance to be pumped will be about 12.9 miles and the head to be overcome will be that which is necessary in order to raise the sewage from the level at which it is delivered to the pumps to the level of the tanks where it is to be treated on the island, plus the head required to overcome the frictional resistance offered to the passage of the sewage through the long main. The pumps can be operated by steam, oil or by purchased electric current. It would seem feasible and desirable to drive the pumps with electric power to be obtained from burning the solid refuse of the city in destructors, as is commonly done in England and in certain large cities on the continent.

The Sewage Main to Sea.—The force main through which the sewage will be pumped to the island will be built, for the most part, in tunnel. There will be three shafts so situated as to permit the work of construction being pushed with expedition and economy.

The Artificial Island.—The tunnel to the island, if sewage from the Western Jamaica subdivision is admitted, will be 14 ft. in diameter and will be constructed at a depth of about 60 ft., the material to be penetrated being sand. It will be possible to construct the tunnel with two headings, one from the shore and one from the island, the two meeting and being properly joined.

The point selected for the island has been

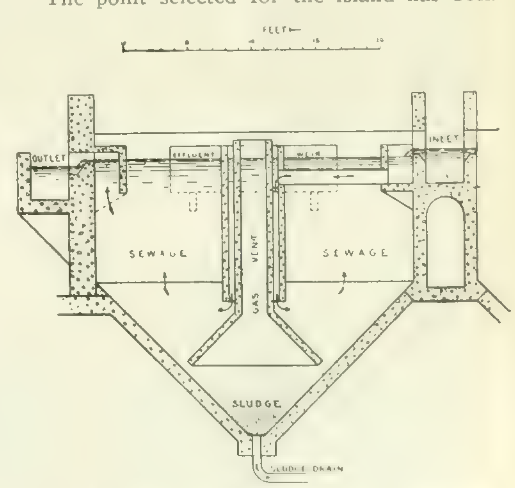


Fig. 4. Sectional Elevation of Proposed Dortmund Settling Tank of Type Recommended for Use on New York Artificial Sewage Disposal Island.

(Capacity 500,000 gals. per day with three hours retention.)

carefully chosen with reference to economy of construction, resistance to the destructive influences of tidal currents and storms, freedom of obstruction to the free flow of tidal water in and out of the harbor and absence of sanitary objections. The location lies to the north of Sandy Hook and to the south of Coney Island. The water within a mile from the island in all directions varies between 7 and 40 ft. in depth, the average being about 20 ft. at mean low tide.

The plan of the island is approximately rectangular, the seaward side being rounded as shown in Fig. 2. The area at the start will be about 20 acres. This can be extended as required.

The outer face of the island will be a wall of riprap composed of large pieces of broken stone carried to the site on boats and laid upon the hard sandy bottom. (See Fig. 3.) It is expected that some settlement will occur, due to the water cutting sand away from under the stone. When sufficient riprap has been bedded to stop this action of the water, no more settlement is to be expected. The main bulk of the island will be composed of sand supplied from a suction dredge, which will take its supply from the bottom of the sea in the vicinity.

The height of the island above mean low water will be about 18 ft. The length will be 1,300 ft. and the width 1,000 ft. The side of the riprap wall which is exposed to the sea will have a slope of 1 vertical upon 3 horizontal below mean high water, and 1 upon 2 above, while the other sides will have a slope of about 1 on 2. The cost of constructing the island has been estimated at about $615,000.

The landward side of the island will be provided with a quay wall for the accommodation of vessels engaged in taking supplies and other materials to and from the island. Shelter from the sea will be provided by a breakwater, which will enclose a small harbor.

The island will contain a plant of settling tanks in which the sewage will have an opportunity to deposit its solid matters during a period of about two hours. These tanks will be of modified Dortmund tank construction, as shown in Fig. 4, similar to those recently constructed at Toronto, Canada. This cut is from a paper by George A. Soper, published in the Journal of the Boston Society of Civil Engineers for February, 1914. Provision will be made for treating the sewage if necessary with a coagulant before passing it into the tanks.

After treatment, the sewage will be discharged through a number of outlets arranged radially on the seaward side of the island. If desirable, it will be feasible to pump sea water into the sewage and provide for the mixture of the two before the discharge takes place. Such admixture would facilitate the immediate diffusion of the sewage in the sea water, but the active agitation and free movement of the great volume of water in the vicinity of the island will probably make the preliminary admixture of sea water and sewage by pumping unnecessary. The material which settles out in the tanks will be carried to sea in boats and dumped.

Recommendation of the Commission as to the First Installation.—In the opinion of the Commission, it would be desirable but not necessary to carry out the island project as a first installation. The fine screening proposed, although the most thorough treatment practicable in this section, would afford relief to the Lower East River only for a time. The improvement effected by discharging through submerged outfalls instead of at the pierhead line would alleviate conditions in the slips and along the shore, but would not make the water permanently satisfactory nor alter the dilution ratios of sewage to water in the river as a whole. Any benefit accruing from this procedure would be neutralized later by the increase in the volume of sewage to be disposed of.

In comparing the projects for local sedimentation treatment as considered at one time by the Commission, with that in which the entire 200,000,000 gals. per day is taken to the ocean, it must be remembered that by the former project the tank effluent would carry to the East River 70 per cent of the organic matter contained in the sewage. Now, if it is assumed, as seems fair, that but 25 per cent of the total organic matter capable of reducing the dissolved oxygen of the harbor is removed by this process, it follows that, to produce an improvement of the water equivalent to the removal of 200,000,000 gals. per day of sewage to the ocean, it would be necessary to treat 800,000,000 gals. per day by screening. This is more sewage than will be produced in the whole city in 1915.

In other words, if all the sewage of Greater New York were treated by fine screening, it would not improve the general condition of waters of the harbor as much as the entire removal of the 200,000,000 gals. per day.

METROPOLITAN COMMISSION RECOMMENDS A CONSTRUCTING COMMISSION.

As stated previously the Metropolitan Commission did not seek authority to carry out its plans. It is the opinion of the Commission that the construction and maintenance of the works should be placed in the hands of a special commission existing for that purpose. It is possible that where parts of the system would be situated wholly within a borough, it might be desirable to turn those parts over to that borough to operate under the regulation and control of the central commission.

If a commission were created to construct such sewerage works as New York requires, it should take over the effects of the Metropolitan Sewerage Commission, make the final detailed plans and estimates required and, after duly submitting its projects to the Board of Estimate for approval, proceed with the construction.

PERSONNEL.

The members of the Metropolitan Sewerage Commission of New York were George A. Soper, president; James H. Fuertes, secretary; H. de B. Parsons, Charles Sooysmith and Linsly R. Williams. Chief among the several assistants employed by the Commission were engineers, Kenneth Allen, D. S. Merritt, Wm. B. Fuller, John H. Gregory and David Lowensohn; chemists, Wm. R. Copeland and Payn B. Parsons; and biologists, Raymond H. Pond and R. N. Hoyt. Among the experts who made special investigations and reports for the Commission were: Dr. Gilbert J. Fowler, John D. Watson, Rudolph Hering, George W. Fuller, George E. Datesman, Samuel Rideal, Karl Imhoff, and X. H. Goodnough.

Notes on the Design and Construction of Intercepting Sewers with Special Reference to Minimizing Infiltration.

Although many sewerage engineers consider extremely flat grades practicable for intercepting sewers, because of the depth of flow and the macerated condition of sewage on reaching intercepters, an occasional warning against too flat grades is sounded. Thus in a paper before the American Association for the Advancement of Science Mr. J. N. Ambler, of Winston-Salem, N. C., urged the advisability of utilizing all the available fall. The following excerpts from the paper, relating chiefly to the minimizing of infiltration, are here republished:

All the sand, mud and illegitimate matter of all kinds which enter the collecting sewers is pushed down their sharper grades and is finally deposited in the intercepter.

The writer once removed some twelve or fifteen hundred bushels of such substances from about half a mile of old intercepter. Most of this was sand, but bottles, rags, straw hats, overalls, stones, wooden articles, etc., were present, and the capacity of the sewer was greatly reduced.

Of course, hard and fast rules cannot be laid down, but the writer does not approve laying a grade as flat as 1 ft. per 1,000. It is his endeavor, in the light of experience with cleaning, to get all the fall possible.

From the very nature of the low level intercepter, it is usually located where the ground is alluvial, and with the ground water level near the surface. In many cases the ground to be traversed is marshy, and in nearly every instance the sewer is laid below the ground water level. These conditions are distinctive for intercepters and require special methods of treatmnt.

Unless the joints are laid very perfectly and are so maintained at the flat grade as to utilize all the fall, without pockets, the infiltration of ground water will be so great as to greatly reduce the carrying capacity of the sewer, or even, in the case of a long line, to completely destroy its usefulness.

Settling of the joints will, of course, render the conditions worse, and may result in the inflow of quicksand until the sewer is put out of commission, as in a case which once came under the writer's charge for correction. It is, therefore, of the utmost importance not only to have perfect joints, reducing the inflow of the ground water to a minimum, but also to provide means to maintain the grade as a perfectly straight and rigid barrel, free from settlement.

The difficulties of meeting this last requirement are often very great, as usually the bottom of the trench is a water-soaked quagmire and frequently quicksand.

Considering, first, the question of perfection in the joints, the writer has specified a mortar of neat portland cement, on account of its density and imperviousness. The shrinkage of neat cement mortar being considerable after the mortar is set, a wash of thick grout is used to fill the cracks. The entire circumference of each joint is rigidly inspected by hand, and no imperfect joint is left on the work.

Special compounds, as melted sulphur, and the various elastic compounds from which it is claimed that perfect joints may be made, even in water, were not tried, either on account of the expense, or because there was doubt felt as to the durability of such substances.

Feeling the necessity of having some criterion upon which to base the engineer's acceptance of the work, so far as ground water is concerned, the writer read up all available data on ground water flows, and came to the conclusion that it is a subjcet about which too little is known, and that many of the existing data are conflicting.

There should, however, be some criterion to apply to work done by contract, and it seemed that nothing better could be done than to determine how much water the sewer might be allowed to carry without materially injuring its usefulness. The criterion should not be so severe as to greatly affect contractors' bids. Such a criterion, it would seem, should be based upon the number and circumference of the joints.

The following clause appears in one of the writer's specifications, and is offered for what it is worth, until something better can be substituted. It at least has had the effect of securing some very careful work, and contractors were not disposed to regard it as severe when it was thoroughly explained to them.

The clause is as follows:

It is the intent of these specifications that no more leakage of ground water into the sewer be allowed than is admissible with a first-class piece of work, in which care has been exercised to get as near as possible to a water-tight result.

To determine the admissible amount of leakage, the length of a joint will be considered as the outside circumference of the spigot end of a pipe.

Leakage not in excess of 2 gals. per day of 24 hours for each foot of circumference of every joint will be considered admissible, the amount of flow to be determined by the engineer's gaging in each section, by means of a notch board.

The contractor agrees that for each 10,000 gals. per day of 24 hours by which the total flow of sewer exceeds what the total flow should be, when figured on the basis already given, a deduction of $100 from the contract price will be made.

This will not apply further than to a total flow resulting from 3 gals. per day of 24 hours, from each foot of joint length, beyond which figure the sewer will be regarded as not in compliance with this contract.

The writer's opinion is that the above requirement is a very mild one, as he once laid a mile of large sewer through exceedingly swampy land, passing under several streams, etc., with a result that not more than a stream ¼ in. deep was flowing in the bottom of the pipe at the lower end on completion. However mild the criterion may be, it has had a powerful effect against bad work.

It is extremely important that the manholes should not leak. To prevent this, the best hard bricks are used, each being dipped in a pail of water, as handed to the mason. The brick work rests on a cement bottom 12 ins. thick with a solid cement invert and the contact of the pipe with the masonry is an object of especial care. Manholes are built up sufficiently high above the meadow to prevent being stopped by floods. The caps are secured to the covers by two set screws to prevent malicious persons from placing foreign substances in the sewer.

The handling of water in the trench during construction is usually less serious than the inexperienced are apt to imagine. A diaphragm pump connected to a small gasoline engine is the cheapest and best way known to the writer for removing the water. A small trench is excavated along the side wall of the main trench which conducts all accumulated water to a pump, where it is promptly removed by pumps, without being allowed to rise on the freshly constructed sewer line.

The writer has used the pile and platform method of founding, and finds it of very general application, of moderate cost to handle, and very satisfactory in result. Two piles, 4 by 4 ins., bluntly sharpened, are driven opposite each other in the trench. Two more piles are driven at a distance of 2½ ft. up the trench, and so on, wherever a thoroughly firm bearing for the sewer cannot be found. Across the heads of these piles a 1 by 4-in. batten is spiked, and a platform of two 1½-in. planks is laid on and spiked to the various battens over each pair of piles.

Piles are driven to practical refusal by means of a rammer, consisting of a cast iron disc 8 ins. in diameter, 3 ins. thick, into the center of which a 2-in. iron rod is screwed. This makes a very heavy rammer, resembling the piston rod and head of a steam engine. It takes three or four strong men to operate it, but it is the quickest and most economical way the writer has been able to find for this work.

The piles vary in length from 18 ins. to 9 ft. Even where the short ones are used, the ground being tolerably firm, a far superior foundation results than is to be had by laying loose planks in the bottom of the trench.

The platform was laid to a more or less regular grade about 0.3 ft. below the flow grade of the sewer, the pipe being supported in a cradle of two wedges cut from 1½-in. plank, and driven under the pipe from opposite sides until the flow line was precisely at grade.

The wedges were then spiked to the platform to hold them in position, and about ½ gal. of concrete was placed behind them to prevent a tendency for the back-fill to crush the pipe.

CONSTRUCTION PLANT

MACHINES DEVICES MATERIALS

Large Revolving Steam Shovel for Canal Construction.

Contributed By Jean M. Allen, Consulting Engineer, Chicago, Ill.

The large size revolving steam shovel, while a comparatively new development in steam shovel construction, is quite well known for its performance in coal stripping work. It is only recently, however, that contractors have begun to realize its possibilities as an excavating tool. It is interesting therefore to note that Byrne Brothers Dredging & Engineering Co. of Chicago are using a machine of this type in connection with an inclined tipple in excavating Section [illegible] of the Calumet Sag Canal. An interesting feature of this plant is its ability to excavate the entire prism of the canal at one cut (except the solid rock at the bottom) and place the material in the spoil bank at one operation.

The Calumet Sag Canal at this point has a bottom width of 36 ft., a depth of cut of 36 ft. and slopes of 2 horizontal to 1 vertical, thus giving a top width of 180 ft. There are about 6 or 8 ft. of solid rock in the bottom of the cut, the remainder being a glacial drift consisting of sand, gravel, boulders and clay. The length of the contract is 5,500 ft. and there are approximately 875,000 cu. yds. of glacial drift, and 50,000 cu. yds. of solid rock to be excavated. This includes the protecting drainage ditches.

Figure No. 1 shows a typical cross section of the canal prism and the general dimensions of the shovel and tipple. The steam shovel is a Marion model 251 with a 3½ cu. yd. dipper and a 75-ft. boom. The extreme height of dump is 53 ft. above rails; extreme radius of dump 83 ft.; and the radius of cut at 34 ft. above rails is 88 ft. The working weight of the shovel is 355,000 lbs. The tipple consists of a cantilever incline of structural steel, which is carried by two parallel standard gage tracks on the canal berm with a hinged apron extending down the slope into the prism. On this incline are two standard gage tracks, which carry the two dump cars of 10 cu. yds. nominal capacity. These cars are operated independently of each other by a double cylinder double drum

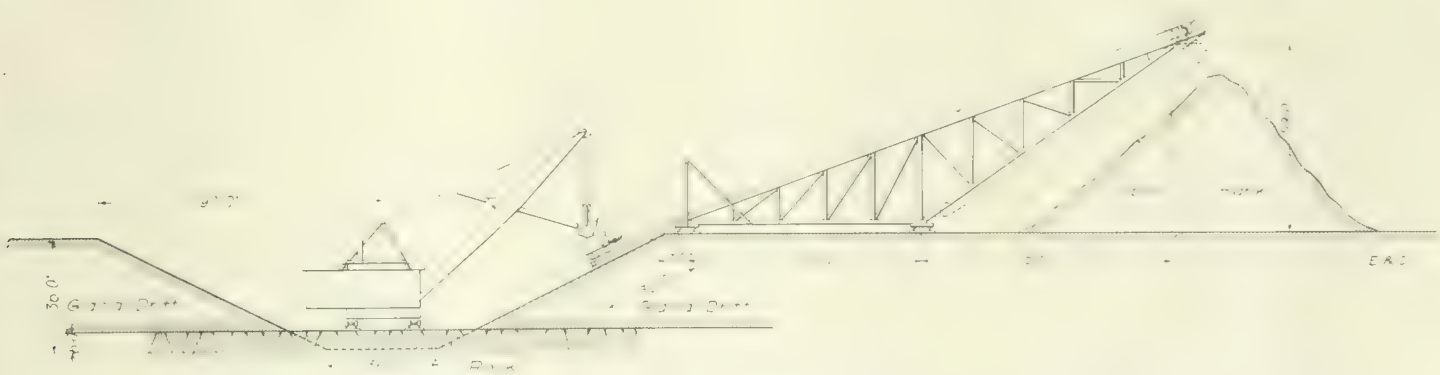
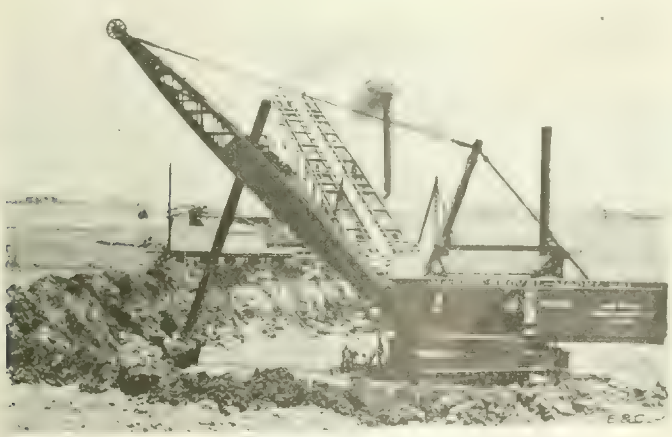


Diagram Showing Arrangement of Revolving Steam Shovel and Steel Tipple for Canal Excavation.



View of 3½ Cu. Yd. Revolving Steam Shovel. Excavating Canal.

View of Revolving Steam Shovel and Steel Tipple Excavating Canal.

engine, with cylinders $10\frac{1}{2}$ ins. diameter by 12 ins. stroke. This engine is of heavy special design. The gears are of cut steel 6 ft. in diameter, and the drums are 4 ft. in diameter. The frictions are of the outside band type actuated by steam rams. A 100 h.p. locomotive boiler furnishes steam for the engine at 125 lbs. pressure.

The method of operation is as follows: The car is lowered into the pit on the inclined apron and filled with two loads of the $3\frac{1}{2}$ cu. yd. dipper, then hauled to the top of the incline, where it runs onto a steel tipple frame, which is hinged to the top of the incline by a heavy shaft. The car is securely held on this frame by dogs which engage automatically. As the car reaches its position on the tipple frame it releases a latch which permits the frame with the car to tip outward, thus dumping the load. A pendulum counterweight attached to the tail of the tipple frame by a wire cable prevents it from tipping too far and also returns it to its normal position after the load is dumped. The car is then lowered to the bottom of the incline by the foot brake. While one car is being dumped the second car is being loaded by the shovel, thus there are no delays waiting for cars.

The complete weight of the tipple with cars, engine, boiler, fuel and counterweights is approximately 300,000 lbs., but as nearly all of this weight is carried by the rear truck, which is over 80 ft. from the edge of the slope, no trouble has been experienced by caving of banks due to the weight of the machine.

In the accompanying views, Figs. 2 and 3, the shovel has not yet worked down to grade or rather to the rock, for it is proposed to excavate to the rock at one cut and then blast the rock and spoil it on the slopes as rip rap.

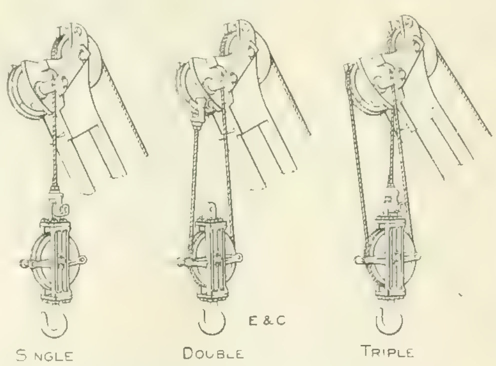
The plant was installed in March and April of this year and started work about May 1, and has been working in a very satisfactory manner, the average output being from 1,800 to 2,000 cu. yds. per 10 hour shift.

The plant was designed under the supervision of Mr. Thos. Byrne, who successfully developed this style of tipple in connection with the standard shovel or the Chicago Drinage Canal. The tipple was designed by the Page Engineering Co. of Chicago, and was installed by the writer.

A Portable Derrick for Installation on Motor Trucks.

Hand Operated Derrick Mounted on Motor Truck.

A motor truck equipped with a hand operated derrick for handling loads up to ¾ ton is illustrated here. This derrick is in two parts, a "base" weighing 100 lbs. and a "beam" weighing 128 lbs. The reach of the beam is 3 ft. 4 ins., and the height to the block is 5 ft. The base section consists of a steel post ribbed inside for strength and having a truck collar. This part is mounted on a channel steel right angle base braced to the post as shown. The beam carries the drum and has a ring which drops over the head of the post and also has two rings which straddle the post and carry rollers which ride on the truck collar. The beam has a full circle swing. The drum carries 300 ft. ¼-in. steel cable, and is operated by hand cranks. From the drum the cable passes to the beam head shown and then to

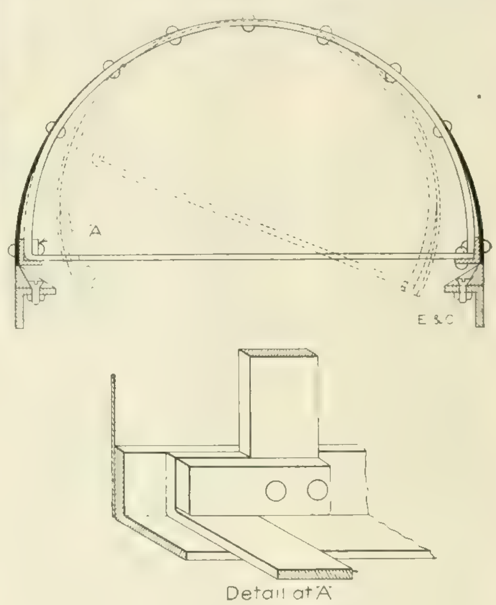


Rope Arrangements for Hand-Operated Derrick.

the block, the different arrangements being as indicated by the beam head sketches: *Single:* Pass cable over top sheave and under roller and then to the block; hoists 300 lbs. 70 ft. per minute. *Double:* Pass cable over top sheave, under roller, around sheave in block, connect to swivel shackle and attach swivel shackle to lower sheave; hoists 600 lbs. 35 ft. per minute. *Treble:* Pass cable over top sheave, under roller, around sheave in block, around lower sheave in beam, around to swivel shackle and attach swivel shackle to the head of block; hoists 900 lbs. 25 ft. per minute. By wrapping the cable around the lower sheave the hoist is geared treble and for short lifts one man can hoist 1,800 lbs. The derrick is tested to 1,900 lbs. in the shop. The derrick is mounted on the truck by simply bolting the bars to the truck frame or flooring. The list price of the derrick for motor trucks is $50. It is made by the Taylor Portable Steel Derrick Co., 1034 West Lake St., Chicago.

A Light Concrete Culvert Form.

The metal concrete culvert form illustrated by the accompanying drawing and view is manufactured by the C. F. Co., Union Bldg., Syracuse, N. Y., to be used in constructing



Details of Metal Concrete Form.

culverts which conform to the requirements of the New York State Highway Commission. It is built of 12 or 18 gage metal, in four sections—two 7 ft. end sections with walls and two 5-ft. barrel sections. Pressed metal end walls and parapet forms are provided. The approximate weight of the form complete, 24 ft. long, 24 ins. wide, and 27 ins. high, is 1,600 lbs. This form is also constructed with a flat top. The end forms are designed so that skew culverts may be built. Metal or wooden side forms of any height may be used.

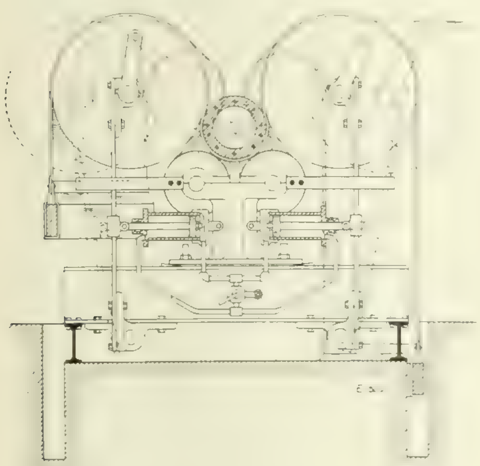
View of Erected Metal Culvert Form.

A Machine for Manufacturing Reinforced Concrete Poles or Piles by Rolling.

(Contributed.)

A concrete pole hollow or solid, straight or tapered, reinforced as desired, in all lengths up to 35 ft. is made by rolling by the machine illustrated here. Referring to the illustration, the compression rolls are made on the same ratio of taper that the manufactured article will be. When pipe is made, straight rolls are used. A very heavy canvas belt is placed on the top of the compression rolls, and the reinforcements and the inner core are laid on this and between the rolls. Each end of the belt is weighted down with an iron bar along its entire width. These weights operate as counterbalances and maintain a tightening effect throughout.

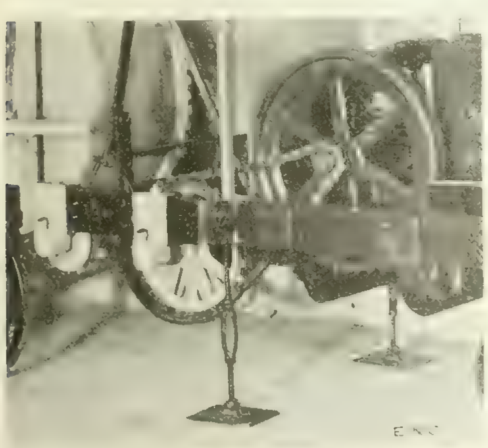
The compression rolls are opened by the cylinders shown, the top rolls are turned inwardly by suitable driving equipment, the counterbalance weights rise, and the reinforcement and inner core are lowered to contact with the idler rolls below. The air or steam is then reversed, imposing a compression upon the collars at each end of the reinforcement and core. The concrete is fed in from the top by means of a measuring and distributing box, and one compression roll is then driven very



Rolls for Manufacturing Reinforced Concrete Poles and Piles.

slowly, not exceeding 10 ft. travel per minute. The product is turned with the concrete applied, approximately two and one-half revolutions in one direction; the driving power connecting this roll is then disengaged by clutch, and the other compression roll is driven in the opposite direction. These reverse operations continue until the product being formed is in shape and thoroughly compressed.

The vertical steel standards composing the frame, in combination with the pressure applied by the air or steam cylinders and the tension constantly retained by the weights upon the belt, cause a more thorough compression effect than a hydraulic press, because the aggregates composing the concrete mass receive their compression from different directions in succession, and the compression is maintained by the belt.



Adjustable Legs to Reduce Vibration of Concrete Mixer.

It is found in practice that this peculiar rolling compression will expel all water that is not actually needed in filling the voids. More water is unnecessary until the curing period is reached, the concrete should then be treated with ample water.

The pole, pile, or pipe becomes a hard and true surface within five minutes from the time the concrete is applied, but in order to retain the moisture in the product during its original setting period, a sheet of canvas wide enough to encircle the product two turns and a lap, coated with a mineral rubber paint, is fed between the belt and the outer surface of the product by placing its edge between them, and revolving the article two turns. Brads with small heads are then driven through the three thicknesses of canvas and into the concrete, and the product is discharged from the forming machine in the following manner: The pressure is reversed, thus parting the top compression rolls. These top rolls are then turned in opposite directions and the counterbalance weights on the belt bring the finished product to the top of the rolls whence it is rolled away to a distant point on the circular curing table. After 24 hours have elapsed, the canvas cover is removed and the product is kept wet until thoroughly cured.

This rolling process in compression causes better tamping effect than can be accomplished in any other manner. The concrete mix is quite dry, but the compression is sufficient to cause water to run from the surface.

The core is composed of sheet steel made collapsible and remains in the pole until the concrete becomes set. The core is then removed; the pole rests in an undisturbed position for five or six days, on the curing table with ample water given to it.

The ratio of taper adopted is 1 in. in diameter for each 6 ft. in length, as applied to poles 6 ins. top diameter. The minimum thickness of the concrete wall is 1½ ins., but 2 ins. are preferable. Poles 35 ft. in length should be 12 ins. diameter at the butt and 6 ins. at the top, or 14 ins. butt and 7 ins. top, or 16 ins. butt and 8 ins. top.

The reinforcement is centrally located in the concrete wall and composed of ¼, 5/16, ⅜, 7/16 or ½ in. square twisted steel reinforcing bars, woven with steel wire warp at desired intervals.

The five sizes named will accomplish the desired differences in strengths for application to various services. Reinforcement consists of 10 bars of any selected size and the entire length, 18 bars two-thirds of the length, and 26 bars one-third the length of the poles. The reinforcement is woven on a tapered mandrel in a specially prepared loom and after being woven and taken from the mandrel, is placed upon the forming core, and the warp which forms the circumferential reinforcement is united by twisting one wire end around the other. The warp may vary in size from No. 6 to No. 10 steel wire spaced at intervals as required for producing the desired strengths. The woven reinforcement is held centrally between the inner core and the outer circle, by pole-step sockets woven between two bars of the reinforcement. A scheme is also provided for centralizing the reinforcements at other points around the pole.

These poles may be formed just as desired, and the reinforcements may be heavy or light and spaced as desired. The labor and concrete cost is the same for all strengths, therefore the steel used is the only element that governs the difference in strength and cost.

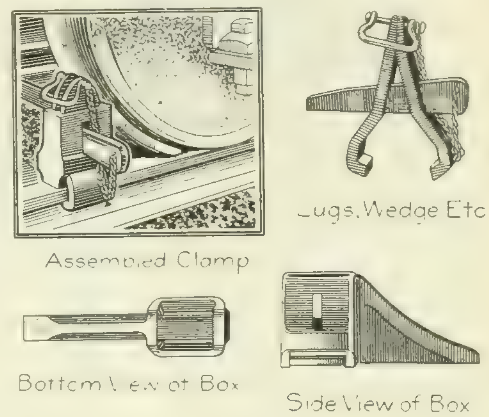
This method of making reinforced concrete poles is the invention of R. M. Jones of Denver, Colo., who states that the National Reinforced Concrete Pole & Pipe Co. having all of the patents of Mr. Jones in the United States excepting California and Territories, has made arrangements by which the company will immediately install a pole and pile manufacturing plant in the near vicinity of Chicago, to be in operation by the first of September, 1914. Shop work on equipment is now being done. R. M. Jones, 1644 First National Bank Building, Chicago, Ill.

A Vibration Absorber for Truck Mounted Concrete Mixers.

Light concrete mixers mounted on front and rear wheels have a considerable length of unsupported frame between axles, and the spring of this frame under the beat of the engine results in vibration of greater or less magnitude depending on the stiffness of the frame girders. To reduce this vibration the makers of the Atlas concrete mixer have as illustrated here provided adjustable legs, one under each side girder about midway between axles. The shaft of the leg is lengthened or shortened by turnbuckles, it is hinged to the flat shoe to permit standing firmly on slope, and it is hinged to the frame so that it can be swung up and fastened clear when moving the mixer. The Atlas mixer is made by the Atlas Engineering Co., 780 Thirteenth St., Milwaukee, Wis.

Rail Clamp for Steam Shovels and Cars.

An adjustable rail clamp which will fit any rail from 60 to 100 lbs. is shown by the accompanying sketches. This clamp is made in two styles for 33-in. and 28-in. wheels. It

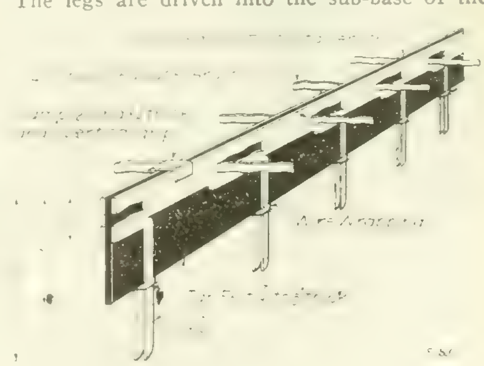


Adjustable Rail Clamp for Steam Shovel and Cars.

clamps the head of the rail and so can be attached anywhere regardless of tie spacing. The bottom bar of the box holds the jaws spread when the clamp is being placed on or lifted from the rail; a hammer blow loosens or tightens the wedge. All parts are made of cast steel. The price of the clamp is $15. It is made by the M. & M. Rail Clamp Co., Room 415 Oliver Building, Pittsburgh, Pa.

A New Joint Plate for Concrete Pavement.

A joint plate which requires no special device or machine for its installation is illustrated here. The plate is of soft steel and when shipped the legs and lugs produced by shipping are flat with the plate. For installation the legs are bent down and the side tongues bent outward by means of a piece of short pipe. This is done on the job. They are then wired together and clamped with the tar paper in between, as shown by the drawing. The legs are driven into the sub-base of the

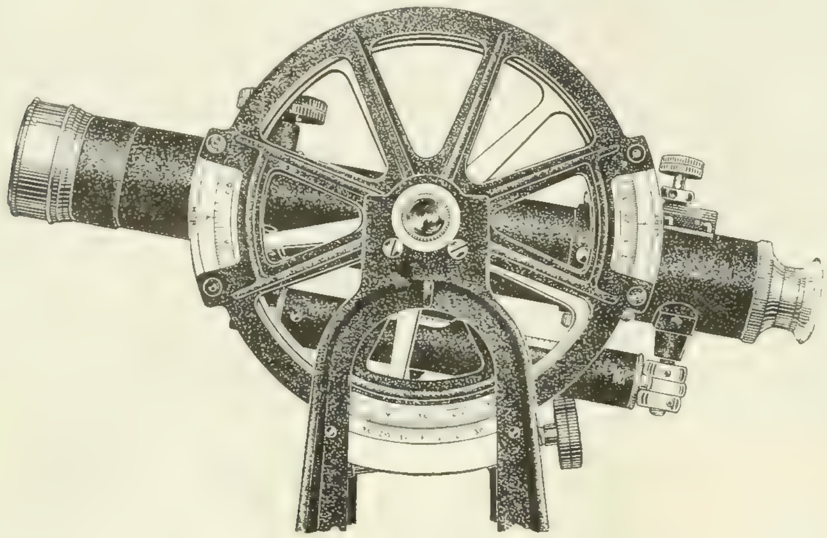


Moyer Joint Plate for Concrete Pavements.

road so that the top of the plate will be the grade of the top of the finished concrete. This is made certain by using a duplicate of the template which will be used in crowning the concrete. This template is merely set on the side forms and the plate hammered down until the bottom of the template will rest accurately on the top of the plate. The joint cannot settle and is convenient to the contractor, as it can be placed as far ahead of the concrete gang as the sub-base has been prepared. If concrete is placed over old macadam this joint may be used by bending the legs out so as to form a chair. This joint plate is manufactured by Albert Moyer, 200 Fifth Ave., New York.

A Stadia Circle for the Transit Head.

The Keuffel and Esser Co. of Hoboken, N. J., has recently applied to the transit a stadia circle, here illustrated, which facilitates the taking of field notes and makes all subsequent computations of a simple arithmetical nature. The stadia circle is a modification of the regular transit circle in which the degree graduations on two opposite segments are replaced by special graduations which give directly the per cent of the observed stadia distance represented by the horizontal and vertical components.



A Stadia Circle for the Transit Head.

Through an arc of approximately 60° at the right and left-hand sides of the circle the degree graduations are replaced by the special stadia graduations. At the index marked Hor. is read the percentage factor to be applied to the observed stadia distance to obtain the correct horizontal distance. At the index marked Vert. is read the percentage factor to be applied to the observed stadia distance to obtain the difference in elevation between the rod and instrument. Subsequent complication in the calculations is avoided by bringing the center cross hair of the telescope to the level target or to a mark on the level rod which has been placed at the instrument height before reading H and V.

Over a long series of tests of the circle by different observers the average error in the reading of the horizontal correction factor was found to be 0.05, which in a 500-ft. sight would introduce an error of 0.25 ft. in the computed horizontal distance. The same trials applied in the reading of the Vertical correction factor disclosed an average error of 0.02, which in a 500-ft. sight would introduce an error of 0.10 ft. By the method of least squares the average error in reading was computed to be: horizontal $+0.09$; and vertical $+0.07$. These larger errors, in a 500-ft. sight, would affect the computation of the horizontal and vertical distances by $+0.45$ and $+0.35$ ft., respectively, and as the allowable error in stadia work is 1 per cent, it is seen that, in accuracy, the stadia circle compares favorably with any method of computation now used.

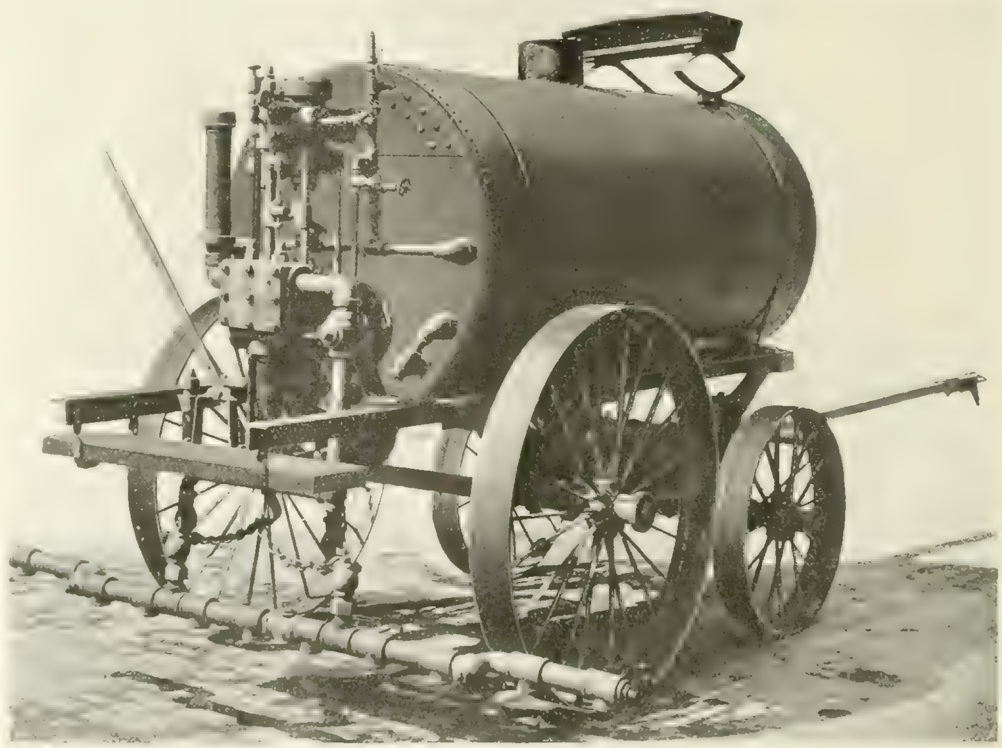
The greatest advantage of the device, however, obviously lies in the rapidity with which field notes and the subsequent calculations can be used, and it is this saving that promises to popularize the device among engineers.

A Pressure Bitumen Distributor of Simple Design.

There are many types of pressure bitumen distributors varying in completeness of equipment from a simple tank and pump to a self driven machine with many accessories. The simpler machines are used primarily in the construction of bituminous macadam and the more complicated types for maintenance and repair work at isolated and widely separated points.

The distributor illustrated is of the simple tensive repairs where the use of a road roller is desirable. It is manufactured by the Standard Mfg. Co., 184 Union St., Worcester, Mass. This machine in its present form is the result of several years' development and the mechanism of it has been reduced to a minimum.

The complete machine weighs 3,500 lbs.



Standard Pressure Bitumen Distributor—Trailer Type.

type suitable for construction work and ex- The tank holds 700 gals. The outfit is drawn by a steam roller which supplies steam to the pressure pump and the steam heating coils in the tank. The relief valve is set at 80 to 90 lbs., and unusually high pressure. The spray pipe is hung on swing joints which permit a lateral motion, controlled by a lever from the operator's platform, of 3 ft. to the right or left. The extended arms of the spray pipe are on swing joints permitting their ends to fold in within the wheel tracks, eliminating danger of striking objects and permitting close storage. The compactness and simplicity of detail of their outfit is worthy of note.

Air Pressures Used in Tunneling.—The pressures of air employed at different tunnels as compiled in Bureau of Mines Bulletin 57 by D. W. Brunton and J. A. Davis have been as follows:

Tunnel—	Lbs	Tunnel—	Lbs.
Carter	112	Nesqually	90-95
Central	120	Rawley	100
Gold Links	100	Raymond	90
Gunnison	90	Rondout	100
Laramie-Poudre	120	Roosevelt	110
Mauch Chunk	100	Siwatch	80
Los Angeles Aqueduct	100	Snake Creek	110
Lucania	115	Stilwell	100
Marshall Russell	110	Strawberry	85
Mission	100	Utah Metals	110
Modern	95-100	Walkill	110
Newhouse	110	Yak	90
Average			102

Recording Sub-Surface Structure in Philadelphia.—The use of field books in recording sub-surface structures has been discontinued in Philadelphia. An 8½x11-in. sketch card is used in the field on which data are plotted. These cards are forwarded to the office where they are properly colored and inked in by the draughting force and filed as an office record. This system does away with much labor and possibility of error in transferring the field notes to office records.

Tractive Resistance of 28-Ton Electric Car.—The Engineering Experiment Station of the University of Illinois has just issued a bulletin, No. 74, on "The Tractive Resistance of a 28-Ton Electric Car," by Harold H. Dunn.

This bulletin records the results of tests made to determine the tractive resistance of a 28-ton electric car when running on straight track in still air. The tests were planned so as to eliminate wind resistance. The results are finally expressed in the form of a curve whose co-ordinates are car resistance and speed, which shows that the resistance varied between 3.25 pounds per ton at 5 miles per hour, and 26.12 pounds per ton at 45 miles per hour. The bulletin contains also a description of the car. Copies may be obtained by writing to the director of the Experiment Station.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design and to Methods and Cost of Construction

Volume XLII. CHICAGO, ILL., JULY 8, 1914. Number 2.

Reform in Making River and Harbor Appropriations.

The river and harbor bill of 1914 appropriates $43,289,000. Of this amount some $5,000,000 are alloted for beginning works whose completion will necessitate a further expenditure of $33,000,000. There is no essential difference between this bill and the river and harbor appropriation bills of many previous years. It is no more nor any less illogical and wasteful. Like previous bills it will very likely be passed by congress. There is no word that it will not be approved by the president. It will, however, not become law without contest, and this is the point on which for several reasons comment is pertinent.

The present contest against the river and harbor bill is unique in the degree to which it is directed against the system by which appropriations are determined. It is notable also because in congress the attack is perhaps more concerted than ever before, and because it has enlisted to a greater extent than previously the support of powerful newspapers. The contention by the bill's opponents in congress and by the papers which uphold their objections is not that federal expenditure for river and harbor improvement is unwarranted; it is not primarily an attack on specific items; it is an argument against continuing a vicious procedure in formulating such bills. This statement is needed to clear the ground. Contractors and engineers who draw business from river and harbor work naturally regard with disfavor any action which may curtail its volume. They are apt to conclude without much consideration that opposition, whether within or outside of congress, to river and harbor appropriations, is designed directly or indirectly to decrease federal expenditure for river and harbor improvement. It is important, then, to make it plain that this conclusion is erroneous.

The present method of making federal appropriations for river and harbor improvement is vicious. Consider the procedure followed, and put to oneself only the two fundamental questions which follow: Where is the line drawn between streams and harbors that are worth improvement and those that are not? What proportion is deemed proper between the commerce benefited and the amount of money expended on a given improvement? The answers must be that there is no rule or principle followed; that both choice of work to be done and allotment of expenditure depend upon the power of local lobbies and the members of congress from different localities.

But, it will be urged, every project for improvement is passed upon by the army engineers. This is true, but disapproval by the army engineers by no means prevents adoption by congress of a project. Many instances can be mentioned of appropriations made for improvements not approved by the army engineers. The army engineer does not initiate the various items composing the appropriations bill. His labor is restricted to carrying out in the field the work which congress authorizes. In making recommendations he is restricted to approval or non-approval of the item of work specified, without regard to the claims of other items. The congressman asks that surveys be made and a report presented; a complacent congress grants his request; then army engineers make the surveys and the report ordered by congress; the congressman then sets about getting the improvement included in the river and harbor appropriation bill and seeing that it is not afterwards cast out, and if he has influence enough he succeeds, even against an unfavorable army engineers' report; congress passes the appropriation bill; the army engineers then plan and construct the improvement which the congressman sought. As amusingly put by Congressman Frear in his speech of March 26 against the 1914 appropriation bill:

> Responsibility, like a caged squirrel, runs round and round from the congressional recommendation for a survey to the engineer's report, based on the recommendation, and after it is all over the engineer takes the blame because, although acting at the behest of the local member, as shown by these reports, he cannot squeal for fear of losing his job.

Nor is the congressman so much to blame for, as he is the victim of, the system. He is more often than not judged by his constituents on the basis of the amounts of local appropriations secured, and so is compelled, whatever his opinion is of their justification to seek appropriations.

As already said, the present method of making appropriations for river and harbor improvements is vicious, and any sincere and sensible attempt to improve the method deserves encouragement. Of what form the improvement shall be is a moot question. Senator Burton in his recent minority report against the 1914 appropriation bill recommends:

> 1. Provision for the completion of an improvement in the bill in which it is adopted.
>
> 2. Careful review of pending projects in the light of present conditions and the omission of all improvements which are no longer profitable.
>
> 3. A more careful consideration before the adoption of projects. There should be especial care to avoid lock and dam construction save in streams which are capable of being made important arteries of commerce.
>
> 4. A division of expense when exceptional advantages accrue to private property or specific localities, or when the protection of private property is the main object and navigation subordinate.
>
> 5. The exclusion from the bill of proposed improvements which do not have to do with navigation.
>
> 6. A general policy of improving the main stream before attention is given to branch streams, and the adjustment of depths and dimensions, with a view to a uniform and comprehensive plan for the development of such waterways as can be utilized.
>
> 7. Such adjustment of the relations between railways and waterways as will secure the utilization of routes partly by land and partly by water when co-operation and utilization by both routes can be made profitable as a means of transportation.

No thinking man, we imagine, will find any statement in this quotation of which he cannot approve. The question is how are we to arrive at these reforms? The only way is the creation of a department or a bureau which shall initiate and plan river and harbor improvements, and against whose recommendations congress will proceed only upon strong justification. Such directing authority may be a new federal department of public works or it may take some other form, but it is only by such means that present faults can be avoided.

The County Administrative Unit in Road Construction and Maintenance.

Students of economic problems connected with country road building have noted in recent years a marked tendency toward strengthening the authority of the county as a unit in highway maintenance and construction. This is indicated by the recently enacted state highway laws of Iowa, Wisconsin and Kentucky and the strengthening of the New York and Illinois laws. This trend of legislation shows a healthy condition in road affairs and, it is possible, may result in eventually placing our highway laws on the sound basis so badly needed.

Road officials and legislators in attempting to follow too closely successful European systems of organization—notably that of France—neglected to some extent the old established and strongly organized county unit (which is an English institution) and reproduced in certain instances as faithful copies of this foreign system as local conditions would permit. As in all copies important details of the original were lost.

Perhaps, also, the early development of modern highway construction in some of the smaller states in which the county unit is not so strongly organized as in the average state of this country, and the success of the system adopted there, may have influenced the framers of highway laws. Notwithstanding the success of some of these organizations, the return to a distinctly American and English type of organization is to be commended. Its simplicity is undoubtedly preferable to a piling up of commissions and officers each having authority over a phase of the same general work—a condition of affairs which exists in several state highway organizations.

In this connection, an important point in favor of the selection of the county as the unit of administration which will become more important in the course of time, is the necessity for strongly organized maintenance districts. Political units gather strength not so much by laws of establishment as by age and the habits of thinking of the citizens of the unit. It is probable that by the selection of suitable officials under the regulation of the state commissions efficient executives may be secured. For maintenance work the continuous, personal supervision of a resident county engineer who is also a capable executive is desirable. A study of maintenance work in English counties and the importance of the duties of the county engineer is enlightening. On the other hand, the desirability of undertaking new construction except under the close supervision of a state commission is perhaps open to question.

It would seem that the logical and common sense plan of organization to which, according to all indications, the larger states having a largely rural population are coming, is the plan by which a small, efficient state commission supervises the work of the various counties: a simple plan and one adapted to American conditions.

Methods of Appraising Water Rights and an Important Court Decision Relating to Water Rights Values in Rate Making Cases.

Since the tendency of public service commissions, other than those of Wisconsin and Washington, has been to regard water right values as being analogous to franchise values, and therefore not to be considered in a rate case, the decision rendered April 27 by the

Supreme Court is far reaching in its importance, for it affirms that water rights must be valued in a rate case. Just two weeks prior to this decision the Public Utilities Commission of Idaho had said:

What has been heretofore said concerning the inclusion of other intangible value, such as "going concern," "good will" and "franchise value [All of which the Commission excluded.—Editor] would apply to the inclusion of what is termed "unearned increment value," accruing by reason of the right to the use of water for the purposes to which it has been dedicated. We find, from a careful examination of the authorities cited, and these examined in our own research, that the courts and public utilities commission refuse to allow a valuation of this character other than the actual cost thereof for the purpose of fixing rates (Appl. of Pocatello Water Co.).

The Idaho Commission apparently overlooked two decisions of the Wisconsin Railroad Commission on water right values.

On Jan. 29, 1914, the Public Service Commission of Nevada rendered an opinion in the case of the Nevada-California Power Co., involving the water right values of that company. The opinion was unanimous except as to water right values. One member of the commission held that water right values should be excluded, except in so far as actual cost of securing them was concerned. But the other two members dissented; and Mr. Bartine, Chief Commissioner, wrote an excellent summary of the law of water rights, from which he concluded that water right values could not be excluded.

In the Truckee River General Electric Co. case, decided May 27, 1914, by the Nevada Commission, water right values were also involved. In this case, as well as in the one previously mentioned, the water right values had been appraised by Mr. H. P. Gillette, consulting engineer for the companies. While the majority of the commission favored allowing a value for water rights, they did not indicate what value nor how to ascertain it. From the last named decision we quote:

It does not appear that the water controlled by the company, and used in rendering the service now under consideration, could be used in any other way that would give it a value even approximating the estimate of Mr. Gillette. Clearly, a water right, for which there is only one use, and, consequently, one demand, is not so valuable as it would be with many uses and many demands for it. If the present patronage of this company were to be withdrawn, it is difficult to see any other use to which its water rights and privileges could be put that would bring it more than a small fraction of its present earnings.

Does not this line of reasoning "prove too much?" "If the present patronage of this company were to be withdrawn," not only would the value of its water rights suffer, but most of its power plant would lose its value. In other words, a reason that is tenable as an argument against the estimated value of a company's water rights is equally tenable against the estimated value of its entire plant —generating stations and transmission and distribution system. The Commission says:

In a number of such cases the courts, while recognizing the fact that a franchise has value, have been unable to say how much that value was, for the reason that there was no satisfactory evidence going to that point. The same difficulty exists with respect to the respondent company's water rights.

We fail to see the analogy between vague franchise values and definite water right values. Mr. Gillette calculated the water right values in these cases, using several different methods, which were fully described by him in two articles that were published in Engineering and Contracting, Apr. 17 and Dec. 4, 1912. No evidence was submitted to controvert Mr. Gillette's methods or his final conclusion as to water right values.

We might quote from the decision (Jan. 13, 1912) of the Railroad Commission of California in the Northern California Power case, as also indicating opposition to including water right values as a basis for estimating adequate net earnings. Similar opposition has appeared in one of the decisions of the New Hampshire Commission in a capitalization case. In general, then, the attitude of public service commissions has been unfavorable to capitalizing the profits derived from the ownership of water rights. Therefore the recent decision of the Supreme Court, given in full in another column, is of particular interest to companies owning water rights used or usable for power, irrigation or municipal supply. In brief, the court decision makes it compulsory to estimate water right values and to include them with the value of the rest of the property in arriving at a base upon which to calculate the "fair return."

We have never been able to understand the position of those who concede that the "unearned increment" on land belongs to a utility company yet who deny its right to the "unearned increment" on water rights. The Supreme Court has now settled this question, but there still remains in controversy the method to be used in estimating water right values.

Mr. Gillette has proposed what he calls the "capitalized profit method," and has used other methods as a check upon the reasonableness of the conclusions reached by this method. Among these other methods is the "next available source of supply" method. This latter method, by the way, was entirely misinterpreted by the California Railroad Commission in the Eureka Water Co. appraisal decision of Mar. 23, 1914.

The company's representative testified that to go to Mad River, the next available source, would cost $100,000 more than to use the existing source of water supply, Elk River. From this the inference was drawn that the Elk River water rights were worth $100,000. Curiously enough the Commission failed to grasp the logic of this inference, for it says:

But, of course, the $100,000 excess cost has no reference whatsoever to the value of any right to take water from Elk River, and is purely the relative computation and gives no light whatsoever upon the proper value to be put upon the Elk River right, if such right is to have value at all. For, plainly, if cost is to be the criterion, as here urged, and the cost of the Elk River rights is $25,000, then the cost of the Mad River rights would be $125,000. While if the cost of the Elk River rights is $100,000, the cost of the Mad River rights would be $200,000, and only in the event that the cost of the Elk River rights is nothing, is the cost of the next available source of Mad River to be assessed at $100,000. And we have here a situation where the net result of the reasoning of the applicant is that if cost is to be the criterion of value, its Elk River rights have no value, and if cost is not to be the criterion of value, then the excess cost to develop the Mad River supply can not be considered. Because of the fact that this theory is being urged in other cases, as well as this one, I believe it well to state definitely that whatever shall be the final determination of the Commission on this question, that certainly the theory of the next available source must be rejected as being absolutely untenable for any purpose whatsoever.

The Commission has erred in reaching its conclusion. First, it has erred because it inferred that the $100,000 was to be added to the *cost* of the water rights of Elk River, when, in fact, it should be added to the *value* of the water rights of Mad River, in order to arrive at the *value* of the water rights of Elk River. Thus, if the Mad River water rights had neither a positive nor a negative value, but had a zero value, the Elk River water rights would have a value of $100,000. But it might be that the Mad River water rights actually had a minus value as a supply for the city of Eureka, as would be the case if the total annual income from the sale of the Mad River water would not equal the operating expenses and fixed charges on the plant used to deliver the water. The Commission did not undertake to show a minus value for the Mad River water rights, which was the only logical method of attacking the $100,000 valuation of the Elk River water rights. On the contrary, the Commission fell into a curious error through failure to understand the method it was criticising.

Perhaps the error originated from lack of understanding of the principle of capitalizing annual *costs* as means of determining the difference in *value* between two alternative instruments of production. To illustrate, if it *costs* $120 a year more to haul farm produce from farm A than from farm B, and if money is worth 6 per cent, then the capitalized *value* of this $120 annual difference is $2,000. Hence, if all other things are equal, farm B has a value $2,000 in excess of farm A. Similarly if a distant water supply *A* causes an annual cost that is $6,000 greater than is incurred with supply *B*, then supply *B* has a value of $100,000 in excess of supply *A*, if money is worth 6 per cent. The Commission missed the reasoning entirely, for it speaks of the cost of Mad River *water rights* when it should be speaking of the excess cost of the Mad River *plant*. It is this excess cost of the Mad River *plant* that should be used as a criterion by which to judge the *value* of the Elk River *water rights*.

This is not the first instance of a Commission's failure to understand the principle of capitalizing annual *costs* as a means of determining the relative *value* of alternative plants or properties. Engineers, therefore, should be particularly careful to explain every step in such a process, for what seems quite elementary to an engineer is often confusing to an attorney or to a business man unaccustomed to engineering problems in economics.

Concerning Special Building Privileges

Building ordinances should, above all things, stand for safe construction and impartiality. As so many interests are concerned in such ordinances it is evidently an exceedingly difficult task to frame a building code which will be fair to all concerned. It therefore follows that wide publicity should be given to such ordinances before a final vote is taken to adopt them. After they become law no violation of their provisions should be permitted without equal consideration and publicity. It is doubtful if the interests of the public are best protected by giving a small committee the power to permit infractions of the provisions of a building code.

There is, of course, difference of opinion as to the limiting height of buildings and as to the fairness of the "zone" system. In considering such matters as these the rights of the entire population of the city must be given careful consideration, even though individual interests may be seriously affected. Great pressure is often brought to bear upon building committees to permit modifications and slight violations of building provisions, and any tendency to rush such infractions without permitting wide publicity may well be looked upon with suspicion.

The recent action of the city council of Chicago in defeating by a decisive vote an ordinance intended to remove the 200-ft. building limit in a very restricted "zone" along Michigan Ave., and to permit buildings in this district to build to 260 ft., is to be commended. The ordinance was introduced at the behest of the owners of a hotel property on Michigan Ave., and it had received the recommendation of the committee on buildings by a vote of nine to three. This ordinance specified that buildings 260 ft. high could be built on streets 130 ft. or more in width adjoining parks or public grounds, the dedication of which was such that buildings could not be erected thereon. It is evident that such restrictions made the ordinance one of exceedingly limited scope —it is claimed that Michigan Ave., facing Grant Park, is the only street in Chicago which meets the provisions of this ordinance. The fact that the ordinance was introduced at the last meeting of the city council before summer adjournment—the same day that it was approved by the building committee—is sufficient cause for its defeat; as an ordinance of such importance to the public should have full consideration before a final vote is taken. We believe that the overwhelming defeat of this ordinance, under the stated conditions, will do much to prevent the granting of special privileges in the future.

GENERAL

Design of and Test and Operating Cost Data for New Municipal Refuse Incinerator at Regina, Saskatchewan.

In the early part of 1907 the City Commission of Regina, Saskatchewan, decided to erect an incinerating plant to care for the refuse of their city. The contract was let to the Decarie Incinerator Co. of Minneapolis, Minn., and a 50-ton, single-unit plant of the steel water-jacketed type was installed at that time. The incinerating furnace proper was constructed entirely of steel 10 ft. square by 12 ft. 6 ins. high, in side dimensions, with a 4-in. water space on all four sides and with a 2-ft. steam and water space above the crown sheet. Along two sides of the furnace were placed 1½-in extra heavy pipes connected to the crown sheet at the top and to the firebox sheets at the bottom. The pipes were spaced at 9-in. centers and bent so as to form a basket grate to receive the refuse which was charged in from the wagons on the floor above through four 3-ft. square hopper openings in the crown sheet. By this means the refuse was suspended in an indestructible grate about 3 ft. above the lower or cast-iron shaking grates on which the material was finally consumed, giving the fire free access to all parts of the newly charged material without in any way obstructing the draft, or deadening the fire below.

This one 50-ton unit was installed in a brick building with wooden driveways to the upper floor, enabling the refuse wagons to come in on the floor above and deposit their loads directly into the furnace. Nothing but natural draft was provided and that was furnished by a steel stack 135 ft. high by 4 ft. diameter at the top.

At that time it was thought that a plant of 50 tons daily capacity would be amply large for several years to come; but Regina has shown a phenomenal growth during the past few years and late in 1912 it was plainly seen that in a few months the plant would need to have additional capacity. Consequently, steps were immediately taken by the city commissioners to outline a larger refuse disposal system for the city and one of sufficient size to care for future requirements. The city already had under construction additions to their 2,500,000-gal. sewage disposal plant which is located in the extreme northwestern portion of the city. The pumps to be installed here were to be electrically operated, and immediately the question arose as to the advisability of utilizing the heat generated by the burning refuse to help furnish the power for operating these pumps. It was known that the heat from a city's refuse is converted into electrical energy in many European cities and used to produce a source of revenue. It was also known that the incinerating plant built by the Decarie Incinerator Co., at Minneapolis, had for years been furnishing the power to light several wards of the city, and, consequently, it was decided to investigate the feasibility of such an arrangement in connection with the sewage plant.

Owing to the satisfactory results obtained by the original incinerator it was decided to let the Decarie Co. make the city a proposition on a plant of larger capacity installed at the sewage disposal works with means provided for converting the heat generated into electrical power. The company agreed to construct a modern plant at the site of the sewage disposal works for the sum of $64,000, the plant to consist of a new 60-ton unit and the old 50-ton unit renovated and moved into the same building with the new one, making in all a plant with a guaranteed capacity of 110 tons in 24 hours. The plant also was to be equipped with two 100-HP. Babcock and Wilcox water tube boilers, together with forced and induced mechanical draft.

In May, 1913, the contract was awarded to the Decarie Incinerator Co. at the above figure. On December 1 the new plant commenced operation. The essential terms of the proposal as furnished by the contractors to the city were as follows:

CONTRACTOR'S GUARANTEE.

The new unit to be installed should have a capacity of incinerating 60 tons of refuse in 24 hours and the old unit after being remodeled and installed in the new plant should have a capacity of incinerating 50 tons of refuse in 24 hours. The refuse was understood to consist of kitchen garbage, combustible material, manure, and dead animals mixed together in proportions as created by the city of Regina from day to day, no attention being paid as to the selection of any particular kind of refuse or garbage. It was understood that manure would constitute 60 per cent of the refuse to be destroyed. All material delivered to the incinerator to be destroyed without creating any noxious odors or gases.

The total cost of operation was not to exceed 65 cts. per ton and if credit were given for power developed by the boilers, at the rate of 70 cts. per 1,000 lbs. of steam, the cost to be reduced by one-half, and if 30 per cent of the total refuse proved to be dry combustible the addition of fuel would be unnecessary.

It will be noted that the guaranteed cost of operation is somewhat higher than this firm usually guarantees. This was due to the large percentage of manure that had to be handled. However, as will be noted from the results of the final tests here given, this was a safe guarantee as the operating cost came to only 40 cts. per ton when burning 60 per cent of manure. The old plant had a guaranteed cost of 50 cts. per ton and operated at an average of 30 cts. per ton for the five previous years. The old plant handled no such percentage of manure as the new plant was required to handle. In fact in the old plant the city had difficulty in consuming the manure without additional fuel, as this plant was equipped with no mechanical draft whatever.

DESIGN OF PLANT.

The new plant is located on the bank of Wascana Creek, the building being of concrete, brick and steel construction 44 ft. by 54 ft. inside dimensions. Two sides of the first floor of the building were formed by the concrete retaining walls. The remainder of the building walls are of brick. The hopper or charging floor is of reinforced concrete supported on steel beams. The standard gage steel car track, on which the refuse is delivered to the plant in special dump-cars, passes through the building at the hopper floor level. A plan of the operating floor is shown in Fig. 1.

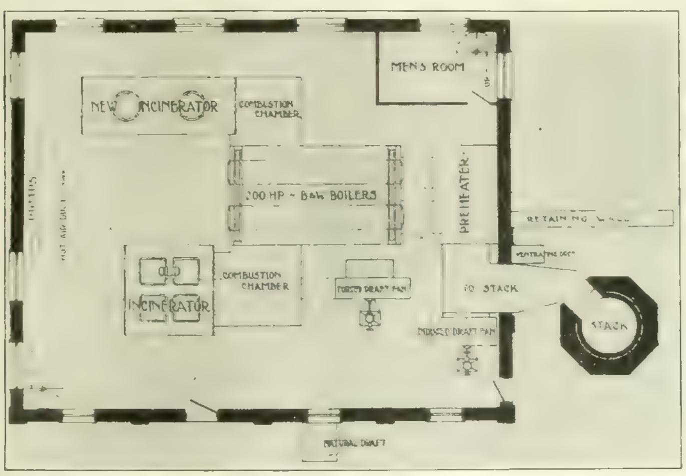


Fig. 1. Plan of Operating Floor of New Municipal Refuse Incinerating Plant at Regina, Saskatchewan.

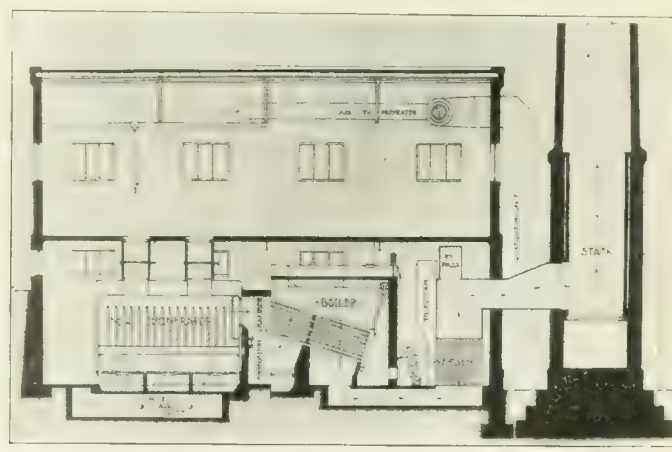


Fig. 2. Longitudinal Section of Regina Refuse Incinerator.

The roof is of 3-in. concrete slab construction covered with tar and gravel roofing supported on steel purlins and trusses. I-beam trolley tracks are attached to the under side of the trusses over each unit, on which chain blocks operate to raise the heavy cast iron hopper covers and also the carcasses that are brought to the plant for destruction.

All material as it is delivered to the plant is dumped directly into the furnaces from the cars, without storage of the refuse being necessary. Drain connections to the sewer as well as water and steam connections are provided on the charging floor for keeping the cars and the floor in a sanitary condition. The incinerating furnaces and boilers, together with all the other necessary machinery, are located on the lower or operating floor. The general arrangement of the plant is shown by longitudinal and transverse sections in Figs. 2 and 3.

The two separate units are of 60 and 50 tons capacity as stated above. Each unit consists of an incinerator, combustion chamber and one 100-HP. Babcock & Wilcox boiler. A pre-heater or regenerator for heating the forced draft fan, direct connected to a the chimney. An American Blower Co.'s induced draft fan, direct connected to a 11-in. x 8-in. automatic high speed self-oiling steam engine, is provided, as well as an American forced draft fan, direct connected to a

9-in. x 7-in. steam engine of similar construction. Four $5\frac{1}{2} \times 3\frac{1}{2} \times 6$-in. March feed pumps are provided; one for each incinerator and one for each boiler.

The chimney is 5 ft. diameter at the top and 125 ft. high, of radial brick construction with octagonal, common brick, base set on a heavy concrete foundation just outside the building wall.

As can be noted from the illustrations the new incinerator is made with larger and narrower dimensions than the old unit; being 6 ft. wide by 18 ft. long by 10 ft. high, inside dimensions. It has 4-in. water legs and a 2-ft. steam and water space above the crown sheet. With these proportions the labor necessary for stoking is materially reduced, and as the number of stoking doors is much smaller, not nearly as much cold air is drawn into the furnace.

The longitudinal header, connecting the two end water spaces and into which all the 2-in. basket pipes are connected at their lower ends, is made of 12-in. extra heavy pipe in the new incinerator and 8-in. extra heavy pipe in the old one. As will also be noted from the sectional illustrations the pipes forming the basket grate do not connect into the inside plates but come through the water leg and by means of union tees and elbows connect into the outside sheets. This arrangement enables these pipes to be kept thoroughly clean by means of plugs located in the tees.

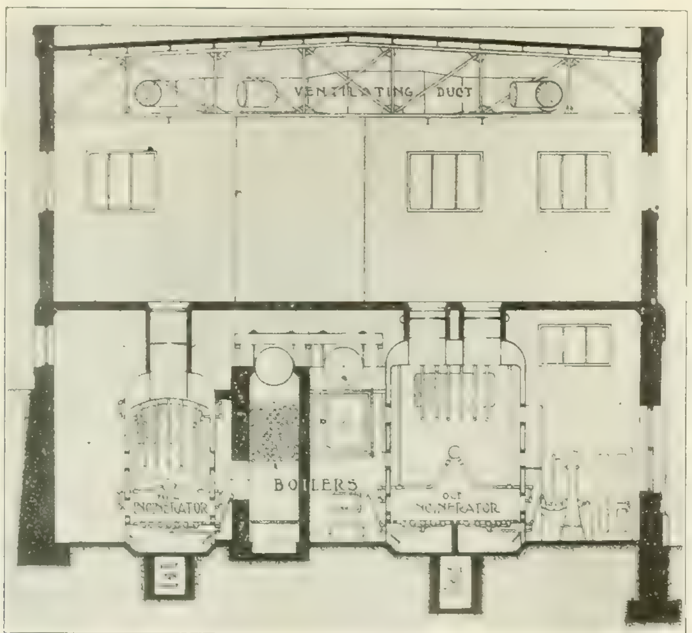


Fig. 3. Transverse Section of Regina Refuse Incinerator.

OPERATION OF PLANT.

The refuse enters from the upper floor through the balanced doors in the brick lined hoppers and drops into the basket grate. As it burns in this position, and on the shaking grates directly beneath, the gases pass into each respective brick-lined, steel-cased cumbustion chamber where they are thoroughly mixed and burned before entering the boilers through the side wall directly over the boiler grates. The gases then pass around the water tubes of the boiler giving up a large amount of their heat to the generation of steam in these tubes. It will be noted that by this arrangement the boilers can be operated simultaneously with the waste gases and with coal in case the heat from the refuse is not sufficient to maintain a constant pressure. This condition is likely to occur in wet weather or when the heat value of the refuse is exceptionally low.

After leaving the boilers the gases pass under the floor and up through the pre-heater, or generator, direct to the chimney; or they may be drawn through the induced draft fan to the chimney as the conditions may require.

The one pre-heater and the induced draft fan serves for both units or for either one of the two units if only one unit is in operation. The pre-heater contains 740 2½-in. boiler tubes expended into ⅜-in. steel plate heads, the gases passing through these tubes on their way to the stack. The air supply for the forced draft fan is taken through a duct leading from the ceiling of the upper floor to the pre-heater, thus removing all the foul air from the building that might come from the refuse as delivered. This air is drawn through the pre-heater around the tubes through which the hot gases are passing, taking up enough of this heat from the waste gases to raise the temperature of the air going into the forced draft fan to the temperature of from 150° to 400° F. The forced draft fan forces this heated air through concrete ducts below the floor as indicated in Fig. 1 to the ash pits of the two incinerators. It enters the pits directly under the grates through suitable control nozzles at a maximum pressure of 4½ ins. It might be stated that the ash pits of each incinerator are divided into sections so that the forced draft can be shut off entirely, in any one section and the fire cleaned on that section of grates without hindering the operation of the other sections of the grate. The use of this hot forced draft has shown itself to be the most essential feature in the burning of manure and the saving of additional fuel. In case only natural draft is used, as in starting up when the steam pressure is down, an opening is provided to the outside of the building so that air can be drawn directly into the ash-pit, thereby removing the necessity of opening the ash-pit doors. This feature is especially valuable in the winter time when the lower floor is well sealed up to prevent the cold air from coming into the building.

Each unit is also provided with a by-pass direct from the respective combustion chambers to the chimney, thereby enabling the boilers and pre-heater to be entirely cut out in case, for any reason, repairs should be necessary, or if occasion should arise, the boilers and incinerators can be operated entirely independent of each other.

The boilers are installed to work at 160 lbs. pressure per square inch; the ultimate plan being, as stated, to use the steam from the boilers for the generation of the electric current for the sewage pumps. The generator set has not yet been installed, but very likely will be in the near future.

The incinerators are designed to operate at 100 lbs. per square inch steam pressure, but in the continuous operation of the plant as a steam generating station each incinerator operates as a feed water heater for its respective boiler, the fans being operated by the steam from the boilers. The piping is arranged so that in starting up the plant, or, when the boilers are shut down, the incinerators can furnish the necessary steam for the fans and pumps.

The blow-offs from the incinerators lead into the respective combustion chambers and those from the boilers lead to the atmosphere through a 5-in. pipe. The exhausts from the pumps and fan engines pass through a 6-in. pipe directly into the base of the chimney.

A men's room containing toilets, wash stands and lockers, as well as other modern conveniences is provided on the operating floor.

The installation of the complete equipment, except the old incinerator, was completed Dec. 1, 1913, and the plant commenced operation that day. It was agreed by the city that a test of the capacity for the 60-ton unit would determine the fulfillment of the guarantees as the city could not without great inconvenience supply more than 60 tons of refuse in 24 hours. The 50-ton unit could not be dismantled at the old plant until the new plant was in operation and consequently the installation of the old unit in the new plant has just recently been completed after it has been overhauled and equipped with the new system of piping.

REFUSE HAULING.

Near the site of the street car barns the city has erected a loading station and a stable, and it is here that collection wagons deliver their loads of refuse into the dump-cars.

The loading station is a brick and reinforced concrete building two stories high 46 ft. x 146 ft., and was erected at a cost of $16,000. A siding of the Municipal Street Railway runs through the building at ground level and in this siding are placed the 5-cu. yd. steel dump-cars especially designed for this service. The loaded collection wagons enter on the second floor by means of an approach at the end of the building and dump their contents directly into the cars.

The stable is also of brick construction erected at a cost of $23,000 and provided with single stalls for 32 teams, together with 4 box stalls. The wagons are housed under the second floor of the loading station.

TEST DATA.

The first of the three test periods held on the new plant occurred on Dec. 1, 1913, under the supervision of the builders, with the following results:

Date of test—Dec. 1, 1913.
Duration of test, hours—22
Grate area, sq. ft.—108.
Material incinerated—

Manure, lbs.	57,240
Garbage, lbs.	55,480
Fruit, lbs.	2,800
Fish, lbs.	3,500
Horses, lbs.	1,500
Total material destroyed, tons	60.26
Percentage of manure	47.5
Percentage of ash	10.0
Refuse burned per hour, tons	2.74
Equiv. incineration in 24 hours, tons	65.76
Labor required—	
1 engineer 22 hrs. at 50 cts.	$11.00
3 firemen 22 hrs. at 30 cts.	19.80
Labor cost per ton, cts.	51.1
Cost of fuel required—200 lbs. coal and ¼ cord kindling	$3.00
Total cost of fuel	$3.00
Fuel cost per ton refuse, cts.	.049
Total cost of operation per ton refuse, cts.	.56

Although this first test fulfilled the guarantees of the builders the city desired to run another test with the regular city crew operating the plant. They also desired to have the requisite quantity of manure on hand to make up the 60 per cent that was stipulated in the contract. Consequently on Dec. 12, 1913, a second test was run with the city's crew operating the fires, the following results being obtained:

Date of test—Dec. 12, 1913.
Duration of test, hours—9.5.
Grate area, sq. ft.—108.
Material incinerated—

Manure, lbs.	23,310
Garbage and refuse, lbs.	24,460
Total, lbs.	47,770
Total material destroyed, hrs.	23.89
Percentage of manure	48.8
Percentage of ash	10.0
Refuse burned per hour, tons	2.51
Equiv. incineration in 24 hours, tons	60.24

Refuse burned per sq. ft. grate per hour, lbs. 46.5
Labor required—
Engineer, 9½ hours at 50 cts. $4.75
3 firemen 9½ hours each at 25 cts. 7.12

$11.87

Labor cost per ton, cts. 49.7
Fuel required—No coal used. Small amount kindling wood to start.
Total cost fuel required, dollars 0.50
Total cost per ton refuse, cts. 2.1
Total cost of operation, per ton, cts. 51.8

The plant demonstrated in this second test that it could be operated within the guaranteed cost by the city's men, but again the 60 per cent of manure was not available. Consequently a third and final test was held on Jan. 7, 1914, the results being as follows:

Date of test—Jan. 7-8, 1914.
Duration of test, hours—19.0.
Grate area in sq. ft.—108.
Material incinerated—
Garbage and refuse, lbs. 48,870
Manure, lbs. 74,100

Total, lbs. 122,970
Total material destroyed, tons 61.49
Percentage of manure 60.3
Percentage of ash 10
Refuse burned per hour, tons 3.24
Equiv. incineration in 24 hours, tons 77.76
Refuse burned per sq. ft. grate per hour, lbs. 60.0
Labor required—
1 engineer 19 hours at 50 cts. $9.50
2 firemen 19 hours at 25 cts. 9.50

$19.00

Labor cost per ton, cts. 30.9
Fuel required—
½ cord wood $4.00
200 lbs. coal 1.00
Total cost of fuel required, dollars 5.00
Fuel cost per ton refuse, cts. 8.1
Total cost operation per ton refuse, cts. 39.0

The information given in this article was furnished this journal by the Stacy-Bates Co. of Minneapolis.

Discussion of Depth of Drill Holes Most Advantageous in Driving Tunnel Heading.

Accompanying a tabulation of the depths of drill holes employed in driving heading for a number of American tunnels, Mr. D. W. Brunton and Mr. J. A. Davis in Bureau of Mines Bulletin 57 present a discussion of the advantages of shallow and deep holes which, with the tabulation, we abstract as follows:

During the past four or five years there has been some difference of opinion among students of the problems of tunnel driving, as to the proper depth for drill holes in tunnel headings. In view of some of the remarkable results attained in driving the Simplon and Loetschberg tunnels, where, as is agreed by everyone, the holes were much shallower than those in American practice, the question has been raised as to whether the holes in the tunnels of this country are drilled too deep. Numerous tables have been prepared in support of this argument, from which it appears that at most European tunnels the progress is much greater (in some cases more than twice greater) than that of tunnels in America. At the same time consideration is not always given the fact that in many instances the records are by nature in no wise comparable; for in Europe, at the majority of tunnels cited, the work was conducted throughout the entire 24 hours of each day, whereas in America in many instances only two shifts (and, indeed, in some only one) were employed daily. Then, again, the nature of the rock exerts an all-important influence upon progress, and in many cases this has been to the advantage of the European tunnels. A notable example of the influence of the rock encountered is found at the Loetschberg Tunnel, where the same methods and practically the same equipment were employed at the different ends, the north end working in limestone and the south end in gneiss and schist. The progress attained at the south end was much less than that at the north, in some months the progress in the north end being nearly double that in the south. Other considerations, also, especially the labor and the cost of driving, enter into the problem in such a manner as to make it impossible to say (when everything is taken into account) that the greater speed in European tunnels is due solely to the use of extremely shallow holes. That in many instances the holes in American tunnel headings are too deep, however, is impossible of denial, and hence a discussion of the factors that enter into the determination of the proper depth of holes is extremely desirable.

One of the chief advantages arising from the use of shallow rounds is (when the holes are properly directed) the increased efficiency obtainable from a given charge of explosive; for, as the width of the heading is for all practical purposes constant, the angle between the line of least resistance and the axis of the bore hole becomes a function of the depth of round, the width of the angle increasing with shallow holes. This advantage obtains especially with the wedge-cut and with the pyramid-cut, and it should be a fundamental consideration with the bottom-cut method of drilling the holes. Strangely enough, however, in the Loetschberg and the Simplon tunnels, which are so often cited as examples of the "highly desirable" European practice of using shallow holes, this advantage was almost if not entirely thrown away because the holes were drilled in vertical rows and were nearly parallel to the bore of the tunnel. In such a case the line of least resistance and the axis of the bore hole are nearly coincident, a condition that results in the production of the least possible efficiency from the charge of explosive, and it can not be gainsaid even by the advocates of this method that a much greater quantity of explosive was required to break the same amount of rock than is usual in American practice. If to this is added the fact that such a system utterly ignores the advantage to be obtained from connected drill holes by the concentration of explosive at the apex of the core of rock to be removed, there is strong ground for rational suspicion that the extreme shallowness of the holes used in these tunnels was adopted from necessity rather than from desirability; with this system of drilling and directing the holes the difficulty of blasting out the rock with deeper rounds could not fail to be greatly increased.

Among other advantages of the use of reasonably shallow holes may be mentioned the fact that such a method allows the holes to be of larger diameter at their farther end, increasing their capacity for explosive and enabling its concentration at the point where it is most needed. This feature makes possible the European practice of employing extremely shallow holes, but it can hardly be denied that much more effective results in blasting might be accomplished by a change in the direction of the cut holes. Then, too, as in America at least, the holes are rarely charged with explosive to their full extent; the mass of rock between the ends of the charges of explosive in the different holes and the free face of the heading (which can be considered as a measure of the amount of resistance to be overcome) is not so great with the shallow holes. This fact or the customary use of relatively heavier charges in shallow holes may explain, perhaps, why in such cases the major part of the rock is usually thrown farther down the tunnel instead of being piled high immediately in front of the new face, with the double advantage of making loading of the rock easier and saving time in getting the drills mounted. It is fairly well established, also, that the rock tends to break into smaller fragments if shallow holes are employed. Again, if deep holes are not employed the same care in starting them exactly at a given point is not required, nor is it necessary to direct them with such great accuracy, although of course the need of connecting the cut holes must not be overlooked.

TABLE I.—DEPTH OF DRILL HOLES USED IN AMERICAN TUNNELS.

Name of tunnel.	Type of cut	Height of heading, ft.	Width of heading, ft.	Av. depth of cut holes, ft.	Av. depth of other holes, ft.	Av. depth of round drilled, ft.	Percentage of width of heading of average round.	Character of rock penetrated.
Buffalo (water)	Wedge	8	15	8	7	7	46.5	Limestone.
Carter	Bottom	7½	5½	9	8	8	*106	Gneiss, granite and porphyry.
Catskill Aqueduct:								
Rondout (siphon)	Wedge	8	14	10	8	8	57	Limestone, sandstone and shale.
Wallkill (siphon)	Wedge	8	14	12	10	10	71.5	Shale.
Moodna (siphon)	Wedge	8	14	10	8	8	57	Sandstone and shale.
Yonkers (siphon)	Wedge	8	14	8	6	6	48	Gneiss.
Central	Wedge	7	5	8	7	7	140	Gneiss.
Ft. William (water)	Bottom	6½	5	6	5	5	*77	Basalt.
Gold Links	Bottom	8	6	6	5	5	*62.5	Granite and gneiss.
Gunnison	Wedge	6	10	7	6	6	60	Altered granite.
Joker (drainage)	Wedge	11	12	10½	9	9	75	
Laramie-Poudre	Wedge	6½	9½	8	7	7	74	Close-grained granite.
Lausanne	Wedge	8	12	8	7	7	58	Conglomerate shale and coal.
Lucania	Wedge	8	8	9	8	8	100	Hard granite.
Marshall-Russell	Pyramid	9	8	10	9	9	112	Granite and gneiss.
Mission	Bottom	7	5	8	7	7	*100	Shale and slate.
Mission†	Pyramid	7	5	8	7	7	140	Sandstone.
Newhouse	Pyramid	8	8	6½	5½	5½	69	Gneiss.
Nisqually	Bottom	11	9½	8	6½	6½	*59	Rhyolite.
Northwest (water)	Wedge	10	13	10	9	9	69	Sedimentary.
Ophelia	Wedge	9	9	7	6	6	67	Granite.
Rawley	Wedge	7	7½	9	8	8	106	Andesite.
Raymond	Wedge	9	9	12	10	10	111	Gneiss and granite.
Roosevelt	Wedge	6	10	7	6	6	60	Hard granite.
Siwatch	Bottom	7½	6	5	5	5	*67	Granite.
Snake Creek	Wedge	6½	9½	6½	5½	6	63	Diabase.
Spiral	Wedge	10	16	12	10	10	63	Limestone.
Stilwell	Wedge	7	7	6½	6	6	86	Conglomerate and andesite.
Strawberry	Wedge	6	8	7	6	6	75	Limestone, sandstone and shale.
Utah metals	Bottom	8	10	6½	6	6	*75	Quartzite.
Yak	Pyramid	7	7	5	4	4	57	Limestone, sandstone, shale and granite.

*The height of the heading, instead of its width, is considered in this ratio when the bottom cut is employed. †Hard ground.

The principal and unavoidable disadvantage in using the shallow hole round, on the other hand, is the fact that in order to obtain the same daily advance a proportionately greater number of drilling attacks must be made. This results in a waste of time in drilling, for it is possible under ordinary circumstances to drill one hole of a given depth more rapidly than it is two holes of the same aggregate footage, because of the time lost in changing to a new position, starting, etc. But even granting that the difference in drilling time (perhaps because it is too small or because in either case the drilling can be completed before the heading can be cleared of debris) is not an appreciable factor, each extra drilling attack required to obtain the same progress causes a corresponding loss of time in loading and blasting the holes, in waiting for the smoke and gases to be removed, in clearing the debris from immediately in front of the face, and in setting up the drills, all of which is ordinarily dead work and can not be avoided. This loss was seriously felt at the Loetschberg Tunnel, because in the endeavor to compensate for it four drills had to be employed in the heading (6x10 ft.), and as a result the holes had to be drilled nearly straight, with disadvantages already described, because otherwise the drills in the center interfered seriously with the operation of those at the side.

On the other hand, if the holes are too deep the angle between the cut holes may be so narrow and the mass of rock in front of the charge of explosive may be so great that it will be impossible for the cuts to break bottom on the first blast, and thus the entire round is spoiled. The usual remedy in such cases is to blast the cuts separately and not to fire the remainder of the round until inspection has shown that the proper depth has been reached by the cut holes. Some delay can not be avoided when this method is employed, even if the holes break to the end, for it is never possible to return to the breast for such inspection immediately after the cuts have been detonated. But if the cut holes fail to break, the delay is greatly increased, because the remaining parts must be cleaned out, reloaded and fired, with an additional delay in waiting for the smoke to clear.

This system was used at one of the Colorado tunnels, which at the time of first examination was being driven through some very tough rock. A round of holes slightly deeper than the average width of the heading was used, and it had given satisfactory results in the somewhat more frangible ground previously penetrated, the round being drilled and blasted in an 8-hour shift without difficulty, but when the harder rock was struck it became necessary to blast the cuts separately, and frequently to reload and shoot them for the second and occasionally for the third time, the cycle being lengthened to about 10 hours, and several times at least 14 hours was needed. If three drilling shifts had been employed at the time, such a condition would have been fatal, but as only two attacks were being made the difference was not so noticeable, though even in this case the cost of the extra explosives required and the overtime wages of the men added a considerable expense to the tunnel work. Shortly after the first examination of this tunnel by the authors, however, the depth of the rounds was reduced to about 75 per cent of the width of the heading.

This made it unnecessary to load and shoot the cuts separately, and instead of getting two 7½-ft. rounds in 20 to 22 hours, by working three 8-hour shifts it was possible to drill and blast four, and sometimes five, 5-ft. rounds per day, thus increasing the daily tunnel progress from 15 to nearly 23 ft. with only a small extra cost for labor. The consumption of explosive, a considerable item with the old system, was also decreased fully 25 per cent, and the total cost of the tunnel per foot was considerably reduced.

The disadvantage of too deep holes was strikingly brought out in the construction of the Laramie-Poudre Tunnel. During the first part of the work a 10-ft. round was drilled in a heading 9½ ft. wide, but the round was later changed to one of 7-ft. depth with much better results. To be more specific, during the seven months from April 1, 1910, to Oct. 31, 1910, at the east end of the tunnel, 3,171 ft. was driven, an average of 453 ft. per month, with a 10-ft. round; but during the next 8 4/5 months, from Nov. 1, 1910, to July 24, 1911, when the tunnel holed through, 4,798 ft. was driven, or an average of 545 ft., with a 7-ft. round. This is an increase of over 20 per cent in spite of the fact that the greater speed was made when the work was at a greater distance from the portal; and, as there was no essential change in the methods or the equipment, or in the character of the rock penetrated, the increase is attributable solely to the use of shallower holes When the 10-ft. holes were employed to obtain an advancement of 8½ to 9 ft. it was unusual to be able to drill and blast more than two rounds in 24 hours, and oftentimes not that many, as the average of 14½ ft. daily testifies; but with the 7-ft. round not only could three attacks be made, advancing on an average of 6.5 ft. per attack, but a comfortable margin of time was left to provide for delays, and under favorable conditions this extra time meant extra footage. Thus, in March, 1911, the American hard-rock record of 653 ft., or over 21 ft. per day, was established. This advantage of being able to complete an entire cycle of operations during a single shift should be given the weight it deserves in the problem. If crews of men could be found who would work as well without rivalry and without special incentive to push the work, it might be perfectly feasible to choose a depth of round that would require 10 or even 12 hours for preparation, but under the present working conditions, where it is necessary to have some accurate measure of the work performed by each crew, a round is required for which the entire cycle can be completed during a single shift, with a sufficient margin of safety to provide for any ordinary delay.

It is, of course, impossible to set any definite standard or guide for the proper depth of hole that will be applicable to all cases. There are too many variables influencing the result. The proper depth must be determined by experiment in each individual case. However, from an extended examination of the results obtained from the methods employed in American practice, from a careful analysis of European practice as outlined in available published accounts, and from a study of all other procurable modern authority, the authors are of the opinion that for the majority of cases the proper depth of drill hole, the one that most equitably balances the advantages and disadvantages inseparable from the problem, is 60 to 80 per cent of the width of the tunnel heading. Table I gives an analysis of American practice in this respect.

The Number of Holes Employed in Driving Heading for Thirty-Six Tunnels.

Determination of the number of holes which secures the best results in driving tunnel heading is affected by too many conditions to permit in any work of precisely following previous experience. Such experience, however, furnishes hints which are of use and there is interest in the tabulation following, which is quoted from Bulletin 57, Bureau of Mines, by D. W. Brunton and J. A. Davis.

TABLE I.—NUMBER OF HOLES USED IN DRIVING TUNNEL HEADINGS IN VARIOUS AMERICAN TUNNELS.

Name of tunnel.	Number of holes.	Character of rock penetrated.	Approximate area of heading. Sq. ft.	Square feet of heading per hole. Sedimentary rocks.	Igneous rocks.
Burleigh	16	Granite and gneiss	42	....	2.6
Buffalo (water)	22	Limestone	120	5.5	....
Carter	10-11	Gneiss, granite and porphyry	41	....	3.7-4.1
Catskill Aqueduct:					
Rondout Siphon	22	Limestone, sandstone and shale	120	5.5	....
Wallkill Siphon	24	Shale	120	5.0	....
Moodna Siphon	24	Sandstone and shale	120	5.0	....
Yonkers Siphon	21	Gneiss	120	....	5.7
Central	18-24	Gneiss	35	....	1.5-1.9
Chipeta	15-19	....	57	....	3.0-3.8
Fort William (water)	14-20	Basalt	35	....	1.7-2.5
Gold Links	12	Gneiss and granite	48	....	4.0
Grand Central sewer	18	Gneiss	40	....	2.2
Gunnison	24	Altered granite	60	....	2.5
Joker	19-21	....	130	6.2-6.9	....
Laramie-Poudre	21-26	Close-grained granite	70	....	2.7-3.3
Lausanne	15-21	Shale, conglomerate and coal	85	1.0-5.6	....
Los Angeles Aqueduct:					
Elizabeth Lake	25	Granite	145	....	5.8
Little Lake division	14-16	Medium hard granite	90	....	5.6-6.4
Grape Vine division	20-21	Hard granite	90	....	4.3-4.5
Lucania	25	Hard granite	65	....	2.6
Marshall-Russell	18-20	Granite and gneiss	72	....	3.6-1.0
Mission	12-14	Shale and slate	37	2.6-3.1	....
Newhouse	19	Gneiss	65	....	3.4
Nisqually	18	Rhyolite	95	....	5.2
Northwest (water)	22	Sedimentary rock	110	5.0	....
Ophelia	20-24	Granite	80	....	3.6-4.0
Rawley	25-27	Andesite	55	....	2.0-2.2
Raymond	14	Gneiss and granite	80	....	5.7
Roosevelt	24-26	Hard granite	60	....	2.3-2.5
Siwatch	12	Granite	45	....	3.7
Snake Creek	16	Diabase	65	....	4.0
Spiral	21	Limestone	175	8.4	....
Stilwell	16	Conglomerate and andesite	50	3.1	....
Strawberry	16-18	Limestone, sandstone and shale	50	2.8-3.1	....
Utah Metals	12-16	Quartzite	80	5.0-6.6	....
Yak	18	Limestone, sandstone, shale and granite	50	2.8	....

WATER WORKS

Method of Constructing a Concrete Dam and Supplementary Core Wall in Permeable Material for the Water Supply of Toulon, France.

Abstracted by Alfred W. Hoffmann from Annales des Ponts et Chaussées for January-February, 1914.

A large dam presenting interesting and somewhat unusual features of design and construction, was recently finished in France. This structure was built for the purpose of supplying the city of Toulon, one of the largest cities in southern France, and the chief port of the French navy on the coast of the Mediterranean, with water, and is remarkable for the preliminary work which led to the adoption of the final plans. Comprehensive geological and hydrographic studies were made to determine the most favorable location for the dam, so as to insure a sufficient supply of pure water, and also to solve the engineering problems involved. As will be shown later, the chief difficulty to be overcome was the danger of considerable losses of water through the subsoil on which the dam was to be erected. These losses, it was feared, might become a danger to the stability of the structure, and every precaution was taken in order to minimize this danger.

GEOLOGY AND HYDROLOGY.

The valley of the Dardennes extending in a north and south direction is dominated, in the north, by a wide plateau formed by fissured limestones (Calcaire urgonien). From the foot of this plateau to a point near an old mill, about five-eighths of a mile away, where the valley turns northwest-southeast, the ravine is formed by various formations of chalk, belonging to different periods, these various layers being named after places and people, not generally known except by engineers familiar with local conditions. It is sufficient to remember that the "Aptian" stratum (named after the small town of Apt) is a chalk rich in marl, this stratum being more impervious than the others in this group. The "Turonian" (named after the city of Tours) and the "Cenomanian" (named after a Celtic tribe) are of similar description as the Aptian, but contain less marl. The "Neocomian" contains much marl and a green hard sandstone and is practically impervious. The "Urgonian" stratum is full of fissures, and more spongy than the others. Near the old mill, a large fold running from east to west bars completely the geological strata forming the valley, and below this fold more ancient formations of the jura age crop out at the surface.

The location of the dam is shown by Fig. 1, and the longitudinal section through the valley, Fig. 2, illustrates very clearly the somewhat complex geological condition of the subsoil above the down creek fold near the old mill. It will be seen that the Neocomian forms an impervious basin. Directly on top of the Neocomian is a layer of variable but always considerable depth, of the same fissured Urgonian which forms the plateau. This stratum is covered by thick layers of the more or less impervious Turonians, Aptians and Cenomanians.

The hydrologic action of this region is easily explained by the geological conditions. When rain falls upon the Urgonian plateau, the water filters down through the cracks and fissures until it is stopped by the impervious Neocomian. It then follows the surface of the Neocomian down to the bottom of the basin in which it slowly rises. If enough rain falls, the water level rises to the sloping surface, in cracks which, in ancient geological epochs, opened up under the pressure of these waters. The water then flows out as springs. As the water rises higher it comes to the surface through similar cracks located above the first one, until finally the pit of the Ragas is filled, and the latter discharges in addition to all the lower cracks or pits. When the rain stops, the water level is lowered, the pits cease to discharge, beginning with the Ragas, and following in inverse order that in which they commenced to discharge. See Fig. 2.

During the rainy season, from the end of September to the end of May, the Dardennes is frequently fed, and carries sufficient water for the supply of the city of Toulon, provided it can be stored for distribution during the dry season, when the river carries no water at all. To store the required quantity of 1,500,000 cu. m. (about 400,000,000 gals.) a dam 35 m. (115 ft.) high would suffice.

To supply water to Toulon before the construction of the dam, the Ragas tunnel, Figs. 1 and 2, had been constructed. This tunnel had been extended to within a few feet of the pit of the Ragas, the top of which is the highest discharge opening of the underground waters. It was connected with the pit by two pipes of 20 cm. (about 8 ins.) diameter which were combined into a pipe of 30 cm. (about 12 ins.) diameter, this latter pipe discharging into the tunnel. The tunnel connected with the surface by an inclined subway 300 m. (about 985 ft.) long, which could be closed by a gate. The 12-in. pipe discharging the water taken from the pit of the Ragas was operated from the foot of this subway. The lower end of the tunnel was closed by a wall which retained the water, and through which extended the 80 cm. (2 ft. 7½ ins.) diameter cement pipe carrying the water to the city.

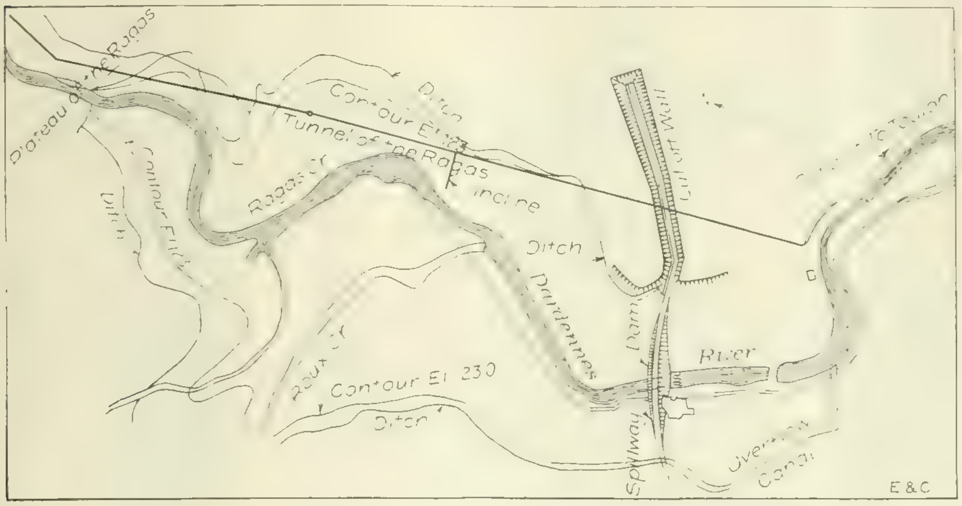


Fig. 1. General Plan of Site and Contiguous Topographical Features of the Dardennes Dam for the Water Supply of Toulon, France.

DAM AND AUXILIARY STRUCTURES.

Upon examination of Fig. 3, which is a section along the bottom of the valley of the Dardennes, it is apparent that the location of the dam could not be varied much either way. By placing the structure higher up in the valley, filtration of the water through the Urgonian under the dam and around the ends would have become a certainty. A location lower down would have meant a considerable increase in the cost of the structure, and the storage of a large quantity of water that would not be required for a long time to come.

The foundation of the dam at the bottom of the ravine, and on the west bank of the river is near the edge of the Neocomian marl which forms the impervious bottom of the basin. On the east bank, the foundation is on an ancient formation of boulders on top of which smaller boulders are found. This condition necessitated a special structure for the prevention of water losses on the east bank and which may be called a cut-off wall.

The center line of the crown of the dam is a circular arc 300 m. (985 ft.) radius; the overall length at the crown is 154.28 m. (505 ft. 10 ins.). The cross section of the dam is shown by Fig. 4, and is computed for a head of water 2 m. (6 ft. 7 ins.) in excess of the normal water level. The maximum soil pressure under the toe of the dam is slightly higher than 130 lbs. per square inch, or 9.35 tons per square foot, which is reasonable on this foundation. The height of the dam is 33.6 m. (110 ft. 2 ins.); the normal head of water is 31.6 m. (103 ft.).

The dam is, in its lower part, traversed by three cast-iron drain pipes of 80 cm. (2 ft. 7½ ins.) diameter which were used during construction to carry off the flood waters. It is estimated that each of the three drain pipes can carry off 10 cu. m. (about 353 cu. ft.) per second, if the water level is at elevation 123.0 (403.5 ft. above datum). In addition, a spillway 70 m. (229 ft. 6 ins.) long with the crown at elevation 123.0 (403.5 ft.) has been provided on the west bank of the river. The spillway was figured to allow 100 cu. m. (3,500 cu. ft.) per second to run off, the water level being assumed raised 80 cm. (2 ft. 7½ ins.) above the crown of the spillway. Since even the highest floods of the Dardennes have never attained 100 cu. m. per second, there is absolutely no danger to the stability of the dam, even if the drain pipes should become clogged. To prevent contamination of the reservoir by surface water a ditch indicated in Fig. 1 was built around its edge. Also the mouth of the inclined gallery to the Ragas tunnel was closed with masonry.

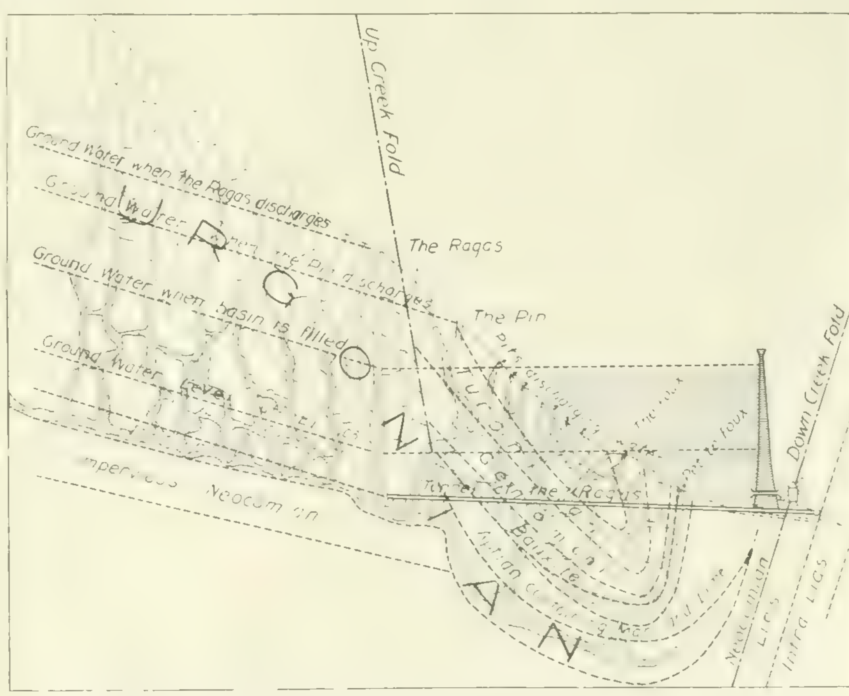


Fig. 2. Vertical Geological Section of the Dardennes Valley Above the Dam.

CUT-OFF WALL DESIGN.

It has been mentioned before that the east bank consists of a layer of boulders on top of an impervious Aptian strata. It was evident that the storage water would leak through the boulders unless some cut-off structure was constructed. While excavating for the foundation of the dam local studies of a very comprehensive and thorough nature were made of various methods for the protection of the east end of the dam. A complete search of the literature on this subject was made and it was found that no dam of the height and magnitude of the dam contemplated had ever been constructed on geological formations of equally questionable character. The only structures resembling this dam, in respect to geological conditions, in France, were some structures erected for the purpose of storing a water reserve for the operation of the Marne-Saone canal. A study of the difficulties encountered in connection with these structures and the methods employed there, resulted in the design of the cut-off wall as used.

Preliminary information was secured by digging test pits A, B and C, located on the east bank as shown on Fig. 5. These pits indicated at various depths impervious Aptian marl covered with boulders. It was apparent that the dam and the cut-off wall would have to rest partly on the Neocomian, at the west end of the dam and at the bottom of the ravine, and partly on the Aptian, at the east end of the structure. Neither the Urgonian nor the Neocomian rocks were found exposed at the surface anywhere; it was evident, therefore, that the Aptian formation, near the surface, leaned against the jura which cropped out at the surface, immediately below the large fold separating the chalk from the jura.

A similar condition was found to exist under the dam proper on the east bank, extending almost to the bottom of the ravine. Nowhere was an exposed surface of Urgonian character found, nor of Neocomian, although it was positively certain that both the latter existed at a certain depth below the surface. It was plain that, on the east bank, the Aptian leaned, at the surface, against the jura, separated from it by a zone of mixed character. This condition was essentially different from the west bank where the succession of Neocomian, Urgonian and Aptian appeared undisturbed at the surface. These conditions are illustrated by Figs. 6 and 7.

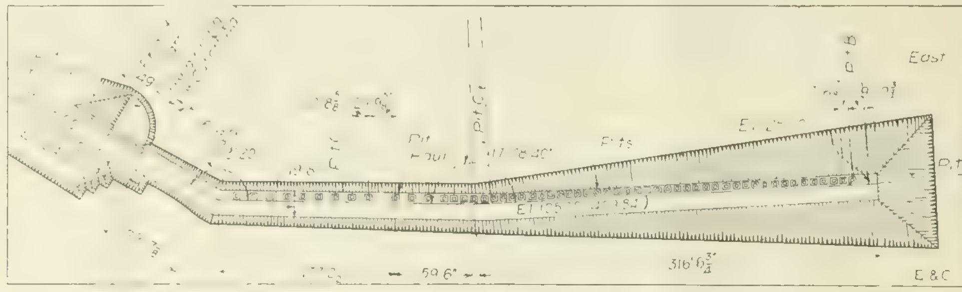


Fig. 5. Plan of Cut-off Wall and Embankment at East End of Dardennes Dam.

The foundation of the dam was to be set, partly on the Urgonian at its juncture with the impervious Neocomian (on the west bank) and partly on the equally impervious Aptian rock (in the ravine and on the east bank). From the standpoint both of stability and watertightness this foundation offered all reasonable guarantees. There remained, however, some doubt regarding the subterranean juncture of the Neocomian with the Aptian. It was feared, and this fear proved to be well founded, that at this juncture, slightly below the dam, a rather thin layer of the porous Urogian occurred separating the two other formations, as shown in Fig. 3. The critical point where the three formations join is indicated by an X.

This condition led to the adoption of certain measures to reduce the loss of storage water through the soil under the dam. Aside from the necessity of taking such precautions, the situation was not considered a dangerous one for the following reasons:

Similar junctures between the Urgonian and the Aptian were plainly discernible on the west bank, and disclosed a somewhat disturbed region of a rather fissured nature. The fissures were, however, not deep, and did not appear to be dangerous. It seemed logical to assume a similar condition to prevail at the subterranean juncture X.

Another, more important reason for not anticipating any serious consequences from this juncture was the fact that, even when the subterranean waters had risen to the top of the pit of the Ragas, i. e., under a pressure considerably higher than the pressure of the storage water would ever be, no pits had ever been found discharging water below the pit of the "Petite Foux," which is near the bottom end of the Turonian, quite a distance above the Aptian and Urgonian. It seemed logical to assume that if these surface junctures remained tight before the water was stored behind the dam, any juncture between Aptian and Urgonian would remain reasonably watertight after the basin was filled. It will be seen, later, that some trouble resulted from this point; it could, however, be overcome by rather simple means.

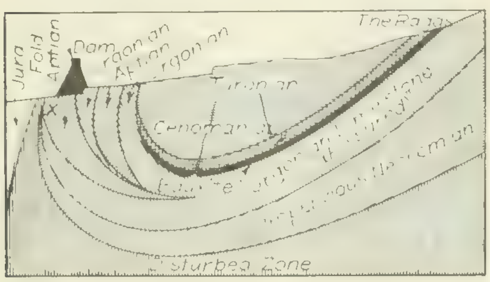


Fig. 3. Vertical Geological Section Along the Bottom of the Dardennes Valley.

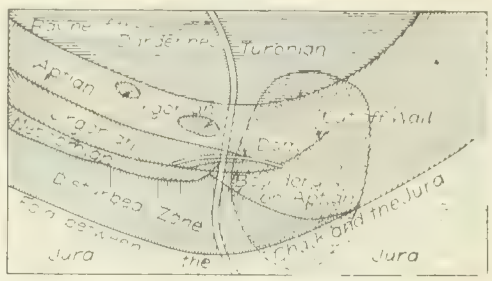


Fig. 6. Sketch Showing the Exposed Areas of the Various Rock Formations Near the Dam Site.

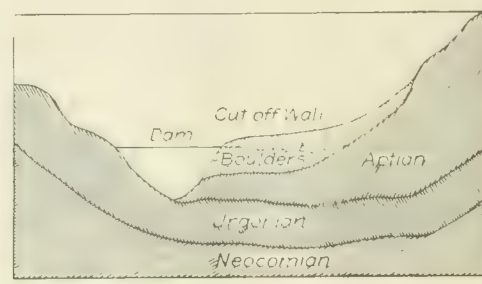


Fig. 7. Geological Section Along the Axis of the Dam and Cut-off Structure.

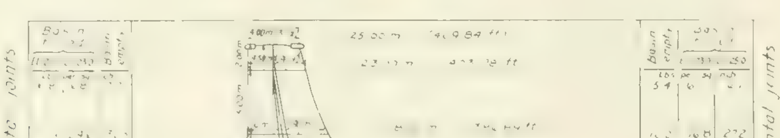


Fig. 4. Stability Diagram of the Dardennes Dam, with Intensities of Pressure at Each Joint.

The cut-off wall on the east bank is shown in Figs. 5 and 8, and is a monolithic structure of compressed hydraulic (maritime) lime concrete, 174.725 m. (about 573 ft.) long. The width is uniform, and is 2.10 m. (6 ft. 10½ ins.). It is built on a foundation of impervious Aptian rock, and the crown, which is at elevation 123.0 (403.5 ft.) at the juncture with the dam, is lowered 25 cm. (about 10 ins.) every 50 m. (about 164 ft).

For the drainage of water penetrating through the masonry of the cut-off wall 62 vertical pits of 75 cm. (2 ft. 5½ ins.) diameter were provided in the structure, as shown in Fig. 8. These pits are connected at the bottom by a cast iron pipe of 15 cm. (about 6 ins.) diameter which drains into a central pit, called pit "Paul," after the contractor who built it. Each pit in the cut-off wall is topped by a concrete funnel and closed by a steel sheet cover at elevation 125.0 (410.0 ft.).

The adopted design of the cut-off wall which is one of the most important parts of the whole structure was the result of extraordinarily careful and comprehensive study. Preliminary designs were made, for a dam of clayey materials, and for a cellular dam of reinforced concrete, and the merits of these alternatives were very thoroughly weighed before the final decision was reached. The main argument against an impervious earth or clay dam was the difficulty of locating leaks and making repairs. A cellular structure of reinforced concrete did not promise

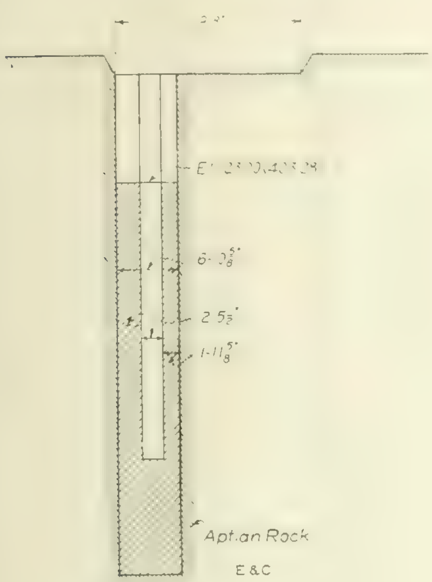


Fig. 8. Vertical Section of Cut-off Wall Showing Inspection Pit.

good results, on account of the difficulties of driving sheet piling to a great depth through a soil consisting of boulders, and the cost of cofferdams and other auxiliary work was considered prohibitive. Besides, a cellular structure with walls 4 to 5 ins. thick did not seem to insure perfect watertightness under a head of water of more than 67 ft. In making the final decision in favor of the plain concrete wall, some weight was also given to the fact that the only structure in France where similar difficulties were met with, and also another structure in Scotland, were treated in a similar way.

The high cost of Portland cement delivered on the ground led the engineers in charge of this work to consider a substitute. Such a substitute was found in the so-called compressed maritime lime, a greatly improved hydraulic lime. This material was to be had, at a reasonable price, in uniform quality. It may be mentioned here that the hydraulic limes of southern France are of exceptionally fine quality, and are generally used near the Mediterranean coast for high grade work. This writer, when in charge of a government contract in Barcelona, Spain, a number of years ago, used hydraulic lime from southern France almost exclusively, and with the very best results. It is, therefore, not surprising that this excellent material was taken into consideration when this question came up. Before an opinion was formed regarding the water proofing qualities of the plain hydraulic lime concrete, extensive tests were made with different mixtures, and under the same conditions as were to be expected in the finished structure.

In the lower part of a cast-iron cylinder of 40 cm. (1 ft. 3¾ ins.) diameter a test block of concrete was cast 75 cm. (2 ft. 5½ ins.) high, on top of which any desired water pressure could be applied. The first test was made with a mortar consisting of 350 kg. (about 770 lbs.) maritime lime per cubic meter (about 1.3 cu. yd.) of sand from the sea, five days after the mortar was deposited in the cylinder. The pressure applied was equal to a head of 20 m. (about 67 ft.) of water. It took about two days for the water to penetrate the cylinder of mortar. After the water had worked itself through the mortar, the leakage through the latter became a maximum very rapidly; this maximum being about 140 c.c. (.035 gal.) per 24 hours. Then the leakage diminished steadily, on account of the pores in the mortar being filled by the mineral substances contained in the water, until, after one month, it practically stopped.

This test had satisfactorily established the fact that masonry of a mortar, or concrete, of the above proportions, mixed and deposited with the same precautions, as were used for the test specimen, would practically stop all leakage. Another test was then made with a mortar consisting of 300 kg. (about 660 lbs.) of maritime lime per cubic meter (about 1.3 cu. yds.) of sand made of finely crushed lime stone such as it was contemplated to use in the cut-off wall. The same water pressure was applied, and the water was forced through the mortar the first day. The losses reached a maximum of 2.95 liters (0.74 gal.) per 24 hours very rapidly, and were reduced to 1.20 liters (0.3 gal.) per 24 hours, at the end of ten days.

A third test was made with a five days' old mortar mixed of 350 kg. (770 lbs.) of maritime lime with 1 cu. m. of the same aggregate as used for the second test, and subjected to the same water pressure. The water leaked through the sample the second day, the maximum loss, which was attained very rapidly, did not exceed 1.75 liters (.44 gal.) per 24 hours, and was reduced to .34 liter (.08 gal.) per 24 hours, at the end of one month.

The proportion used for the second test was considered to satisfy all reasonable requirements concerning watertightness of the cut-off wall, and was, therefore, selected for this work. The tests demonstrated plainly that the pores of the structure were rapidly closed by the waters of the Ragas, to the action of which the structure would be subjected when completed.

CONSTRUCTION PLANT AND METHODS.

The water company specified the use of Urgonian lime stone for the ashlar masonry of the dam as well as for the fabrication of the fine aggregate to be used in place of sand. The quarry was located on the west bank, in the ravine of the Fierraquet. In order to facilitate the handling and moving of the stone, the contractor opened the quarry at a somewhat lower elevation than that recommended by the water company. The top layers of rock were of fine quality; later, however, clay pockets were encountered which rendered the production of stone rather difficult, especially during the rainy seasons.

A narrow gage track 500 m. (about 1,648 ft.) long was laid from the quarry to the mechanical plant, on a down grade of 1.92 per cent, the plant being installed on an elevation of 128.40 (421.0 ft.). The elevation of the track at the quarry was 133 (436.0 ft.). The rolling stock consisted of 35 cars of 500 to 800 liters (.65 to 1.05 cu. yds.) capacity, and of 15 platform trucks. On account of the light grade, one horse was sufficient to pull a train of 8 to 12 loaded cars down grade from the quarry to the plant, and the empties upgrade again. Two men operated the brakes.

The stones were lowered to the foot of the dam, on an inclined track operated by a steel cable of about ⅞ in. diameter. The grade of the incline was 50 per cent, the length was 100 m. (about 328 ft.). The incline extended between points *A* and *B*, as shown in Fig. 9, which is a sketch of the general arrangement of the plant. The loaded cars were, at point *A*, placed on platforms to which the ends of the cables which passed over a pulley were fastened; these platforms moved up and down the incline, on the principle of an endless chain. At the bottom of the incline, at point *B*, the load was delivered to cars running on a track laid upon the dam. These cars carried the load of stones to the place where they were to be set. As the height of the dam rose, the incline and also the length of the cable were shortened accordingly. The maximum load carried down was about one ton.

This arrangement for conveying the stone to the dam gave full satisfaction until the dam was completed.

The cars loaded with the stones which were intended to be crushed for fine aggregates, dumped their charge into the crusher which was installed near and at the elevation of the track coming from the quarry. The resulting pieces of crushed stone, not exceeding the size of the fist, were conveyed to one of the two grinding mills which ground them to a fine sand. The finished product dropped into a silo. The dust was sucked, by fans, into the dust chambers.

An elevator lifted the sand from the bottom of the silo to a height of about 9 m. (29 ft. 6 ins.), into a drum where all grains of larger size than .01 mm. (.0039 in.) were separated from the fine sand. The latter was conveyed to the mixers, or to the storage room, at the south end of the building. The former were returned to the grinding mills.

The average output of the crushing and grinding plant was about 3 cu. m. (3.9 cu. yds.) of sand, or fine aggregate, per hour. The hammers of the grinding mills had to be replaced after from 50 to 60 days of work. The fine dust caused extraordinarily heavy wear on the transmission belts.

The cement and lime were hauled in sacks from the yards at Toulon to point *C*, whence they were lowered to the cement and lime stores by means of two wooden chutes. The capacity of the stores was 30,000 sacks. These materials were conveyed to the mixers over a small bridge across the service tracks.

The water required for the mortar was

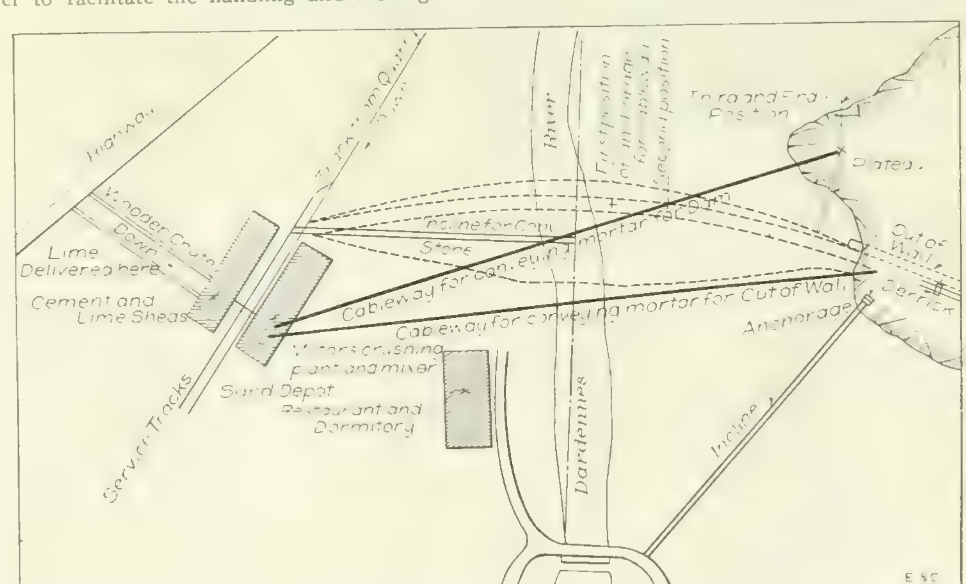


Fig. 9. Arrangement of Contractor's Plant for the Construction of the Dardennes Dam.

furnished by the water company, from the old pipe line which ran to the city, at the elevation 91.0 (298.5 ft.), and at a distance of about 300 m. (about 985 ft.) from the nearest point of the plant. A pump which was run by a 4-HP. oil motor lifted the water from out of the pipe, through a cast-iron pipe of 8 cm. (about 3 ins.) diameter, into three sheet metal tanks, near the mixers, at an elevation 128.0 (428.0 ft.), of a total capacity of 30 cu. m. (about 8,000 gals.). The maximum amount of water required for all purposes was about 15 cu. m. (4,000 gals.) per day. One of the three tanks was used to supply a pipe running along the top of the dam, for flushing the same.

The mortar for the dam was prepared in two mixers where the fine aggregate and the cement were mixed dry, and the supply of water was regulated to be uniform. The output of each of the mixers was about 30 cu. m. (about 39.3 cu. yds.) per day of 10 hours. The mortar dropped directly into the buckets of the cableway, which consisted of two steel cables of about ⅞ in. diameter and of 205 m. (about 672 ft.) length. The buckets had a capacity of 350 liters (1.25 cu. ft.), or about ¼ ton of mortar.

The anchorage of the cableway has been successively moved from the first to the second and then to the third and final position, as indicated in Fig. 9. The aerial cableway was operated by gravity, and controlled by brakes, the average speed of the buckets was about 2 m. (6 ft. 6½ ins.) per second. The buckets discharged into small cars running on the track on the dam itself.

For the construction of the cut-off wall, the above described plant was supplemented by a crushing plant consisting of a crusher and a grinding mill, also by a mixer, and another aerial cableway of similar description as the above. In addition to the plant on the west bank of the river, an inclined plane *M-N* (see Fig. 9) was installed on the east bank, on which all fittings and machinery required for the construction of the cut-off wall were hoisted up to the plateau, in a truck connected with a steel cable operated by hand. A self-propelling steam derrick was operated on a special track along the top of the cut-off wall.

The power required for the plant on the west bank, i. e., for the crushers, mixers and cableways, was generated by 3 steam engines, one of 40 HP. and two of 25 HP. each.

The employes and a large number of the workmen found board and lodgings in the near village of Revest. Since the number of workmen rose, at times to 300, however, the contractors considered it necessary to build a restaurant and dormitory with 60 beds (*K* on Fig. 9) immediately below the dam.

The excavation did not offer any unusual difficulties except in the big trench at the east end of the dam where the excavation had to be made about 24 m. (about 79 ft.) deep. There the heavy rains in the fall, 1910, and spring, 1911, had caused considerable movement of the rock at the sides of the trench.

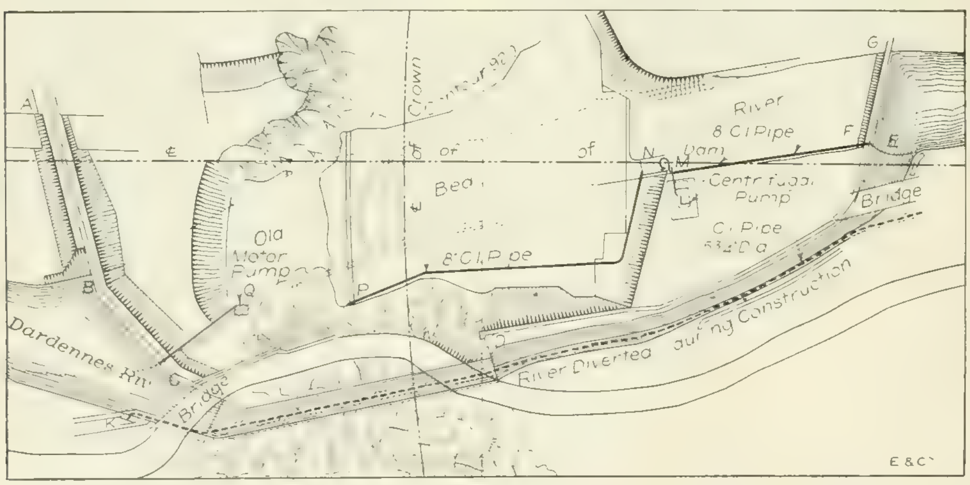


Fig. 10. Plan Showing Works for the Diversion of the River During the Construction of the Dam.

All excavated material, about 26,000 cu. m. (34,000 cu. yds.) was hauled away by horse teams, a distance of about 500 m. (about ⅓ mile).

The flow of the Dardennes River being very irregular and of a somewhat torrential character, it was necessary to divert the bed of the river, as shown in Fig. 10, in order to avoid damage to the structure during construction. The masonry cofferdams, *A-B-C* and *F-G*, and the tunnel, *C-D*, through the rock are the most interesting parts of this work. At *K*, a cast-iron pipe used for irrigation purposes which had to be maintained during construction, entered the canal. The work on the canal, which was started in July, 1909, was interrupted by two heavy floods in the fall of 1909. It was completed by the end of December, 1909. This canal was closed by cement concrete masonry in August and September, 1911, when the three pipes of 80 cm. (2 ft. 7½ ins.) diameter were ready to carry off the water.

The excavation of the dam was drained by a steam pump installed at point *L* in Fig. 10. The water was pumped into the sump *N* at elevation 87.0 (285.5 ft.), which was drained by an 8-in. drain pipe. In order to permit the masonry for the dam foundation to be placed, entirely without disturbance by ground water, the latter was collected in the cast iron pipe, *P-N*, on the west bank, and then discharged into the sump *N*.

Later, after the dam was built up to a height which made it impossible to maintain the drain pipe *P-N*, the ground water was collected in a sump, and then pumped by a gasoline motor pump *Q* into the canal.

This drainage had to be maintained until July, 1910, when the dam was completed to elevation 93.0 (305.0 ft.), i. e., when one of the three 2 ft. 7½ ins. drain pipes was available for this purpose.

The diversion of the roads affected by the construction of the dam presented no unusual features, and will, therefore, not be discussed here.

GROUTING PERVIOUS ROCK AND EXCAVATING CUT-OFF WALL.

The bottom rock was prepared for the foundation of the dam by filling the pores with cement, under pressure. This preparatory work has given excellent results, and is to be recommended in all cases where a high degree of imperviousness is required, and where the natural foundation is not strictly impervious, like all chalks.

With a view to prevent the water of the pit of Ragas, which was used to supply the city of Toulon, to be mixed with the water to be retained in the basin, a 50-cm. (about 1 ft. 8 in.) diameter drain pipe was installed. This work presented considerable difficulties because the bottom of the pit Ragas was accessible only by the steep incline which has been described before, in connection with the description of the water supply of Toulon before the dam was built. The canalization of the Ragas had to be done very rapidly, moreover, to shorten the period during which the city depended on other sources of supply.

In order to prevent water losses through the Urgonian forming the subterranean bed of the Ragas, it was necessary to make some provision for waterproofing by the construction of concrete walls, placed where required in the Urgonian. A cement concrete wall, 10 m. (32 ft. 8 ins.) thick was built 193 m. (632 ft. 8 ins.) above the bottom edge of the Urgonian. But when the water in this soil and more especially when the water behind the dam was allowed to rise for the first time, it was found that this masonry wall did not give sufficient protection, not even after the critical part of the soil was well filled with cement under pressure.

It was therefore decided to build a second wall, 5.75 m. (18 ft. 10½ ins.) thick, 73 m. (239 ft. 3 ins.) above the bottom edge of the Urgonian, i. e., near the pit "Paul" in the cut-off wall, see Fig. 5. This second wall was found to be perfectly impervious, even after the water had risen to the top of the dam.

Before the narrow trench about 20 m. (65 ft. 7 ins.) deep for the cut-off wall could be commenced to be excavated, through the layer of loose boulders, it was necessary to grade a strip 6 m. (19 ft. 8 ins.) wide, at elevation 125.0 m. (410.0 ft.) following the alignment of the cut-off wall. The excavation of the trench was then made in two sections to permit the timber which was used for the first section to be used again for the second section. The first 4 m. (about 13 ft.) from the top were shoveled to the surface, by means of raised platforms. Below this depth, the excavated material was lifted to the surface by the steam derrick operated on the track running alongside the wall.

The water encountered at elevation 114.0 (374.0 ft.) was at first carried off by a ditch in the bottom of the trench, toward the river below the structure. But as it was apparent that this simple method of draining the trench could not be used till the work was finished, a vertical pit was constructed, at the intersection of a vertical plane through the subterranean flow of the Ragas and a vertical plane through the center line of the cut-off wall. This pit is the pit "Paul" repeatedly referred to which discharged, by gravity, the water collecting in the trench, into the subterranean flow of the Ragas. The construction of the pit "Paul" was achieved under very serious difficulties. The work could not be done by digging down from the top, but by starting from the subterranean bed of the Ragas and working upwards through 5 m. (16 ft. 4½ ins.) of the Aptian rock. The shaft was 1 m. (3 ft. 3 ins.) diameter. The pit was drilled down from the surface to the top of the Aptian.

After the completion of the pit "Paul" in October, 1911, the excavation for the first part of the cut-off wall was completed without difficulty, but very slowly, on account of the loose nature of the soil, the presence of large boulders, and the presence of a great earth pressure which made tight joints for the shoring imperative and necessitated frequent renewal of broken pieces of timber. After the rock surface was reached throughout the length of the first section of the wall, a careful examination of the same was made, and the excavation deepened wherever the surface rock was not of sound quality.

The pit "Paul" had originally been drilled down to within about 16 ft. 4½ ins. from the bottom. It was then widened and the shaft lined with concrete, for which 600 kg. (1,320 lbs.) of compressed maritime lime were mixed with 1 cu. m. (1.3 cu. yds.) of sand. The inside diameter of the pit was 75 cm. (2 ft. 5½ ins.). Concrete of the same rich mix was deposited as a first layer of variable thickness on the rock. as a footing for the wall. On this first layer the longitudinal drain pipe was laid. The concrete above the first layer was mixed 300 kg. (660 lbs.) to one cu. m. (1.3 cu. yds.) of sand, and the concreting could then proceed without any particular difficulty aside from the close spacing and frequent breaking of shores and props.

The enormous earth pressure required slight changes in the original design of the shoring, since it was not permissible to leave any timber in the finished wall. Lead pipes were laid horizontally through every pit in the cut-off wall, these pipes connecting with both faces of the wall. By means of these pipes it was possible to inject cement into the pores of the masonry at the joints in the forms where the concrete was not so dense as desired. These injections had the best results as will be seen later.

While the masonry for the first section of the wall was being placed, the excavation was prepared for the second section. This work proceeded very rapidly as all difficulties had been removed before. The masonry for the cut-off wall was completed in October 1911, except a few feet adjoining the dam which could not be finished because the work on the dam was not sufficiently far advanced. The dam itself, and the juncture with the cut-off wall were completed in March, 1912. The remaining work of raising the pits to elevation 125.0 (410.0 ft.), and the grading of the fill were completed on Aug. 5, 1912.

FILLING WITH WATER AND PROVISIONS AGAINST LOSS OF WATER.

The filling of the basin created by the dam had, naturally, to be made very cautiously and gradually. Observations of the losses of water, and their source, were constantly made. There was no danger, in this case, of any losses through the dam itself, but of leakage through the geological formations under a part of the dam, and at the wings, especially the plateau of loose boulders on the east bank. Unfortunately, the testing of this work for its imperviousness had to be spread over a long period on account of the long interval between rainy seasons.

The first attempt of filling was made in March, April, May and June, 1911. The dam was completed to elevation 110.0 (361.0 ft.), the waterproof coat had reached elevation 107.5 (353.0 ft.), the trench for the cut-off wall was excavated, but no concrete deposited, except at the pit "Paul," which was concreted to elevation 100.0 (328.0 ft.). The state of the work limited the height of the water level at the first filling to 107.0 (351.0 ft.).

After a rather heavy rain on March 15, 1911, and the following days, the water rose at once to elevation 100.0 (328.0 ft.), then slowly, at the rate of over 3 ft. a day, to elevation 107.0 (351.0 ft.). As the dam progressed, the water level was permitted to rise to elevation 110.5 (362.5 ft.), at which level it was maintained for about one month.

The results of the first filling were generally satisfactory, and carried out the assumption that, at a certain elevation, the losses were diminishing all the time. The sum of the losses through the soil under the dam attained a maximum of 2.8 liters (.7 gal.) per second when the level 110.5 (362.5 ft.) was reached, and diminished to 2.2 liters (.55 gal.) per second after a month, the water level remaining the same. The sum of the losses through the ground, including filtrations through the cut-off wall, attained a maximum of 1.5 liters (.37 gal.) per second, which fell to .65 liters (.16 gal.) after one month, and the leakage across the trench for the cut-off wall and through the masonry for the pit "Paul" was reduced from a maximum of .25 liters (.06 gal.) per second to .1 liter (.025 gal.) per second after one month. Since the work on the cut-off wall was not yet sufficiently far advanced at the time the test was made, to draw any final conclusions concerning the watertightness of the structure, it was preferred to wait for another test before starting any waterproofing work.

A second filling was decided upon after a heavy rain which occurred in March, 1912. The dam was finished, except the crown, the waterproofing had reached elevation 123.0 (403.5 ft.), and the cut-off wall was completed. It was, therefore, possible to push this test as far as the amount of the flood on one hand and the resistance of the structure on the other hand would permit.

Up to the level 114.0 (374.0 ft.) the losses were just about acceptable, amounting to 5 liters (1.25 gals.) per second under the dam, and a similar loss through the ground and the cut-off wall. But, beginning at this level, the losses commenced to increase out of all proportion. The filling was stopped, and the drains opened on March 7, when the water level was 116.5 (382.5 ft.). At this level the losses around and under the dam were 10 liters (2.5 gals.) per second, the sum of the losses in the tunnel, and through the cut-off wall which it was impossible to measure separately, amounted to more than 30 liters (7.5 gals.) per second.

After the basin was drained again, a series of injections of cement under pressure was made into the soil under the dam, by means of extensive borings. Of greater importance, however, was the work on the cut-off wall which deserves a short discussion.

The results of the second filling were plotted graphically, and from these diagrams, not reproduced here, it was evident that the horizontal pipe connecting all vertical pits at the bottom was, during the test, not sufficient to carry off all the water that leaked through the wall. This was proved by the fact that the water level in the pits was raised above the top of the pipe. A close study led to the following conclusions:

Between pits 10 and 1 the water flowing through the pipe connecting the pits must have been about 27 to 30 liters (6.75 to 7.5 gals.) per second, calculated according to Darcy's formula. As this flow is about the same near pit 9 as it is near pit 1, it appears that in this zone no leakage of any importance occurred.

The water level in pit 1 was 112.6. Under this head the flow into the pit "Paul" should have been about 75 liters (18.75 gals.) per second. It was, however, only 30 liters (7.5 gals.) maximum. It must be concluded that the longitudinal pipe was badly clogged up in this section.

The fact that the water level in pit 10 was 114.10 (374.5 ft.), which was noticeably lower than in the pits 9 and 11, together with the fact that the water flowing from pit 10 toward the pit "Paul" amounted to about 27 to 30 liters (6.75 to 7.5 gals.) per second, demonstrated that the source of the very large leakage was to be looked for in pit 10.

The relative heights of the water levels in the pits located west of pit 10 seemed to prove that, under the pressure accidentally produced by the waters rising in the pits, leakage occurred through the cut-off wall.

These conclusions drawn from diagrams were found to be correct, after a thorough local investigation. The longitudinal pipe was found to be clogged up in places by deposits of lime which had set, and in pit 10 it had been forgotten to close the lead pipe mentioned before which had been installed to facilitate future injections of cement if such should be required.

After the various faults were located and corrected, and extensive injections of concrete under pressure made, a third filling of the basin was decided upon, which took place in June, 1912. This test was made slowly and cautiously, and the water was permitted to rise to elevation 120.5 (395.5 ft.).

It was considered wise not to exceed this level at that time. The amount of water stored behind the dam was sufficient to supply the city during the summer; on the other hand, the losses although greatly reduced compared with the second filling, were still of sufficient magnitude to make additional repairs imperative. The maximum loss of water through the dam was about 11.5 liters (2.87 gals.) per second, and the maximum loss through the cut-off wall and tunnel combined was 4.9 liters (1.22 gals.) per second. The basin remained partly filled until fall, when the heavy rains could be relied upon to furnish suffiicent water for the supply of the city, without aid from the storage water.

Beginning on June 11, when the level 120.5 (395.5 ft.) was reached, it began to fall, very slowly at first, but more and more rapidly later. On July 1, the level was still 119.66 (392.5 ft.); on August 1 it had fallen to 115.57 (379.0 ft.); on September 1 it had fallen to 113.67 (372.5 ft), and on September 28 the level was as low as 108.73 (356.5 ft.). On that day the first rainfall occurred, the basin was emptied, and the supplementary work of waterproofing commenced, at the foot of the dam.

The conclusions drawn from observation of the losses of water during the time the basin was filled were mainly that the losses were diminishing, not merely as a function of the reduced head, but on account of the filling of the pores by the minerals contained in the water. It was also concluded that the losses through the cut-off wall were so low as to be of no concern at all, while the losses through the soil below the dam were still of sufficient importance to require special attention.

A large number of injections of cement were made through drilled holes into the soil, most of them on the west bank. The holes were made 13 to 27 ft. deep and filled with cement under pressure. Some required large quantities of cement; thus it was proved that large cavities or extensive cracks existed in the stone. This work, begun in August, 1912, was completed Jan. 1, 1913.

The final filling of the basin was commenced April 1, 1913, after a heavy rain. Extraordinarily heavy rains followed on April 2 and 3, which permitted the water to rise so rapidly that not only the elevation 123.0 (403.5 ft.) was reached during the night of April 3 to April 4, but in addition, 100 cu. m. (131.0 cu. yds.) per second were being carried away over the spillway and through the overflow canal. The elevation of the water retained by the dam rose to a maximum of 124.0 (406.5 ft.), which was a very severe test for the structure, in view of the suddenness of the flood.

The results of this test were entirely satisfactory. Some leaks were observed, but they were of no importance, and were later repaired. Exact measurements were made to determine any deformations; it was, however, impossible to detect any.

The water losses were, after these repairs were completed, under the dam 1.0 liter (.25 gal.) per second, through the cut-off wall 4.33 liters (1.08 gals.) per second. On May 27, 1913, the day when the water level commenced to fall, the losses were 0.73 liter (.18 gal.) per second under the dam, and 3.4 liters (0.85 gal.) per second through the cut-off wall. These losses are insignificant, and of no danger to the structure and they have been diminishing from day to day. Besides, another series of injections of cement has in the meantime been carried out which has, no doubt, reduced the water losses even more.

The contract price for the work here described was $316,000. Including all extra expenses not included in the contract, like the injections of cement, property damage, etc., the complete cost of the structure was $420,000.

ACKNOWLEDGMENT.

The translator wishes to acknowledge his indebtedness to an article by Mr. Boutan, chief engineer, and Mr. Veilhan and Mr. Mercier, civil engineers, published in the first issue, 1914, of the "Annales des Ponts et Chaussees," Paris, France, for the information on which the present article is based.

The design was worked out by the authors of the article, who were also in charge of the construction. They were co-operating with Mr. Bernier, chief engineer of the Toulon Water Co. The designers obtained information on the geological formation of the region from published records. In addition to those, two geologists were called into consultation. The sanitary and hygienic conditions affecting this work were investigated and reported upon by two sanitary experts.

Selection and Use of Concreting Materials at McKeesport Water Softening and Filtration Works.

The water softening and filtration plant at McKeesport, Pa., was placed in operation in October, 1908. An article on the operation of the plant was published in this journal of May 11, 1910. The present notes on the selection and use of concreting materials for this plant are taken from a very complete description of the design, construction and operation of the plant as prepared by E. C. Trax, chemical engineer of the plant, for publication in the 1913 annual report of the Board of Water and Lighting Commissioners, Mr. D. M. White Superintendent. These notes are of interest to all who have occasion to seek watertightness in reinforced concrete construction.

The fundamental principles underlying the design of the reinforced concrete were given careful consideration from both the metal and the concrete standpoint.

Mild steel was adopted, first, because of the possible vibrations due to the operating machinery in the floor system; second, because of the probability of securing a steel of more even structure; third, because low carbon steel is more easily bent, facilitating construction.

The selection of the aggregates for the concrete was also given careful consideration, and a practice was adopted a little out of the customary. The gravel and sand used for the concrete came from the Allegheny River, and it was necessary to screen it before it was used. We required that the gravel range in size from ¼ to 2½ in. with not more than half of it to pass a 1-in. mesh, and thus insured a more uniform mixture.

Much of the concrete used in the construction was for 6-in. floor slabs, 8-in. tank walls, and for work in general where the percentage of steel was 1.2. There was some criticism from time to time because of the adoption of the large stone, but the mixture took care of the large stone satisfactorily, without voids, due to the proper placing of the concrete around the steel reinforcement. This was watched very closely, and upon the removal of forms, the percentage of honeycomb was less than in other construction inspected from time to time where the maximum size of stone was 1 in. As this practice was more or less unusual, careful inspections were made from time to time to see that the finished concrete was satisfactory. To test for internal voids, several openings were cut through the walls, and in no instance were any found.

Satisfying ourselves in respect to the freedom from honeycombing by using larger stones, we also satisfied ourselves that this proportioning of the aggregates was the best for all practical purposes and convenience of construction. As the general practice recommends a maximum size stone averaging from 1 in. to 1½ in., tests were therefore made to determine also whether or not the concrete was the best mixture. These tests showed that with stone of the maximum size of 2¼ in. to 2½ in., a stronger concrete was procured than with stone of a maximum diameter of 1 in. and 1½ in. Mechanical analyses were made of the sand and gravel, and from time to time we had to change the mixture due to the slight changes in the proportions of the constituent parts of the gravel. These tests corroborated the findings of W. B. Fuller, published in the Transactions of the Am. Soc. of C. E., in which he shows that for an equal strength of concrete with 1 in. gravel it required 17 per cent more cement than with a maximum size of 2½ in., and that a concrete with a maximum size of 2½ in. required 33 per cent less cement than concrete with 1 in. maximum size gravel.

As an example of the success of this method, one of the main circular walls was 24 ft. in height and 13 ins. thick, reinforced with ¾ in. rods both horizontal and vertical. An approximate 1:3:5 mixture was used. It was deposited from the top and fell the entire height. Upon the removing of the forms, no honeycomb surfaces were found. The concrete was mixed quite wet, using as much water as it would hold and great care was taken not to give it an excessive tamping.

Proper consideration was given to the matter of building forms. To build the forms part way up and deposit the concrete at different intervals in the height, required a costly building and changing of runways. It also made it absolutely necessary to have a horizontal joint in the concrete whenever the runway was changed. Horizontal joints should be avoided wherever possible; when vertical joints are used keys should be placed in the forms. In no instance did the vertical joints give any trouble whatever. In the few instances where horizontal joints were necessary, great care was taken to wash the old concrete surface thoroughly clean before the placement of fresh concrete. Even after taking this precaution, however, some leakage was found in the horizontal joints. It was finally decided in all instances to build the forms continuous from top to bottom and to place all concrete in the section of the wall so that a complete vertical section could be finished in one day's work. By taking great care in the proper placing and tamping of the concrete, we obtained a wall of quite even texture which gave good results when subjected to the pressure even of clear water.

The method of tamping the concrete was a little contrary to ordinary practice. At first the spade method was used by inserting a sheet of steel about ¼ in. thick between the concrete and the form. This gave a finished surface which did not have any honeycomb, but invariably gave a surface showing a sand deposit which could readily be rubbed off. This form of tamping was finally abandoned and wooden tampers were used with an iron shoe on the end 1x6 ins., and instead of inserting this tamper between the concrete and forms the concrete was pushed against the forms. This prevented the separation of the cement from the sand, and upon the removal of the forms the concrete gave a harder, more metallic appearance. In concluding to construct the forms so as to permit the depositing of concrete from top to bottom in one operation, a question presented itself, whether or not it would be better to build the forms in one piece or in sections. As before stated, the section scheme was finally adopted. The sections averaged 6 ft. high and 8 ft. long. The ribs were cut off to the radii, and the lagging placed on always in a vertical direction; in a few instances the lagging was placed on horizontally or bent to the curvature of the form, but the results were quite unsatisfactory; the tendency to become straight pulled the section out of its true curvature.

The concrete throughout the structure varies in classification and includes groined and plain arches, retaining walls, tanks, cantilever constructions, columns, beams, and floor slabs.

An Important Supreme Court Decision on Water Rights.

In this case on appeal of the San Joaquin & Kings River Canal & Irrigation Co. v. the County of Stanislaus, Calif., the Supreme Court has rendered an important decision which we reprint here and on which we comment editorially in another column:

This is a bill to restrain the enforcement of orders passed by the Boards of Supervisors of the three defendant counties, Stanislaus, Fresno and Merced, establishing water rates to be charged by the plaintiff, the appellant; the ground of the bill being that the orders deprive the plaintiff of its property without due process of law. By a statute of March 12, 1885, the boards are authorized to fix these rates for their several counties, but so that the returns to the parties furnishing the water shall be not less than 6 per cent upon the value of the 'canals, ditches, flumes, chutes, and all other property actually used and useful to the appropriation and furnishing of such water.' The rates when fixed are binding for one year and until established anew or abrogated. The bill concerns rates fixed in 1907, and the question before the court has been narrowed to a single issue. If the plaintiff is entitled to 6 per cent upon its tangible property alone it is agreed that the orders must stand. But if the plaintiff has water rights that are to be taken into account, the rates fixed will fall short of giving it what it is entitled to and must be set aside. The Circuit Court dismissed the bill, 191 Fed. Rep. 875, and on this appeal figures are immaterial, the only question being whether the principle adopted is right.

It was suggested to be sure at the argument that it does not appear that the plaintiff offered any evidence as to water rights at the hearing before the supervisors, and therefore that it ought not to be allowed to complain now that nothing was allowed for them. But this evidently is an afterthought. In general, a party may wait until a law is passed or regulation is made and then insist upon his constitutional rights. Prentis v. Atlantic Coast Line Co., 211 U. S. 210, 227, 229. This we understand to be the view of the California courts as to these very boards. Spring Valley Water Works v. San Francisco, 82 Cal. 286, 315. San Diego Water Co. v. San Diego, 118 Cal. 558, 564. Moreover as the defendants contend that the plaintiff is entitled to no compensation for water rights, to offer evidence would have been an idle form.

It is not disputed that the plaintiff has a right as against riparian proprietors to withdraw the water that it distributes through its canals. Whether the right was paid for, as the plaintiff says, or not, it has been confirmed by prescription and is now beyond attack. It is not disputed either that if the plaintiff were the owner of riparian lands to which its water was distributed it would have a property in the water that could not be taken without compensation. But it is said that as the plaintiff appropriates this water to distribution and sale it thereby dedicates it to public use under California law and so loses its private right in the same. It appears to us that when the cases cited for this proposition are pressed to the conclusion reached in the present case they are misapplied. No doubt it is true that such an appropriation and use of the water entitles those within reach of it to demand the use of a reasonable share on payment. It well may be true that if the waters were taken for a superior use by eminent domain those whose lands were irrigated would be compensated for the loss. But even if the rate paid is not to be determined as upon a purchase of water from the plaintiff, still, at the lowest, the plaintiff has the sole right to furnish this water, the owner of the irrigated lands cannot get it except through the plaintiff's help, and it would be unjust not to take that fact into account in fixing the rates. We are not called upon to decide what the rate shall be, or even the principle by which it shall be measured. But it is proper to add a few words.

The declaration in the Constitution of 1879 that water appropriated for sale is appropriated to a public use must be taken according to its subject matter. The use is not by the public at large, like that of the ocean for sailing, but by certain individuals for their private benefit respectively. Thayer v. California Development Co., 164 Cal. 117, 128. Fallbrook Irrigation District v. Bradley, 164 U. S. 112, 161. The declaration therefore does not necessarily mean more than that the few within reach of the supply may demand it for a reasonable price. The roadbed of a railroad is devoted to a public use in a stricter sense, yet the title of the railroad remains, and the use though it may be demanded, must be paid for. In this case it is said that a part of the water was appropriated before the Constitution went into effect, and that a suit now is pending to condemn more as against a riparian proprietor, for which of course the plaintiff must pay. It seems unreasonable to suppose that the Constitution meant that if a party instead of using the water on his own land, as he may, sees fit to distribute it to others he loses the rights that he has bought or lawfully acquired. Recurring to the fact that in every instance only a few specified individuals get the right to a supply, and that it clearly appears from the latest statement of the Supreme Court of California, Palmer v. Railroad Commission, Jan. 20, 1914, that the water when appropriated is private property, it is unreasonable to suppose that the constitutional declaration meant to compel a gift from the former owners to the users and that in dealing with water "appropriated for sale" it meant that there should be nothing to sell. See San Diego Water Co. v. San Diego, 118 Cal. 556, 567. Fresno Canal & Irrigation Co. v. Park, 129 Cal. 437, 443 et seq. Stanislaus Water Co. v. Bachman, 152 Cal. 716. Leavitt v. Lassen Irrigation Co., 157 Cal. 82. Decree reversed.

BUILDINGS

Design and Construction Features of the Central Railroad of New Jersey Locomotive Terminal at Communipaw, N. J.

(Staff Article.)

To provide increased facilities for handling locomotives rapidly and economically the Central Railroad of New Jersey has recently had constructed a modern locomotive terminal at Communipaw, N. J. The new plant is located on the south side of the main line tracks about one mile west of the passenger terminal and is in close proximity to the freight yards. As the freight and passenger yards are on opposite sides of the main line tracks, the terminal facilities are arbitrarily divided so that the freight and passenger engines are handled separately. The track arrangement, however, provides the necessary flexibility to permit the free use of either house for passenger or freight engines. The track layout leading into the terminal is especially designed for the rapid handling of passenger engines, formerly cared for in the two old engine houses at Fiddlers and Communipaw. These houses were located on the north side of the main track, and 30,823 locomotives were handled through them during the first four months of 1914, an average of 255 per day. During the summer months about 300 engines are handled per day.

The completed improvement consists of a power house (to serve not only the engine terminal but also to take care of all electrical requirements of the railroad from the Jersey City water front to Newark Bay); one 100-ft., 34-stall roundhouse; one 90-ft., 32-stall roundhouse; machine shop; blacksmith shop; storehouse and office, material platform; oil house; cinder pits; coaling station; sand storage; roundhouse office and toilet building; engineers' locker building; telephone tower; and all equipment required for the building and yard. Figure 1 is a layout of the terminal, and shows the location of each unit of the plant.

CHARACTER OF SOIL AND TYPE OF CONSTRUCTION.

The ground is principally cinder fill, varying in depth from 2 ft. to 10 ft., the underlying strata consisting of blue clay, sand and bog, except where the old shore line crosses the west end of the site. In consequence all buildings rest on pile foundations, with the exception of the west half of the roundhouses where the ground conditions are favorable for footings. The footings of the buildings consist of concrete pile caps and piers, where the concentrated loads are imposed, with reinforced concrete wall girders supporting the building walls. The power house, however, rests on a concrete slab 4 ft. thick, which extends under the entire building. This slab is supported on piles, equally spaced under the entire mattress. All buildings are constructed of reinforced concrete, steel and brick, with steel sash, wooden doors and concrete floors. The roofs of all buildings, except the coaling station, are covered with three-ply asbestos felt roofing.

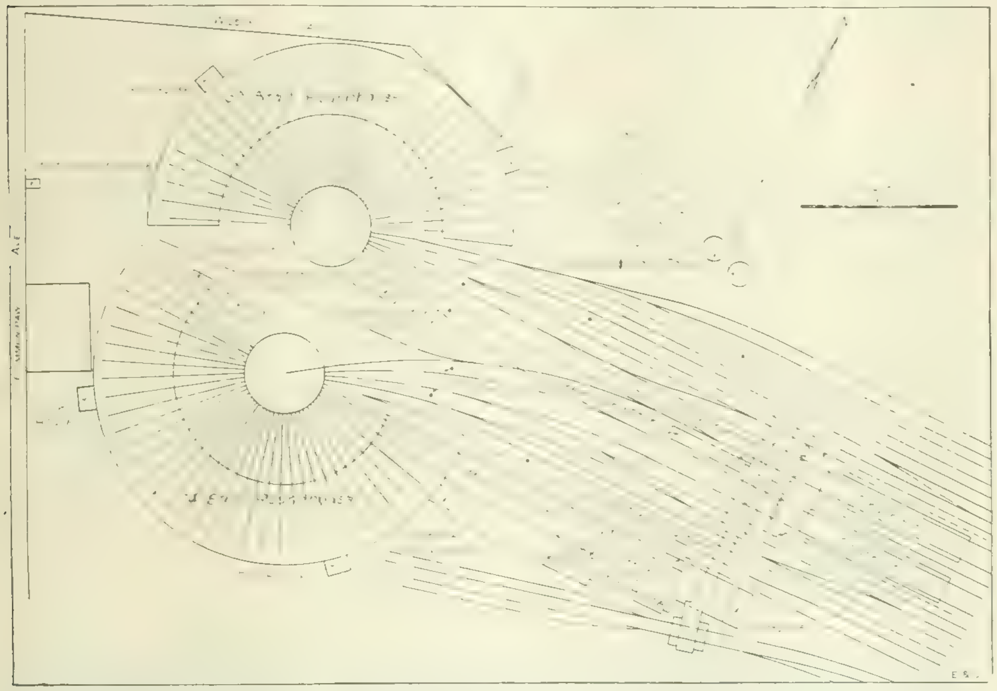


Fig. 1. Layout of Buildings and Equipment for Central Railroad of New Jersey Locomotive Terminal at Communipaw, N. J.

POWER HOUSE.

The power house is 135 ft. long and 92 ft. wide, with concrete boiler and machine foundations. The building proper is of brick, with a structural steel frame for supporting the boilers, stack and coal bunkers. It is equipped with steel sash and steel doors.

Six 250-HP. "Babcock & Wilcox" water tube boilers arranged in three batteries of two each are installed, and space is provided for an additional battery. The boilers are fed by automatic stokers and two reciprocating, duplex, plunger-type pumps, either of which is capable of furnishing the maximum amount of water needed for the boiler plant. A lined steel stack 10 ft. 6 ins. in diameter and 75 ft. in height above the roof furnishes natural draft, aided by automatically controlled turbine blowers. The feed water and steam piping are of the loop type. A "Cochrane" feed-water heater provides feed water at a temperature of about 200° F.

Hopper-bottom cars deliver coal into a track hopper, from which it is elevated by a bucket elevator, discharging into a flight conveyor which distributes the coal into bunkers located over the boiler room. The coal is fed by gravity to the stokers through chutes, the supply being regulated by gates operated from the boiler room floor (see Fig. 2).

The ashes are dumped from the stoker into a hopper directly underneath. A bucket, supported on a car which travels on an industrial track, is pushed underneath the hopper and is loaded by gravity with ashes. It is then pushed to a point below an electrically driven hoist, which elevates the bucket and automatically dumps it into a hopper above the railroad siding. The ash and coal handling arrangement is such that the railroad car is used to bring in coal and after being unloaded the same car is loaded with ashes without changing its position.

Three 600-kw., 2,200-volt alternating current turbo-generators are installed, with space provided for a fourth unit, together with one steam-driven and one motor-driven exciter. Two 2,500-cu. ft. capacity, two-stage air compressors furnish air for the engine terminal, and for operating the switches and signals in

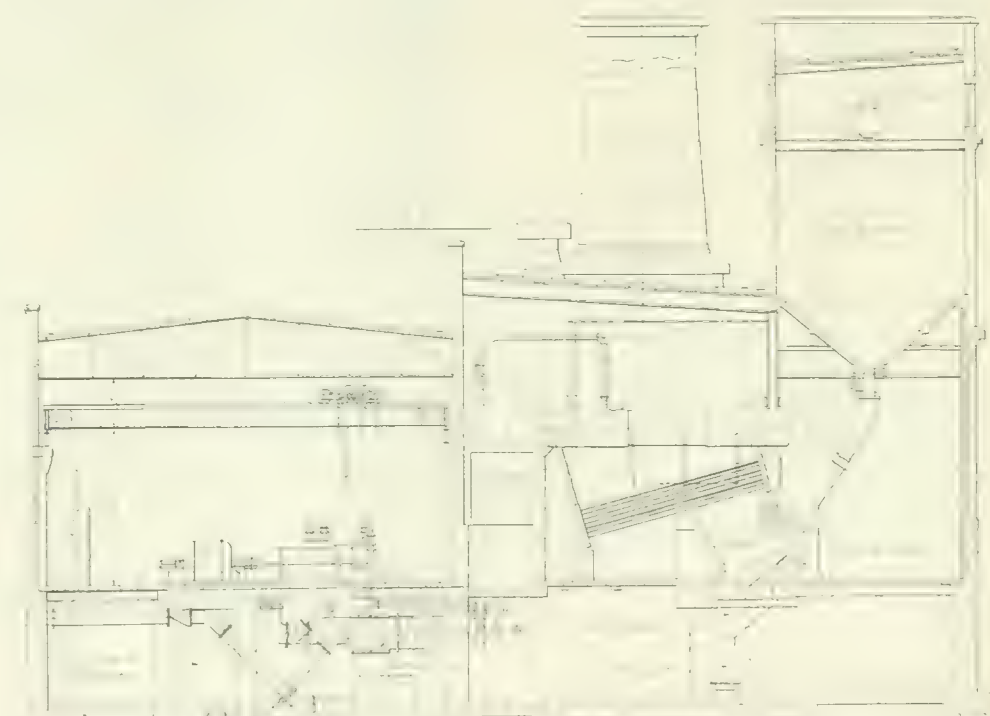


Fig. 2. Cross-Section of Power House for Locomotive Terminal Showing Type of Construction and Arrangement of Equipment.

the terminal yard between Communipaw and Jersey City, and also to Elizabethport and Newark on the main line and on the Newark branch.

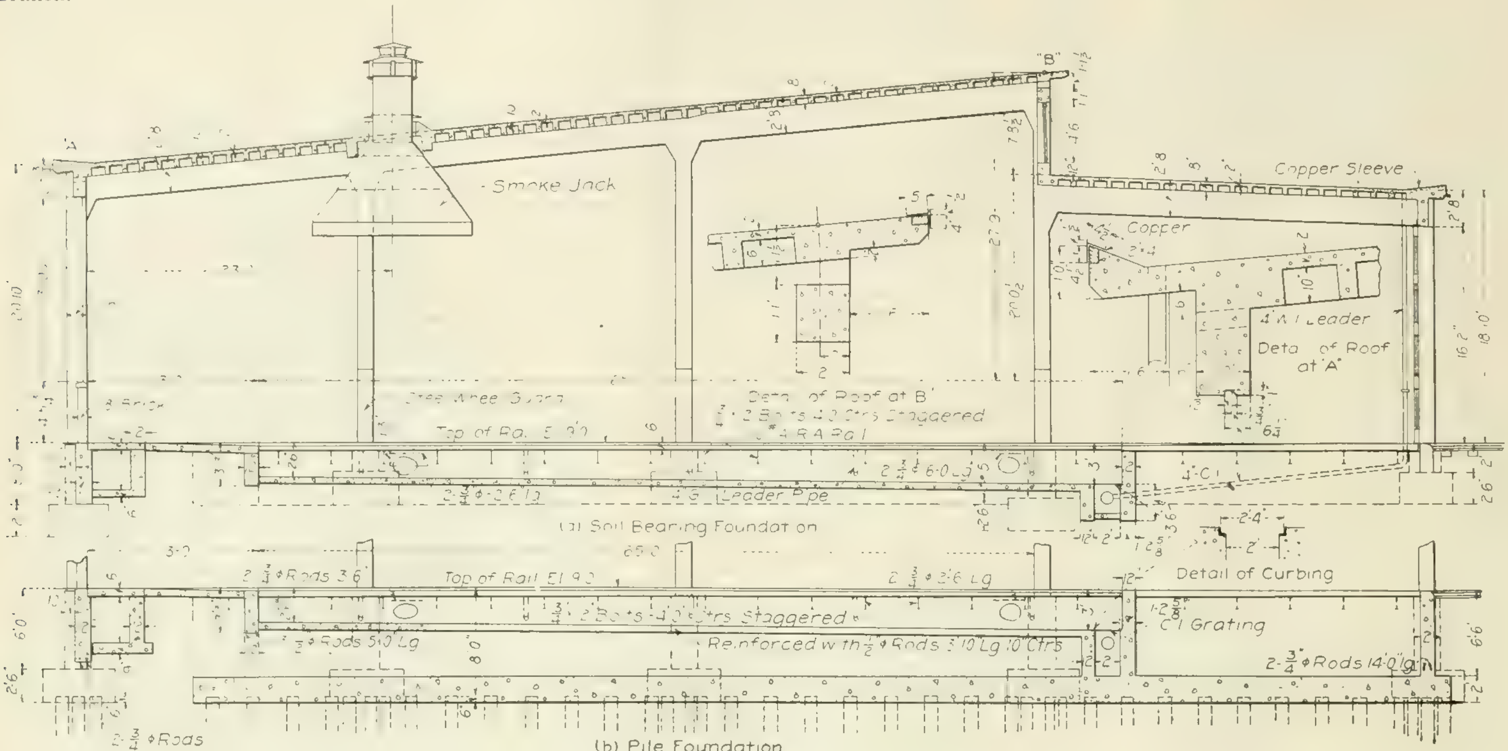


Fig. 3. Cross-Section of 100-ft. Roundhouse of Locomotive Terminal Showing Details of Construction and Types of Soil Bearing and Pile Foundations.

The plant operates normally as condensing, although in cold weather the exhaust steam is used for heating the buildings of the terminal. A mixing condenser is located in the engine room basement. As the water used for condensing purposes is taken from the Jersey City mains and is metered, it is cooled in a cooling tower and is again used. An automatically controlled, motor-driven centrifugal pump located in the basement of the plant is used for raising water into elevated tanks when necessary. There is also a 1,500-gal. underwriters' pump connected to high-pressure fire lines in the yard and buildings. The jacket circulating water is returned to the water system by means of a duplex steam pump.

The main switchboard is located on the engine room floor, with high tension switches in the basement where the transformers are located. The current is generated at 2,250 volts and is stepped down to 550, 220 and 110 volts for use at the terminal proper.

A 10-ton "Maris" hand-power crane spans the engine room floor (see Fig. 2).

ROUNDHOUSES.

The roundhouses are constructed with reinforced concrete columns, piers and roof girders and with combination hollow tile and concrete roofs. The rear wall consists of concrete piers approximately 5 ft. wide with steel sash between them and with an 8-in. brick wall below the windows. This arrangement permits a maximum window space both for lighting and for ventilating. The roof line is broken at the first row of columns at the front of the building to give a row of hinged sash over each stall. Figure 3 shows a cross-section of the 100-ft. roundhouse and indicates the type of construction. Additional ventilation facilities are provided by means of three-chamber 4-in. hollow tiles set in the rear wall above the windows and in the locomotive door lintels. Ventilating openings are also placed above the sash in the roundhouse monitors. This arrangement follows the Central Railroad of New Jersey standard practice, and provides an outlet for any gases which may collect underneath the ceiling (which is flat), thus giving an unobstructed path for gases to pass out through the ventilator openings. Both houses are heated by the indirect system. The fans and heaters are located in the fan houses, of which there are two in each house. The hot air is delivered through underground ducts and is discharged through outlets located in the pits and around the rear wall. The floor wearing surfaces are concrete throughout, and are reinforced along the sides of the pits to provide bearing for jacking.

Fig. 4. Interior View of 100-ft. Roundhouse of Locomotive Terminal. Note Asbestos Smoke Jacks.

A boiler washing plant serving 32 stalls has been installed in connection with the 90-ft. house. The piping is of such size as to permit an extension of the system to the 100-ft. house should it be desired later.

Asbestos smoke jacks are at present installed, but the roofs of the houses were designed strong enough for cast-iron jacks should it be found desirable to install them later.

Both houses are lighted by tungsten lamps, the wires being carried in conduits under the floors and in the columns.

There is installed in the 90-ft. house one driver and one truck drop-pit and in the 100-ft. house two driver drop-pits and one truck drop-pit, each extending over three stalls. These have pneumatic jacks for the wheels and ½-ton cranes for handling other parts.

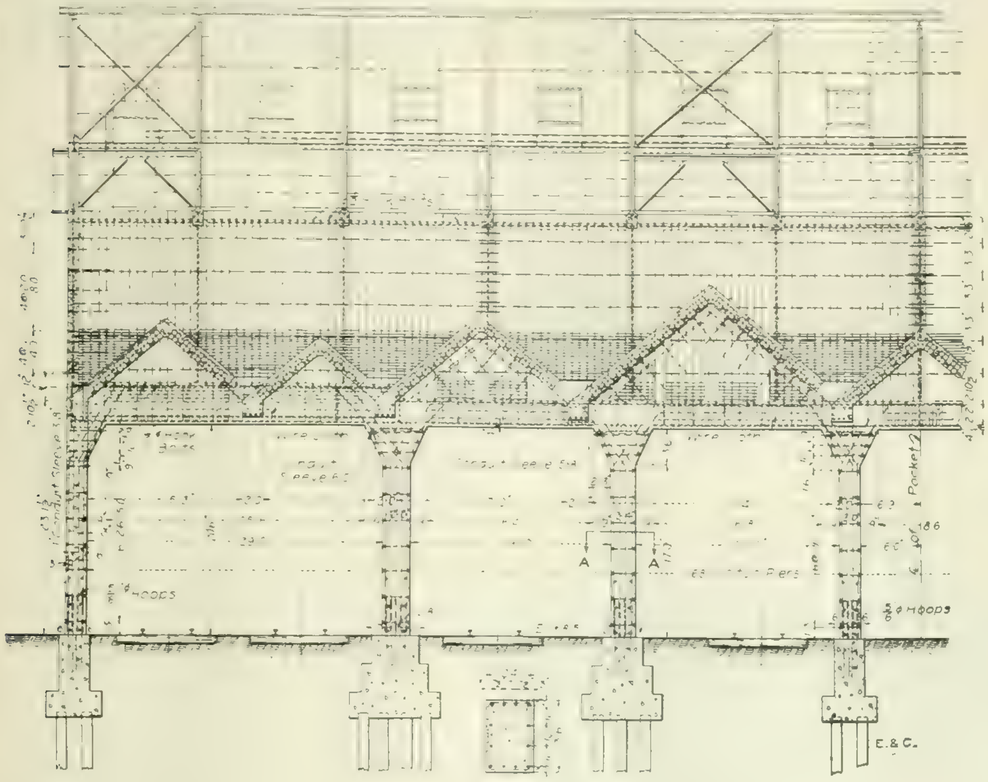


Fig. 5. Half Longitudinal Section of Coaling Station of Locomotive Terminal Showing Type of Construction and Dimensions.

The entrance doors to the stalls are hinged to pintle posts, which are entirely separate from the building construction proper. They are fastened to the building columns in such a manner that the accidental wrecking of a door will not damage the building proper. Figure 4 shows an interior view of the roundhouse and indicates the type of construction.

TURNTABLES.

Each roundhouse is served with a 100-ft. deck turntable of heavy construction operated by electric tractors. Owing to the extreme depth of these turntable pits and to the shallow grade of the sewer in the vicinity it was found necessary to provide against the contents of the sewer backing up into these pits. This was accomplished by constructing a deep sump into which both pits are drained. Automatic ejectors discharge this drainage into the nearest sewer.

MACHINE AND BLACKSMITH SHOPS

Adjoining and directly connected to the 100-ft. roundhouse is the machine and blacksmith shop building (see Fig. 1). The total length of this building is 200 ft., its total width is 80 ft., and its total height is 28 ft. A monitor 13 ft. wide extends the entire length of the building and is provided with a continuous top-hunt steel sash, operated from the machine shop floor. Toilet and locker room facilities are provided in a small extension located between the main building and the 100-ft. roundhouse. Access to this toilet room and to the locker room may be gained either from the machine shop or from the roundhouse.

The machine shop space is 140 ft. long by 80 ft. wide, with a concrete floor throughout. The equipment consists of small lathes, crank planers, and other machines such as are required for light repairs. Two motor-driven line shafts near the north wall furnish power for the small machines. A motor-driven wheel lathe for driving and truck wheel work is located near the center of the building and is served by a four-ton overhead trolley. One of the roundhouse tracks is extended through the machine shop into the blacksmith shop.

In the southeast corner of the machine shop, space is provided for pipe work. Besides two pipe forges this space contains an 8-in. pipe machine, pipe racks, benches, etc.

The blacksmith and boiler shop is located at the east end of the building, being separated from the main or machine shop by a fireproof wall. This space is 40 ft. wide by 80 ft. long, the south half being occupied by the blacksmith shop, which is equipped with five down-draft forges, each served with a ½-ton jib crane. A 2,000-lb. steam hammer, served by a three-ton jib crane, is located in the center of the blacksmith shop. The equipment of the boiler shop consists of a motor-driven punch and shears, hand-bending rolls, a flange fire and a screw flanger. Heating of the building is by direct radiation and lighting by tungsten lamps, varying from 25 to 500 watts. A concrete ramp is located at the northeast corner of the building. This leads to the material platform, by means of which material may be brought directly to the machine and blacksmith shops from the storehouse.

STOREHOUSE AND MATERIAL PLATFORM.

The storehouse is 100 ft. long by 60 ft. wide and directly adjoins the blacksmith shop. Its construction is fireproof throughout. Steel bins and counters are provided for storing material, so that, except for their combustible contents, the fire hazard is reduced to a minimum. The building is at present one story in height, but the foundation, the walls and the columns are designed sufficiently heavy for a second story. The east part of the building is divided by fireproof partitions into offices for the general foreman and the storekeeper, and into the toilet and wash rooms. The material platform is 48 ft. wide by 80 ft. long, and extends 12 ft. in width along the north side of the storehouse. This structure is built of reinforced concrete and hollow tile, with a concrete wearing surface.

OIL HOUSE.

For the storage of the various kinds of oil which are used at the engine terminal, an oil house is located at the extreme east end of the material platform. This building is 20 ft. wide by 48 ft. long; it is one story in height; and is provided with a basement 10 ft. high, in which the various oil storage tanks are located.

The measuring pumps and boxes for filling the storage tanks are located on the main floor where space is also provided for storage of waste and grease cakes. The building is lighted by tungsten lamps and is heated by direct radiation to a high temperature to keep the oil in a fluid condition in cold weather.

COALING STATION.

The most interesting structure of the group, both on account of its size and construction, is the coaling station. The main building spans eight tracks and serves an additional track at each end. This structure is 168 ft. long, 34 ft. wide and 55 ft. high and is of reinforced concrete throughout. The bunkers rest on steel I-beam girders encased in concrete, while the hopper bottoms are built of reinforced concrete and terra cotta tiles. The sides of the bunkers are heavily reinforced to withstand the lateral pressure of the coal when the bunkers are filled. A monitor, which extends the full length of the structure, has steel trusses with 2-in. plastered concrete sides, and an asbestos roof. Figure 5 shows a longitudinal section of the north half of the building taken on the center line of the structure, and Fig. 6 shows a view of the completed structure, looking north.

The coal is received from the cars by two receiving hoppers, from which it is discharged by means of reciprocating feeders into bucket conveying elevators. These conveying elevators carry the coal to the top of the hopper house, where it is discharged onto two 30-in. belt conveyors, which run up the conveyor bridge and over the top of the bunkers. Traveling trippers, which run on rails above the bunkers, discharge the coal into the various compartments.

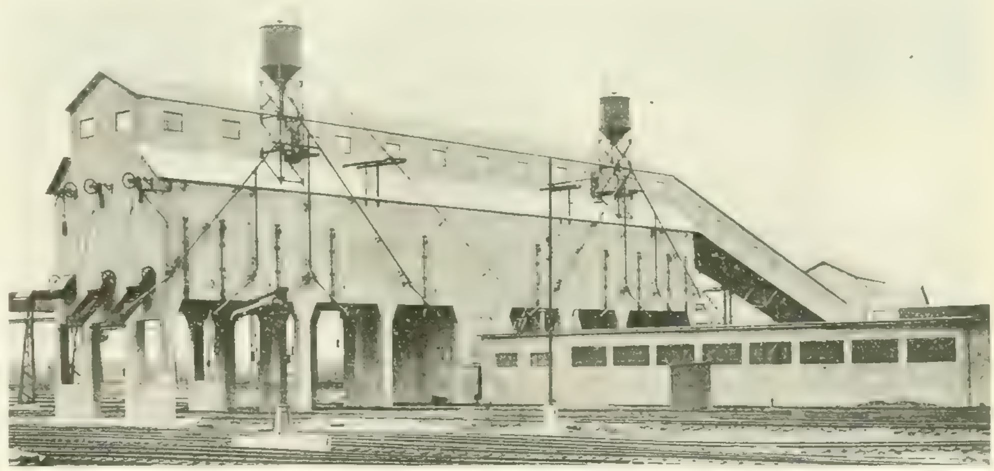


Fig. 6. View of Coaling Plant (LookingNorth) of Locomotive Terminal. Note Sand Storage Tanks at Top of Building.

There are three longitudinal bunkers, having capacities of 430 tons of bituminous, 813 tons of broken, and 430 tons of buckwheat coal, respectively. These bunkers are each divided into four compartments by transverse concrete partitions (see Fig. 5). Each track is served by three coal chutes so that an engine on any one of the ten tracks may be coaled with either bituminous, broken or buckwheat coal. The conveying machinery is divided into two separate units from the track hopper to the tripper over the bunker, each unit having a conveying capacity of 100 tons per hour. Provisions are made, however, whereby either track hopper, elevator or conveyor of one unit may discharge into the other unit, and in addition the trippers are so arranged as to discharge into either one of the three bunkers. This flexibility reduces to a minimum the possibility of shutting down the entire plant due to a breakdown or to any other emergency. The entire machinery is electrically driven.

Suitable stairways, platforms and walks are provided from which an inspection of the apparatus may safely be made while the machinery is in operation. Guards are placed over all exposed gears as a protection to the attendants, and in addition there are eleven emergency stations from which the entire machinery may be shut down by pressing a button.

West of the coaling station and south of the machine shop and storehouse are the coal storage tracks, with a capacity of 40 cars. Provisions are made for thawing out frozen coal in the cars on these tracks by means of live steam.

Under the center bay of the coaling station, at the ground level, there is a toilet and locker room for the use of hostlers, cinder pit employees and coaling station employees (see Fig. 6).

SAND HOUSE.

Provisions for the storing and drying of sand are made in a building west of the coaling plant. This building is of reinforced concrete throughout, and is 103 ft. long, 16 ft. wide and 14 ft. high. The green sand is dried by means of two coal stoves of standard Central Railroad of New Jersey design, located in a separate room in the east end of the building. The dried sand is then screened and elevated by means of compressed air to two storage tanks of 15 cu. yds.' capacity each, located on top of the coaling station (see Fig. 6). From these tanks the sand is delivered to the locomotive through cast-iron delivery pipes and wrought-iron telescoping spouts which serve each of the ten tracks.

CINDER PITS.

The cinder pits are located about 60 ft. east of the coaling plant, and are of the submerged type. They are each 202 ft. long, 30 ft. wide and 12 ft. deep, and are built of reinforced concrete throughout.

Each pit serves two tracks, spaced 26 ft. center to center (see Fig. 7). The pits are parallel and are spaced about 58 ft. center to center, with a track for cinder cars between. The cinders are cleaned out of the pits by a four-ton electric traveling crane, operating a 1⅝-cu. yd. clam-shell bucket. This crane is located on a steel runway, 240 ft. long, 99 ft. 6 ins. between girders, and 26 ft. above the rail. Aside from the economy and speed in handling engines over the pit this arrangement permits the coaling of engines from cars by means of the clam-shell bucket should occasion arise.

MISCELLANEOUS BUILDINGS.

Among the miscellaneous buildings are the engineers' tool storage building and the roundhouse toilet and office building. These buildings are 20 ft. wide by 55 ft. long and 22 ft. wide by 52 ft. long, respectively. Both buildings are heated by direct radiation and are lighted by Tungsten lights. The engineers' tool storage building is equipped with steel lockers of special design for storing the locomotive engineers' tool chests, etc.

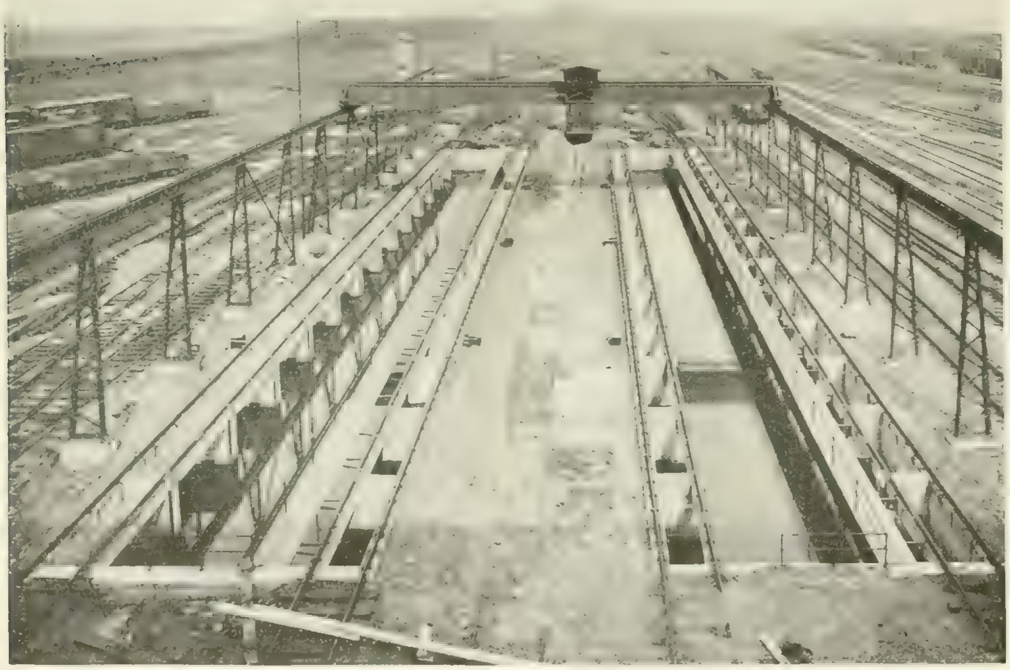


Fig. 7. View of Cinder Pits of Locomotive Terminal and of Runway and Traveling Crane Serving Same.

The roundhouse office and toilet building is divided by a tile partition into an office room at the east end and toilet and locker room with steel clothes' lockers and toilet facilities at the west end. The building is so situated that an unobstructed view of all inbound and outbound tracks in the terminal yard is obtained from the office of the engine dispatcher.

The telephone tower is located at the east end of the yard, from which the operator has full view of all outbound engines so that he can notify the tower man as to their location. This is a wooden building with the operator located on the upper floor.

TUNNEL SYSTEM.

A tunnel is provided from the power house to the roundhouse for carrying all steam, air and water service piping, and also all light and power wires and cables for the buildings. This tunnel is of heavy reinforced concrete construction, waterproofed, and is 6 ft. wide, 7 ft. high, and 367 ft. long, with a branch 60 ft. long running to the 90-ft. roundhouse. The tunnel is well lighted and affords ample working space for making repairs.

WATER SERVICE.

The water piping is divided into two systems:

(a) The water supply is taken from a 16-in. main of the Jersey City water service and is discharged by city pressure first through altitude valves into two 100,000-gal. steel elevated tanks and then through the low, or service, system of piping to eight water columns in the yard, these columns being used for filling the engine tanks and also for general use in all of the buildings.

(2) A high-pressure system of piping is carried around the property and into the various buildings from the fire pump in the power house for fire protection.

SEWERS.

A complete system of sewers has been installed to take care of all roof drainage and all drainage from engine pits, power house, toilets, turntables, etc., and in addition catch basins have been installed throughout the yard for track drainage.

YARD LIGHTING.

The engine terminal yard is lighted throughout by fifteen 125-volt, alternating current flaming arc lamps. Four of these lamps are suspended from reinforced concrete poles located around the inner circle of the roundhouses, the remainder being suspended from tubular steel poles located at convenient points. Provisions are made for lowering these lamps to the ground for trimming. All conduits and all wires which supply current for this lighting system are underground.

FUEL OIL PIT.

Fuel oil is piped from an oil pit of 8,000 gals. capacity located between the west ends of the two roundhouses.

SUMMARY.

The principal quantities of materials used in the construction of this plant are as follows:

Concrete used, cu. yds.	27,000
Structural steel erected, tons	966.3
Number of bricks used	1,473,100
Steel for reinforcing concrete, tons	736.5
Gravel and broken stone for concrete, cu. yds.	26,565
Sand for concrete, cu. yds.	16,000
Lumber for forms, scaffolds, sheet piling, etc., ft. B. M.	841,990
Bags of cement used	175,494
Excavation and fill required, cu. yds.	54,366

The following dates indicate the time required to construct the plant:

Job authorized	Oct. 23, 1912
Field engineering work started	Nov. 8, 1912
Demolition of old building started	Nov. 27, 1912
First pile driven	Jan. 9, 1913
First concrete poured	Jan. 17, 1913
100-ft. roundhouse, power house, machine shop, storehouse, oil room, two turntables, and cooling station, ready for use	Nov. 15, 1913
90-ft. roundhouse ready for use	Dec. 15, 1913

PERSONNEL.

The locomotive plant was designed and constructed by Westinghouse, Church, Kerr & Co., of New York, in co-operation with and under the direction of Mr. Joseph O. Osgood, chief engineer, and Mr. A. E. Owen, principal assistant engineer, of the Central Railroad of New Jersey. We are indebted to the former company for the data contained in the article.

Results of Some Experiments to Determine the Distribution of Wind Forces on a Building.

Our present assumptions as to the distribution and magnitudes of wind loads on buildings are undoubtedly far from correct, and there is need of experimental data on the distribution of wind loads for different types of buildings. The following tests, although by no means exhaustive, at least show that our present methods are inaccurate and emphasize the need of more data. The experiments were conducted on a small model, at low wind velocities, but the results are of particular value in showing the distribution of the wind pressure and suctions. The data were abstracted and largely rearranged from a paper presented before the Western Society of Engineers by Albert Smith.

THE MODEL.

The model used in the experiments consisted of a roof, semi-circular in section, having a span of 6 ft. and a length of 10 ft. This roof was covered with tin, and was mounted on walls 5 ft. high, constructed of 2x6-in. tongue-and-groove lumber. The use of tongue-and-groove lumber made the walls comparatively tight, and also give stiffness to the structure when an end or side was taken out during the course of the tests. At a middle section of the building ½-in. diameter holes, spaced about 1 ft. apart, were bored in the walls and roof, the row of holes extending from ground to ground. In the roof a short nozzle was the recording instrument was then placed on these wedges. The 3x3x¼-in. angle which formed the base of the instrument was planed on the bottom and also on the upper edge. The twenty ¼-in. glass tubes (curved at one end) of the instrument were placed in a rack which was attached to the base angle, the tubes being separated by spacers of equal thickness. Great care was used to make the tubes sene. Convenient length units were marked off on the plaster of paris surface between the tubes. The upper ends of the scale tubes were connected to the holes in the roof and walls.

In selecting the scale tubes great care was taken to secure tubes of the same inside diameter and of uniform diameter from end to end.

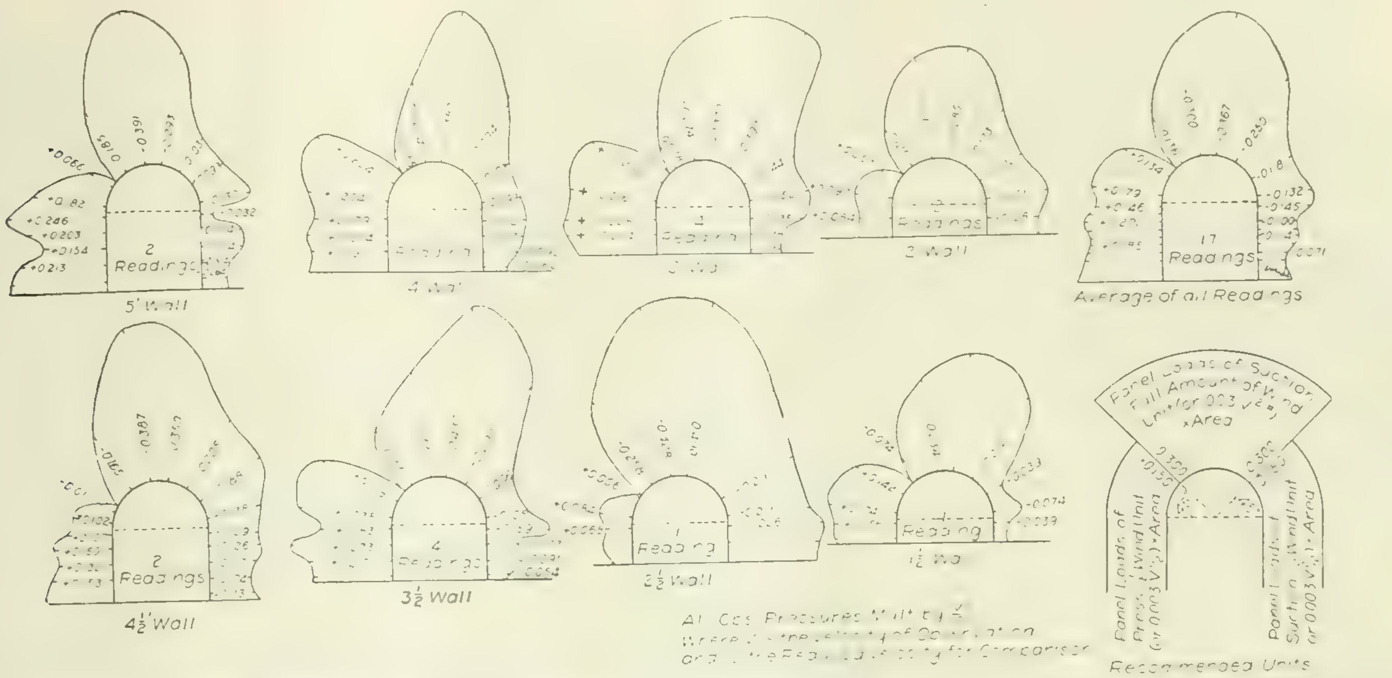


Fig. 1. Diagrams Showing Distribution of Wind Pressures and Suctions on a Model Building for Various Heights of Walls—Also Recommended Distribution and Units.

soldered on the inside. A series of holes at various levels was also carried completely around the building. The model was so constructed that after each set of observations the roof could be lifted and a 6-in. course of the walls taken off.

PRESSURE REGISTERING APPARATUS.

The pressure recording apparatus was constructed as follows: A concrete base was set

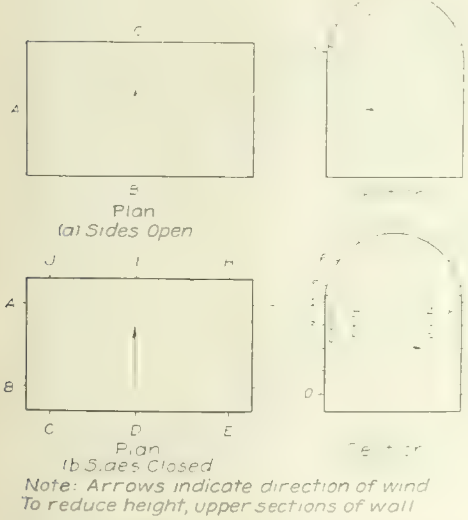


Fig. 2. Direction of Wind and Arrangement of Tubes for Building With Open and Closed Sides.

in the ground, and upon it there was placed a cast-iron plate. Another cast plate, planed on its upper surface and equipped with set screws in the corners, was set on this plate, the upper plate being brought to a horizontal plane by means of a delicate level. Two iron wedges, differing a known amount in their heights, were placed on the upper plate, and

straight in side elevation—i. e., to turn them until all of their curvature was in planes normal to the rack. The space between the backs of the tubes was then filled with plaster of paris. Tin reservoirs, 3 ins. x 3 ins. x 1 in., were soldered to the sheet-iron back of the rack, the reservoirs being placed in sloping rows on the back, so that when the instrument was tilted ¼ in. in 10 ins. a horizontal plane,

OBSERVATIONS AND REDUCTION OF READINGS.

It was determined that one division rise in the tube corresponded to a pressure of 0.0546 lb. per square foot. In determining the wind velocity from the pressure the formula $V=\sqrt{\frac{P}{0.003}}$ was used; where P = pressure in pounds per square foot on thin, flat plates, and

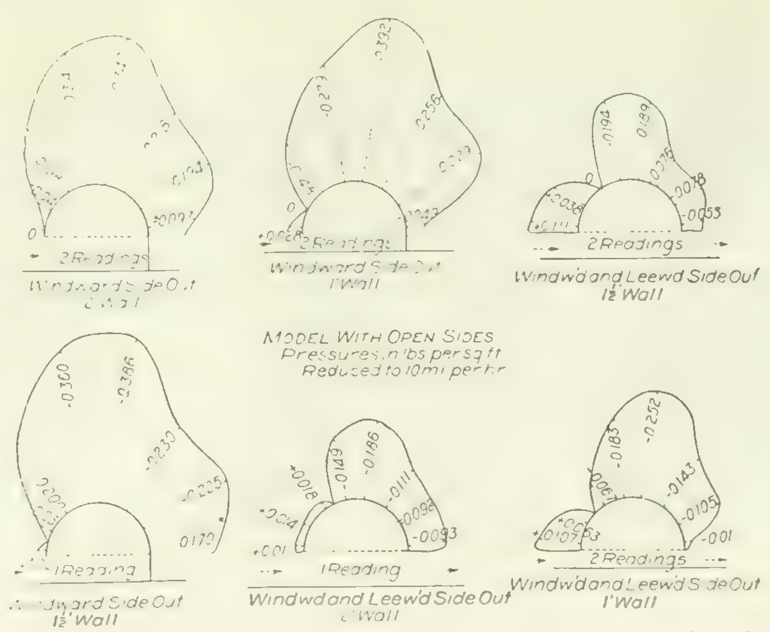


Fig. 3. Pressure and Suction Diagrams for Model Building with One and Both Sides Open.

if passed through the center of any tube, would also pass through the middle of the walls of the reservoir belonging to that tube.

Connections between the tubes and their reservoirs were made with bent tubes, which ran around the back of the instrument, and were joined to the scale tubes at their lower ends. The reservoirs were filled with kero-

V = velocity of wind in miles per hour. The velocity of the wind was determined for each observation.

After each observation, or set of observations, the instrument was disconnected, and it was noted whether or not the fluid in the tubes returned to the zero of the scale. If the fluid in some of the tubes did not return the tubes were readjusted, and duplicate readings were taken to insure accuracy.

After the first set of readings was taken one of the frames making up the side walls was removed, thus reducing the height of the wall 6 ins. The instrument was again connected, and another reading was taken. This operation was continued until the height of the walls was reduced to 1½ ft., which was as low in comparison to the span of the roof as the observers thought would be used in practice.

After the readings were taken, (a) the pressure or suction inside of the building was standardized, (b) the result was reduced to pounds per square foot, and (c) this result was then reduced to that for a velocity of 10 miles per hour.

PRESSURE OR SUCTION INSIDE OF THE BUILDING.

The pressure and suction readings were very unequal, partly because the observers were obliged to work in a somewhat gusty wind, and partly because small variations in the direction of the wind caused great changes in the conditions of exposure of the various leaks which were not symmetrical around the building.

The amount of pressure or suction was determined as follows: During each observation "Pitot" tubes were set up in front of the building and connected with two scale tubes on the instrument. The readings of these two tubes were compared with the readings which would have been given if there had been zero pressure inside of the building (which was obtained by calibration). In calibrating the Pitot tubes, these tubes and a 6x6-in. plate were set up 30 ft. in front of the house and a long series of readings was taken. It was found that the plate readings were 1.2 times the Pitot tube readings, comparing the sums of the pressure and suction of each instrument. These readings were taken while the house was very thoroughly calked, therefore conditions were nearly ideal. The average ratio of the Pitot tube pressure reading to the Pitot tube total (pressure plus suction) was found to be 0.917. If for any observation the Pitot tube pressure exceeded this percentage of the total an undue rarefaction existed inside of the building, and this was removed, as is shown by the following example:

For observation No. 1 the Pitot tube readings were $+1.8$ and -0.5, a total difference of 2.3.

$2.3 \times 0.917 = 2.11$.

$2.11 - 1.8 = 0.31$, which is the excess pressure inside of the building. All pressure readings were therefore increased by 0.31, and all suction readings were diminished by this amount.

As has been noted, one unit on the scale of the instrument corresponds to 0.0546 lb. For observation No. 1, tube No. 1, we have $1.7 \times 0.0546 = 0.0928$ lb. per square foot; and $0.31 \times 0.0546 = 0.0169$ lb. per square foot.

Total pressure $= 0.0928 + 0.0169 = 0.1097$ lb. per square foot.

Using the formula $P = 0.003\,V^2$ for the pressure on thin plates (Proc. Inst. C. E., Vol. CLXXI, p. 191), we have for thin plate pressures, at a wind velocity of 10 miles per hour, $P = 0.3$ lb. From the ratio determined by calibration the Pitot tube pressure will be

$$P = \frac{0.3}{1.2} = 0.25 \text{ lb. per square foot.}$$

The velocity, V, for observation No. 1 (given above),

$$\sqrt{\frac{2.3 \times 0.0546}{0.0025}} = 7.1 \text{ miles per hour.}$$

The total pressure $= 2.3 \times 0.0546 = 0.1256$ lb. per square foot.

For observation No. 1, tube No. 1, we have

$$\frac{0.1097 \times 0.25}{0.1256} = 0.219 \text{ lb. per square foot, when}$$

reduced to a velocity of 10 miles per hour.

PRESSURE DIAGRAMS.

All of the observations taken at one height were averaged together and the results plotted. Figure 1 gives the computed pressures, in pounds per square foot at a velocity of 10 miles per hour, for various heights of wall, together with the recommended units and their distribution.

These tests do not give satisfactory data for a scientific analysis of the action of the wind around an obstacle, as the winds with which these results were obtained were gusty and the exposure was far from ideal. It is evident that the wind current, when the observation on the 2½-ft. wall was taken, was moving upward at a small angle with the ground. The contrary condition seems to be traceable in some of the other readings. The average of 17 such readings, however, is quite reliable to show the effect in a horizontal stream of air.

Based on the average of the readings the writer proposes a conventional loading of the full wind specification over 90° of the roof, applied as suction, and one-half of the wind specification on the windward and leaward walls, applied as pressure and suction, respectively (see Fig. 1). These units might be used conveniently by giving each panel point which fell within the specified limits a panel load equal to the area carried multiplied by the unit for those limits.

OBSERVATIONS WITH PORTIONS OF WALLS REMOVED.

A number of observations were also made on the model with different sides and ends omitted. Figure 2 shows the marking of the sides with reference to the wind direction and also the arrangement of the tubes. Tubes Nos. 1, 3, 5, 6, 8, 10, 11 and 13 were connected to nozzles soldered in the roof, and therefore give outside pressures or suctions; while Nos. 2, 4, 7, 9 and 12 were attached so that their open ends were exposed just under the roof at the points indicated.

To secure uniform air-pressure conditions over all of the reservoirs a curtain was hung over the back of the instrument, so that the "inside pressure," which was standardized or reduced to ideal conditions in the observations on the closed house, here represents only the pressure conditions inside of the curtain around the instrument. Although the pressure was thus made uniform over all of the reservoirs it is to be noted that the pressure under the curtain is not even approximately the same as that in a closed building and varies widely in different observations. A slight displacement of a fold of the curtain made a great difference in the pressure over the reservoirs, so the ratio between pressure and suction in the Pitot tube readings varies widely. The readings on the roof might have been reduced by comparison with the standard experiments on thin plates, and thus made to show the actual pressure or suction, but the writer considered the sums of inside and outside effects to be the most interesting results of the tests.

The results of these experiments are shown in Fig. 3. Since there are no inside readings opposite the points Nos. 3 and 11 (see Fig. 2) the averages of the readings for Nos. 2 and 4, and of Nos. 9 and 12 were combined with these readings.

The values shown in Fig. 3 indicate very plainly that the forces on a building with one side open, although greater than for a structure of the same height with the sides closed, do not exceed the proposed units. With the wind in the opposite direction the pressure on the closed wall would be increased by a rarefaction inside of the building, and the suctions over the roof would be diminished. In such a case it would be proper to make the wind loads on the windward wall equal to the total wind unit.

The observations on the model with both sides removed show that the stresses for this case are less in magnitude but have nearly the same distribution as for the closed building.

APPLICATION OF LOAD UNITS TO ROOFS OTHER THAN CIRCULAR.

In applying the proposed unit loads the writer would draw, on the cross-section of the building, a semi-circle with the half span as a radius, placing the center of the semi-circle at such a distance below the peak that the area of the cross-section of the building above the base of the semi-circle is the same as that of the semi-circle; the 90° radii will then intersect the roof at the point of change in the loading units.

ROADS AND STREETS

Construction Features of an Asphalt Block Pavement Built at Newburgh, N. Y.

Contributed by James L. Kehoe, General Contractor, Newburgh, N. Y.

The work described herein consisted of the construction of concrete curbs, the excavation for and paving with asphalt blocks on a concrete base of a street within the corporation of Newburgh, N. Y., which street is one of the main approaches to Route No. 3 of the New York state highway system.

The length paved was 6,000 ft. and the width 40 ft. Figure 1 shows an average street cross section without car tracks, which occupied the center for a portion of the distance. The total contract price was $74,292.72 and included the following items: 25,000 sq. yds. of excavation, concrete foundation and 2 in. asphalt block. 11,912 ft. of concrete curb with protected corners, 3,107 cu. yds. extra excavation, and 35 inlet basins, manholes and catch basins.

CONCRETE CURB.

The concrete curb was 6 ins. wide at the top, 8 ins. at the bottom, and 24 ins. deep, laid on a 6x12 in. cinder foundation. Wainwright steel corner bar was used with expansion joints every 10 ft.

The method of placing curb was to start a gang of men on each side of the street removing old curb, flagstones and excavating to grade. Two men followed and placed forms in position from grade stakes set 2 ft. back from the curb line. Hotchkiss steel metal forms were used, consisting of steel side rails 10 ft. long and 6 ins. wide with slots at 1 ft. intervals to receive division plates. Steel templates and wedge shaped keys fastened the parts together. Rails were joined longitudinally by a tongue and socket.

Supply piles of cement, sand and stone were placed every 200 ft. containing enough material to complete 400 ft. of curb. The concrete mixed fairly stiff in a low charging Standard gasoline mixer, was wheeled in barrows to forms. Immediately after concrete was placed two men started leveling and tamping keeping

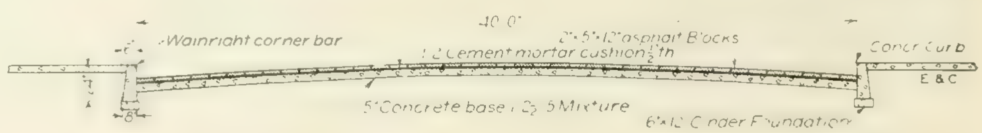


Fig. 1. Cross Section of Street Paved with Asphalt Blocks, Newburgh, N. Y.

the concrete back 1 in. from face and top. Following this the face mixture was placed. Concrete was mixed quite stiff with just enough water so that when tamped into forms there was sufficient moisture to face and top finish nicely. Two men began striking off and finishers started floating and edging top without waiting for a puddle surface to dry out. After one hour setting, forms were taken down (knocking keys out of templates), side rails removed, templates drawn through and the face of the curb floated and finished. Forms were cleaned, oiled and moved ahead.

At least 50 ft. of forms on each side of the street were always ready before pouring concrete started. Water for the mixer was supplied from a 1-in. pipe line laid by contractors from the hydrants along the sidewalk with a ¾ in. T at 100 ft. intervals. Where car tracks interfered one entire side was finished at a time. A gang, consisting of 1 foreman, 2 finishers and 10 laborers, averaged 200 ft. of finished curb per day.

EARTH EXCAVATION.

Old macadam was encountered on the section of street having no car tracks, on the remainder, gravel and hard pan. Owing to the short time allowed to complete the work, together with the hard cutting it was deemed advisable to use a steam shovel.

On the old macadam section a No. 0 Thew steam shovel of the traction type was placed in the center of the street loading from both sides into 1½ and 2 cu. yd. dump wagons. Cuts averaged from 12 to 18 ins. deep at the center line, and 4 to 6 ins. at the curb. A level cut to grade was made by the shovel across the street for a width of 25 ft. The material near the curb was piled in front of shovel by means of buck scrapers working evenings, or before starting in the morning. In this way the shovel always had enough material to load all teams for the first trip without loss of time. Owing to the shallow cutting the shovel was moved up about every 5 ft. and when there were no teams to load the shovel kept crowding ahead piling material. The average loading time for a 1½ cu. yd. wagon was 1 minute. Fifteen teams averaged 250 cu. yds. per day of material hauled away.

Fig 2. Steam Shovel Excavating Old Gravel Macadam.

On the hard pan and gravel section the shovel was placed between the trolley tracks and curb, Fig. 2, loading across the tracks into side dumping trolley cars and dump wagons. On account of the boom on the shovel having only a few inches clearance from the trolley feed wire, two extra men were employed to raise and lower the wire, using notched poles. As the traction company maintained a 25 minute schedule some time was lost by the shovel on account of the wire being raised and lowered so often. Earth between the tracks was ploughed with a heavy rooter plow hauled by a trolley car, the shoe on the plow being set so that the point just cleared the ties. This loosened material was shoveled to one side by hand and left for the shovel to load. On this section some hard shale was found about 6 ins. below grade, but the shovel had no trouble cutting through it. Many predictions of failure were heard from the "sidewalk inspectors," but after the shovel had scooped several dippers of macadam they were convinced that it would excavate the entire street. Water for the shovel was piped in the same manner as for the mixer, a ¾ in. T being placed every 50 ft.

FINE GRADING.

A fine grading gang of ten men followed closely behind the shovel grading to stakes set from a grade line stretched from curb to curb. About 600 sq. yds. of fine grading was the average per day. A Buffalo Pitts steam roller was used for rolling.

CONCRETE FOUNDATION.

A concrete base 5 ins. deep was laid, a 1:2½:5 mixture being use. Sand was screened and the maximum size stone allowed was 1½ ins. Concrete was mixed with a No. 14 Koehring batch mixer of the traction type, equipped with a 20-ft. boom and automatic dump bucket. On the section free from car tracks the material was placed in three piles—a row of sand in the center and a row of stone on either side. Cement was placed on the sidewalk in piles at intervals of 50 ft., material for these stock piles being figured beforehand so as to have just enough material dumped to supply the mixer without extra wheeling. One man handled the operating lever on the boom and dumped the bucket. The boom can be swung through an angle of 180°, and 40-ft. width of street was taken care of easily. The bucket was equipped with an automatic tripper which dumped and closed the bucket at the will of the operator. Concrete was smoothed with hand shovels to pins set from a line stretched from curb to curb. Care was taken to have concrete finished smooth, as only ½-in. cushion was used to lay asphalt blocks.

The record day's run was 1,010 sq. yds. of 5-in. concrete laid in 8 hours. Owing to shortage of material the average of this section was 800 sq. yds. per day. The gang consisted of 1 foreman, 1 engineer, 1 fireman and 20 laborers.

LAYING ASPHALT BLOCKS.

The wearing surface, Fig. 3, was made of asphalt paving blocks 2 ins. thick, 5 ins. wide and 12 ins. long, weighing 11 lbs., laid on a ½-in. cement mortar cushion consisting of one part cement and four parts sand. This bed was struck with a template from a grade line marked on the curb.

Blocks were laid at right angle to the center line of street and longitudinal joints broken by a lap of 4 ins. The blocks were covered with a perfectly dry, clean, fine sand, swept into joints and allowed to remain on pavement for 30 days.

Fig. 3. Laying Asphalt Blocks at Newburgh, N. Y.

PERSONNEL.

The making of the mortar bed and laying of blocks was done by expert pavers under the directions of Fletcher Rogers, superintendent for the Hastings Pavement Co. of New York. William J. Blake, Jr., city engineer, supervised the work. Jova & Kehoe, Inc., 64 Second St., New York City, were the contractors, the writer superintending the construction.

The Use of the Abney Hand Level in Highway Location.

Contributed by T. F. Hickerson, Associate Professor of Civil Engineering, University of North Carolina.

The Abney hand level is a more useful instrument in road engineering than the ordinary Locke level because with it both a level and incline line of sight may be established. Its usual form, Fig. 1, consists of a square bronzed

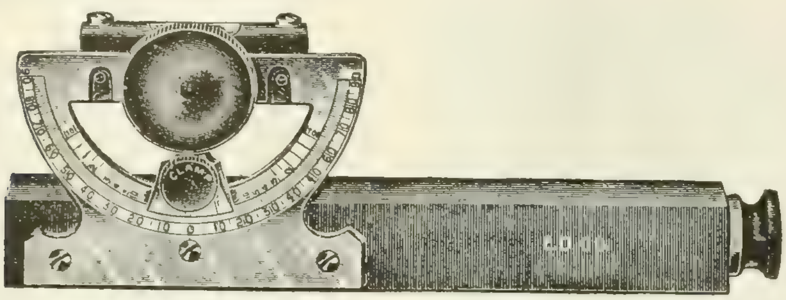


Fig. 1. Abney Hand Level.

sighting tube 5 ins. long, a vertical arc having a radius of 1 in. graduated to single degrees with a folding vernier reading to 5 minutes, a scale of grades from 1:1 to 1:10, and a bubble. The price, including a leather pocket case, is about $13.50.

METHOD OF USE.

Since grades are usually expressed as a per cent, that is, the rise or fall per 100 ft. of distance, it is necessary to know the relation

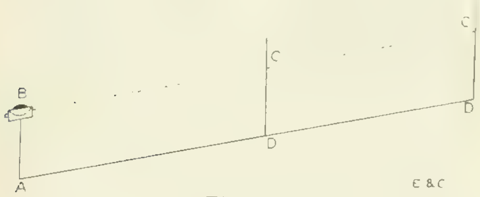


Fig. 2.

between degrees and per cent slopes, which should be remembered as being in the ratio of 4 to 7. Thus a 7 per cent grade is equivalent to a 7x4/7=4 degree grade. The following table gives the degrees and minutes corresponding to per cent grades varying from ½ per cent to 10 per cent.

Per cent slope.	Degree slope. Deg.	Min.
½	0	17
¾	0	26
1	0	34
1½	0	52
2	1	09
3½	1	26
3	1	43
3½	2	00
4	2	17
4½	2	35
5	2	52
6	3	26
7	4	00
8	4	34
9	5	09
10	5	43

To Locate Points on Grade.—Suppose A, Fig. 2, is a known point on grade and it is required to locate a point D, so that

Fig. 4.

the slope A D shall be on a given per cent grade parallel to the line of sight B C. Set the vernier to read the angle corresponding to the per cent grade and with the instrument held at B in position for sighting in the hand or on a jacob staff, then place a rod alongside A B with the bottom on a level with the point A and place a target opposite B on the rod at the "height of instrument" above A. When the "Abney" is swung slightly backwards or forwards until the bubble is in center (this position being shown by means of a prism which throws a picture of the bubble into the field of view), the line of sight B C will be parallel to the desired grade and it is only necessary to have the rod moved about until this line of sight strikes the target in order to locate points at the bottom of the rod so that they will be on grade. Thus, if the target points C and C′ are in the line of sight, the points D and D′ are on the required grade. It should be noticed that this method of establishing grade points is independent of distance.

To Determine the Slope of an Existing Grade.—Choose two points on the grade. With the instrument held above the first point in position for sighting, place a target on the rod at the "height of instrument," and with the rod held on the second point, move the vernier up or down until the bubble remains in the center and the line of sight strikes the target, then clamp the scale and read the angle. This gives the grade expressed in degrees which if multiplied by 7/4 gives the per cent grade.

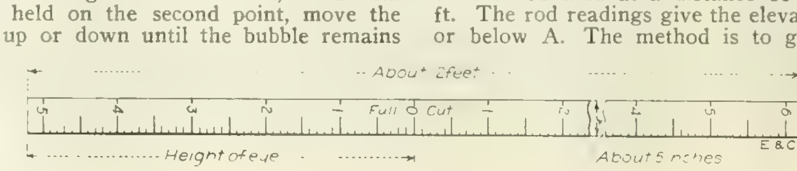


Fig. 3.

There is another way to find the amount of a slope, as in the case of a steep bank. Place the base of the level tube parallel to the surface, move the vernier until the bubble comes in the center, and read the inner scale which gives the slope directly as 1 to 1, 2 to 1, etc.

To Determine Cuts or Fills.—A convenient rod for use in connection with the hand level, Fig. 3, is graduated as follows: Mark the height of eye (same as "height of instrument") and label it zero (this would be the approximate height of eye if the rod is made adjustable at the bottom so as to be adaptable for persons of different heights), then lay off distances measured in feet and tenths downwards from the point zero to perhaps five feet or more, depending upon the height of eyes of the person who is to use the hand level, also lay off similarly from the point zero upwards to say six feet, thus giving a rod about 12 ft. long. Mark "Fill" just above the zero and "Cut" just below it.

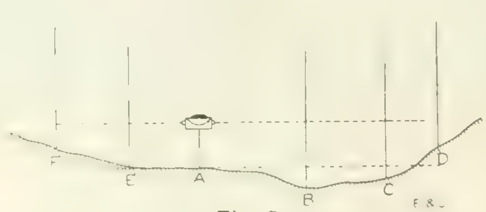


Fig. 5.

Assume A and D, Fig. 4, to be two points on grade and the instrument set to give a line of sight B C parallel to the grade A D. If the rod is held at D′ then the rod reading will be the fill D′ E, also if the rod is held at D″ the rod reading will be the cut D″ E.

To Determine Cross Sections.—Suppose A, Fig. 5, is a point on the ground at the center of the proposed road, and it is desired to find the cross profile of the ground on both sides of the point A. With the instrument set to read zero and the zero of the rod at the "height of instrument" (above A) sight on the rod held at all the breaks in the slope of the ground at measured distances from A and the rod readings will give the elevations of the various points above or below A. If the line of sight should run in the ground or be above the top of the rod, move up to the last point or some convenient point and proceed as before, remembering to add to or subtract from the reading for the second position all subsequent readings, in order to refer all elevations to the first position.

A convenient way to keep notes is to express the data in the form of a fraction, the numerator representing the elevation above or below A and the denominator the distance from A. Thus the following notes represent the conditions shown in Fig. 5:

$$\frac{+1.7}{20} \quad \frac{0.0}{9.0} \;\Big|\; \frac{-2.1}{12} \quad \frac{-1.0}{22} \quad \frac{+1.8}{30}$$

According to the notes, the point B is 12 ft. to the right and 2.1 ft. below A; also, the point D is 30 ft. to the right and 1.3 ft. above A.

To Set Slope Stakes.—Suppose the center fill at A, Fig. 5a, is 2 ft. and the road is to be 30 ft. wide with banks sloping 1½ horizontally to 1 vertically. It is required to find where the slope of the banks intersects the surface of the ground.

Set the vernier to read zero and clamp it. Use the rod shown in Fig. 3. With the instrument at the height of eye above A, sight on the rod held at a distance of at least 15 ft. The rod readings give the elevations above or below A. The method is to guess at the distance and try the elevation, or vice versa. The point S is correct because the rod reading is 0.0 and the distance is 15+2×3/2=18.0 ft. The stake S shows where the fill begins and it should be labeled thus { FILL 2.0 18.0 } indicating that it is two feet below and 18.0 ft. from the center of the finished road. The stake at S′ should be labeled { FILL 1.0 16.5 }

Differential Leveling.—The "Abney" when set to read zero can be used just as an ordinary level to determine the difference in elevation of two or more points, in cases where great accuracy is not required. If a rod graduated up and down from the height of eye is used, the notes may be kept as shown in the table.

Station.	+	—
A	...	...
B	...	5.0
C	...	4.2
D	4.0	...
Total	4.0	9.2

Difference = — 5.2

According to the notes, B is 5 ft. below A, C is 4.2 ft. below B, and D is 4 ft. above C, the difference in elevation between A and D

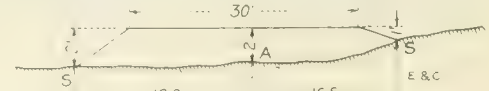


Fig 5a.

being 5.2, the difference between the sum of the plus and minus readings.

Adjustment.—Set the scale to read zero and raise or lower one end until the bubble remains in the center, then reverse the position of the instrument end for end and if the bubble is out of center the level is not in adjustment. To adjust it, turn the capstan headed screw at the end of the level tube until the bubble comes half way to the center. In order to adjust exactly, several trials are often necessary.

A good way to test the adjustment is to stand on some point A and sight on a target held at the "height of instrument" above another point B and note the angle. Then stand at B and sight on the target held above A and see if the angle is the same (on the other side of zero).

ADAPTABILITY OF THE ABNEY LEVEL TO HIGHWAY LOCATION.

The writer has found the Abney level indispensable in side hill location. It enables one to determine grade points quickly and is a time saver in running preliminary trial lines. As to its accuracy, the readings taken not over 100 ft. apart (preferably about every 50 ft.) should be within ½ in. of exactness, provided, of course the instrument is in adjustment and the engineer is careful to keep the "height of instrument" constant. The errors are compensating and for this reason the Abney, in the majority of cases, will give a preliminary line close enough to be adopted as the final location. The writer has used this level in laying out fully 75 miles of road in hilly and mountainous country.

Very few highway engineers seem to be familiar with the merits of the Abney as compared with the Locke level. It has all the advantages of the Locke and many additional uses.

Cost of Steam Tractor Hauling in Scotland.

Foreign wage scales are much lower than in this country especially for skilled labor. The effect of this on the cost of haulage of road stone is shown by the accompanying tables prepared by W. L. Gibson, county road surveyor of Perthshire County, Scotland, presented at a meeting of the Institution of Municipal and County Engineers.

It is said that "the bigger the load the cheaper the cost per ton-mile." If this be true, haulage by heavy traction engines is the cheapest known method of road transport so far as cost per ton-mile is concerned. Nevertheless, road surveyors must consider the question of damage to road by such haulage, and it is therefore necessary for them to keep the axle weights within reasonable limits. If heavy plant is used surveyors must make and maintain their roads to meet this class of traffic, which means that a large initial expenditure as well as a large annual cost would be required to meet the expense of road maintenance for their own traffic. If, on the other hand, light tractor haulage be employed there is a marked saving in the wear and tear to the road surface as contrasted with the heavier plant. We must, therefore, qualify our axiom that "the bigger the load the cheaper the cost per ton-mile" by this rider, "the heavier the axle weight the greater the damage to roads," with its corollary of increased cost for maintenance.

The question of easy working—*i. e.*, turning and shunting on narrow roads, and in quarries and loading banks of railway sidings—is an important consideration, for it must be remembered that in haulage of road material the primary object is to ply between the quarry or the railroad siding and the scene of steam road rolling operations without the intervention of roadside storing depots, and the inevitable and unnecessary cost of depositing and relifting the material.

Let us consider briefly team labor as regards cheapness of transport and damage to roads.

As far as the first is concerned, team labor cannot compete with tractor haulage. But the difference in wear and tear between light tractor haulage and team labor is a more problematical question, though my own experience goes to confirm me in the view that in some cases—if not in most—there is a difference here also in favor of the former. Indeed, I would go further and affirm that in some cases where the effect of the use of team labor was to cut through the surface and cause deep ruts, the substitution of tractors resulted merely in a depression, which was more easily remedied by making up with metal, and that continued use of the tractor and wagon over the metal in the course of haulage actually producing a much stronger road than before. It is naturally to the advantage of the carting contractor, when hauling under contract, to load to the full, but the greater the profit the greater the damage to the road. I am thus led to conclude that here again, though of course in a much less degree, the two aims of the road surveyor cannot be fully secured by the employment of team labor.

In considering as to the adoption of mechanical haulage for West Perthshire, I recognized that many of my roads were lightly constructed and narrow, and the quarries restricted in area and not easy of access. I considered that to adopt heavy traction engines would be almost impracticable for the work contemplated. Under the heavy motor-car regulations the use of self-contained vehicles such as steam and petrol wagons is permitted; but from the author's point of view they have the disadvantage of the great axle weight on the hind wheels, which are as a rule, in the case of the steam wagon, only about 3 ft. 6 in. diameter by 10 in. wide, carrying a legal weight—often, unfortunately, exceeded—of 8 tons.

This is quite an extravagant load for an ordinary country road. This type of vehicle has also another disadvantage to road authorities, in that the whole plant must remain idle while being loaded, whereas the tractor, using two tipping wagons, can be hauling the one while the other is loading. The output of this plant, therefore, is greater than that of a steam or gasoline wagon, and the produce of the crusher or tar-macadam mixer, or material

TABLE II.—ABSTRACT STATEMENT OF WORKING OF NO. 1 PIONEER TRACTOR—3 TONS—FOR 8½ YEARS ENDING MAY 15, 1913.

	Hauling Road Material.				Miscel. Works.							
Year.	No. of days at work hauling road material.	Quantity of road material hauled during year—Tons.	Average cost per day.	Average cost per ton mile in cts.	No. of days at work.	Quantity hauled during year—Tons.	Cost per day.	Av. quantity hauled per day, including road material and miscellaneous works—Tons.	Total distance traveled during year—Miles.	Average distance traveled per day—Miles.	Fuel consumed per day—Lbs.	Weather conditions.
1905*....	79	1,424	$5.88	11.5	..	...	.....	18.02	1,659	21.00	150	Fair
1906....	244	5,506	4.84	7.6	34	..	$3.00	22.56	5,940	20.62	150	Good
1907....	180	4,287	5.20	7.8	21	544	2.49	26.72	5,386	26.79	150	Excellent
1908....	202	4,491	5.56	10.3	31	280	3.58	20.48	5,713	24.51	175	Fair
1909....	226	3,955	5.64	14.6	7	112	3.29	17.45	3,794	16.28	225	Very wet
1910....	197	3,849	4.80	14.	13	148	3.30	19.03	3,659	17.42	200	Fair
1911....	234	4,516	5.30	13.8	..	..	...	19.29	6,268	26.78	275	Good
1912....	217	6,780	5.82	8.8	..	...	.....	31.24	5,098	23.49	300	Excellent
1913....	239	5,357	6.04	11.8	..	.	...	22.41	1,873	20.38	325	Very wet

*Half year only.

ANALYSIS—

No. of days at work in 8½ years	1,934
Quantity of material hauled in 8½ years	41,254 tons
Average quantity hauled per day	21.91 tons
Total distance traveled in 8½ years	42,390 miles
Average distance traveled per day	23.20 miles
*Average cost per day—hauling road material	$5.45
*Average cost per day—miscellaneous haulage	$3.45
Average cost per ton per mile, including filling into wagons, etc.	11.0 cts.
Average quantity of fuel consumed per day	225 lbs.
Average saving over team labor	44.92%

*Depreciation and tear and wear including at the rate of 15 per cent for first 7½ years and 22 per cent for last year.

SUMMARY—

Estimated saving in favor of steam tractor haulage	$11,500.00
Deduct initial cost of tractor and wagons	2,500.00
	$ 9,000.00
Add produce of sale of old plant	730.00
Total estimated saving	$ 9,730.00

TABLE I.—DETAILED STATEMENT OF WORKING EXPENSES OF PIONEER TRACTOR—3 TONS—FOR YEAR ENDING MAY 15, 1913; ALSO COMPARATIVE STATEMENT OF COST OF TEAM LABOR.

	Steam Tractor Haulage.					
Quarries, stations and depots, No.	Average distance material was hauled, miles	No. of days hauling.	Total distance traveled per day, miles.	Quantity of metal hauled, tons	Rate per ton	Carting rate. Team labor.
1	3½	13	21	289	$0.18	$0.74
2	3	73	18	1,647	.18	.42
3	3½	14	28	300	.19	74
4	¾	12	12	450	.11	.16
5	1	8	18	192	.17	.24
6	3	1	24	16	.25½	.64
7	¾	6	10½	202	.12	.16
8	3	10	24	170	.24	.64
9	2	62	20	1,491	.16	.42
10	3½	40	28	600	.27	...

Washing out boiler, cleaning and repairing engine and wagons, 66 days, at $1.14.
Filling metal into wagons at rate of $1.76 per day.
40 days 5 horses and carts at $10.80 per day.
Filling metal into carts at roadside depots for road rolling operations, 5,357 at $0.13 per ton.

ANALYSIS—

No. of days hauling	239
Material hauled	5,357 tons
Material hauled per day	22.4 tons
Total distance traveled in 239 days	4,873 miles
Average distance traveled per day	20.4 miles
*Average cost per day	$6.20
Average cost per ton per mile	$0.12
Fuel consumed per day	300 lbs.

*This includes filling wagons, time of driver washing out boiler, cleaning and repairs, with depreciation, etc., estimated at the rate of 22 per cent.

SAVING ON SEASON'S WORK—

		In per cent
Cost of carting by team labor	$3,180.00	100
Cost of steam tractor haulage	1,460.00	46
Total saving†	$1,720.00	54

†In addition, the tractor (after 8½ years' work) was sold for $740.

from the railway siding, is delivered on the road in a more regular manner. While the higher speed—legal maximum of 12 miles per hour—of motor wagons on rubber tires may be considered an advantage for this type, in the present state of development of mechanical haulage it appears to me that the cost of rubber tires alone, which could be calculated at not less than 4 cts. per vehicle mile, and the cost of gasoline where motor engines are used, make it difficult for this type to be compared with tractor haulage for the conveyance of road material irrespective of the disadvantages already mentioned of the self-contained unit.

Notwithstanding rubber tires, the axle weight of this self-contained vehicle has proved to be very damaging to road surfaces where regular services of motor 'buses, heavy vans and chars-à-banc are in operation. It must, of course, be understood that where the roads are strong and smooth, and where facilities are provided for quick and cheap loading, the use of the fast-running motor wagon—either steam or gasoline—may be seriously considered; but the author has yet to learn of instances in any way similar to his own work where this type of vehicle has been used at a cost which can be compared with his own experience.

The following points may be suggested for the consideration of road engineers and surveyors who may be contemplating the adoption of mechanical haulage:

(1) Strength of Roads.—Whether capable of carrying the maximum axle weight of the type of vehicle it is proposed to use.

(2) Quantity of Material to be Transported.—If the tonnage is small, or the distances unduly short, hire may be cheaper than purchase.

(3) Present Cost of Transport.—There are still some districts where I understand the prices of cartage are still so cheap that purchase of plant may not at present be justified.

(4) Whether the work of the district suits the use of convertible tractor and roller. (The author finds the convertible tractor the most useful plant he has.)

(5) Whether the engine can be utilized for driving small crusher for tar-macadam mixer, supplying steam for rock drilling, pumping water out of quarries, removing accumulations of road scrapings, hauling tar-sprayers and boilers, or any of the many operations for which a surveyor requires power of any kind.

The author wishes it to be understood, lest he should be taken to task by motor haulage experts in general, that he fully realizes that other kinds of traffic may be better undertaken by other types of vehicles than that which he advocates for the haulage of road material.

Details of the actual work and cost of operation of the two first tractor plants purchased by my council are annexed. The third engine replacing 3-ton Pioneer tractor sold last year, has been in use less than one year, and little or no expense for repairs has yet been necessary.

Tables Nos. 1 and 3 are given as examples of one year's work, and the method of keeping the cost.

Tables Nos. 2 and 4 show abstracts and analyses for the whole of the years during which the tractors have been at work.

A comparison between the cost of team labor and tractor haulage is shown in each statement.

TABLE III. — DETAILED STATEMENT OF WORKING EXPENSES OF ROB ROY TRACTOR—5 TONS CONVERTIBLE TO ROLLER —7 TONS—FOR YEAR ENDING MAY 15, 1914; ALSO COMPARATIVE STATEMENT OF COST OF TEAM LABOR.

Quarries, stations and depots. No.	Av. distance material was hauled—Miles.	No. of days hauling.	Total distance traveled per day—Miles.	Quantity of metal hauled—Tons.	Rate in cts.	Carting rate with team, in cents.
1	1½	30	15	676	16½	$0.28
2	1½	17	15	150	14	.23
3	4	28	24	615	16½	.54
4	3	29	32	515	21	.64
5	1½	10	18	278	13½	.28
6	2	10	24	284	13	.43
7	3	5	24	100	13½	.64
8	3½	13	21	240	20	

Filling into wagons, allow at rate of $1.76 per day.

Washing out and cleaning and repairing engine and wagons—Time of driver, 30 days $1.22.

13 days, 6 carts and horses at $12.96 per day.

Add filling and carting metal from roadside depots to rolling operations, 3,158 tons at $0.13.

ANALYSIS—

Number of days' hauling	142
Quantity of material hauled	3,158 tons
Average quantity hauled per day	22.23 tons
Average distance traveled per day	21.95 miles
Total distance traveled in 142 days	3,118 miles
Average cost per day, including filling into wagons and time of driver, washing out boiler, cleaning and repairing (Depreciation and tear and wear included at the rate of 15%.)	$5.76
Average cost per ton mile	$0.103
Fuel consumed per day	200 lbs.

SAVING ON SEASON'S WORK—

Cost of carting by old contract system	$2,250.00
Cost of motor haulage	820.00
Total saving	$1,430.00

Note.—This tractor, being convertible to a 7-ton road roller, was engaged at rolling operations for 141 days at an average cost of $4.41 per day.

TABLE IV.—ABSTRACT STATEMENT OF WORKING OF NO. 2 ROB ROY TRACTOR—5 TONS —CONVERTIBLE TO ROLLER—7 TONS—FOR 6 YEARS ENDING MAY 15, 1914.

	Hauling Road Material.				*						
Year.	No. of days at work hauling road material —Tons.	Quantity of road material hauled during year—Tons.	Average cost per day.	Cost per ton mile in cents.	No. of days at work.	Cost per day.	Av. quantity hauled per day, including road material and miscellaneous works—Tons.	Total distance traveled during year—Miles.	Av. distance traveled per day—Miles.	Fuel consumed per day —Lbs.	Weather conditions.
					Mis. wks.						
1909	151	3,739	$5.24	12.00	12	$3.72	25.00	3,098	19.00	400	Very wet
					Rolling						
1910	99	2,302	6.54	8.28	118	4.40	18.00	2,945	29.74	400	Fair
					Rolling						
1911	145	2,888	5.71	8.00	116	4.50	19.91	3,899	26.88	350	Good
					Rolling						
1912	103	3,318	6.52	10.40	146	4.40	33.00	2,514	24.40	325	Excellent
					Rolling						
1913	121	2,617	6.04	9.16	162	4.41	21.62	2,682	22.16	350	Very wet
					Rolling						
1914	142	3,158	5.76	10.28	141	4.41	22.23	3,118	21.95	300	Excellent

*Engaged as roller at miscellaneous works.

ANALYSIS—

No. of days at work hauling in 6 years	773 days
No. of days rolling in 6 years	683 days
Quantity of material hauled in 6 years	18,262 tons
Average quantity hauled per day	23.62 tons
Total distance traveled in 6 years	18,256 miles
Average distance traveled per day	24.02 miles
Average cost per day hauling (depreciation and tear and wear included at the rate of 15%)	$5.77
Average cost per day rolling	$4.50
Average cost per ton per mile, including filling into wagons	$0.097
Average quantity of fuel consumed per day	350 lbs.
Average saving over team labor	42.13%

SUMMARY—

Saving in favor of steam tractor haulage	$6,500.00
Deduct initial cost of tractor wagons and rolling gear	3,640.00
	$2,860.00
Add present value of tractor wagons and rolling gear	1,820.00
Total estimated saving	$4,680.00

Note.—This engine has also been employed at rolling operations for 683 days during the past 6 years at an average cost of $4.42 per day. The value of this work is not included in the estimated saving.

Standard Method of Monumenting Streets at St. Paul, Minn.

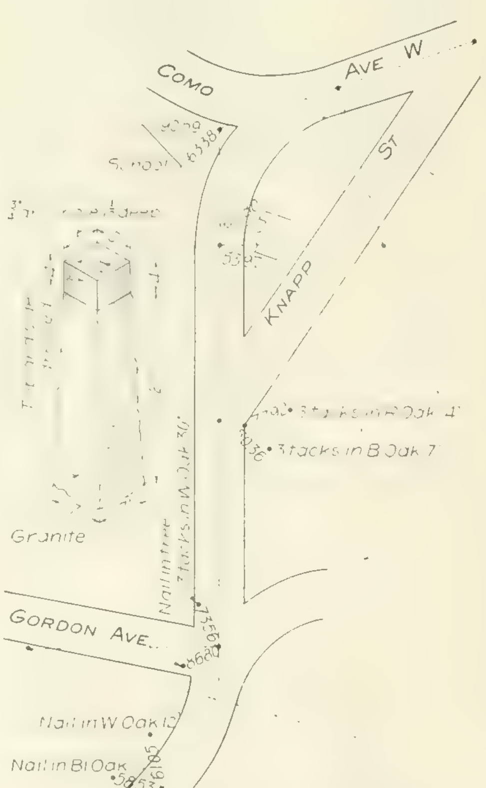


Fig. 1. Standard Method of Street Monumenting Used in St. Paul, Minn.

The method of monumenting all improved streets at St. Paul, Minn., is illustrated by the accompanying sketch redrawn from data given in a recent report of the Commissioner of Public Works.

For a great many years a monument atlas was kept up, it being necessary to refer to this atlas when monuments in any particular locality were required. Monuments were numbered and the number referred to the page of the monument record book on which were given the notes and ties required. This monument record, for some reason, fell into disuse and it has also been found that the records on the standard maps for the last few years are not complete. To remedy this a card index monument record has been started with the intention of bringing up to date all records of monuments in the city. These cards are indexed under the street name. There are three or more intersections to each card and the cards are numbered and run consecutively on each street from the center of the city outward.

Specifications in all contracts for city work have the following clause: "The contractor shall not disturb any monuments or stakes found on the line of the improvement until ordered by the engineer and shall reset any monument as directed by the engineer. A penalty of $25 may be imposed for each monument disturbed without orders and the amount deducted from the estimate."

Notes on Roads and Streets.

Length of Guarantee on Asphalt Pavements.—During 1913 the General Asphalt Company constructed 1,480,501 sq. yds. of asphalt pavement, with an average guarantee of 2.45 years, as shown in the following tables:

LENGTH OF GUARANTEE BY YEARS.

Year laid.	Average length of guarantee Years.	Percentage with no guarantee.
1905	7.2	6.4
1906	7.0	11.1
1907	5.0	27.0
1908	5.2	26.7
1909	3.5	36.2
1910	3.1	40.0
1911	2.99	35.3
1912	2.79	33.7
1913	2.45	51.6

YARDAGE AND LENGTH OF GUARANTEE OF PAVEMENTS CONSTRUCTED IN 1913.

Term.	Yards constructed with guarantee in 1913.
1-year	674
2-year	......
3-year	398
5-year	725,229
7-year	...
8-year	
9-year	
10-year	
15-year	
Total	726,301
Without guarantee	754,200
Total	1,480,501

Control of Excavations on Paved Streets in Philadelphia.—A filing system showing by a "visible" index all streets under guarantee, and other pertinent data, and a follow up system are used in Philadelphia in maintaining control over permits until the completion of the work and the final restoration of the pavements, or the removal of building material occupying highways. After a permit to open a street is issued, a notice of such is sent immediately to the engineer in charge of the district named in the permit, with the request that it be returned to the central office when the pavement has been restored. On the face of this return notice a place is provided for both the approval of the district engineer, and an endorsement by the inspector in charge of the field work, that the pavement has been replaced in a satisfactory manner. All permits are valid for a period of two weeks from the date of issue, and the authorized opening is required to be refilled and repaved before the permit expires.

Results of Clean-Up Week in Philadelphia.—The week of April 28 to May 3, was set aside in Philadelphia as a special "Clean Up" week, to better the health conditions and appearance of the city. In order to bring it to the attention of property holders, the plan was advertised as widely as possible, both through the daily papers and by pamphlets distributed to householders. In addition, placards were placed in store windows, trolley cars, and railroad stations, calling attention to this movement. Every possible means of obtaining active co-operation was used, the civic organizations, public schools, boy scouts, all enlisted their services in this work. During "Clean Up" week the amount of rubbish collected by the city amounted to 34,279 cu. yds.

BOOK REVIEWS

Masonry. By George R. Barham. Longmans, Green & Co., New York. Cloth, 4⅞x7¼ ins.; 181 pp.; illustrated; $0.90.

This is an English book written for students in trade schools and for apprentices. The headings of the seven chapters are: Tools and Machinery; Conversion of Stone; Drawing and Setting Out; Construction and Constructional Details; Erection and Fixing; Materials—Building Stone; and Stereotomy. The book is of most value to masons and to students of architecture.

Rivers and Estuaries.—Or Streams and Tides.—By W. Henry Hunter,, New York, 1913. Longmans, Green & Co. Cloth, 6x9 ins.; pp. 69; illustrated; $1.

This is a brief and very general but very interesting discussion of river and tide flow phenomena and of the doctrine of estuarial river regulation. Mr. Hunter styles his remarks extenuating, and they are so in the respect that they discuss clearly the elemental or fundamental phenomena and principles of current and tide activities. To do this is all the author had in mind. Like most English engineers of his generation he can write English, and this lends attraction to his discussion. One need not be an engineer to read and understand almost every chapter. One wishes that more American engineers who have come to years of counsel would write similarly.

Designing and Detailing of Simple Steel Structures.—By Clyde T. Morris, McGraw-Hill Book Co., New York. Cloth, 6x9 ins.; 249 pp.; illustrated; $2.25.

In this, the third edition of the book the author has changed somewhat the arrangement of the text. Several of the chapters have been rewritten, and the entire book has been reset and brought up to date. A new chapter on highway bridges has been added, together with a reprint of the Specifications for Steel Highway Bridges of the State Highway Department of Ohio. Many of the illustrations have been redrawn and several have been added. The book treats of the designing and detailing of simple steel structures, and is somewhat elementary. It was the intention of the author to present such material as can be covered in the limited time usually given to this subject in engineering schools. The book is clearly written, but is not well balanced. Of the 249 pages of text, 15 are given to designing and estimating, 29 to riveting, 27 to mill buildings, 110 to railway bridges, 10 to highway bridges, 33 to manufacture and erection, and 25 to steel highway bridge specifications.

Laboratory Manual of Testing Materials.—By William Kendrick Hall and H. H. Schofield. McGraw-Hill Book Co., New York. Cloth, 5x 7⅜ ins.; 135 pp.; illustrated; $1.25.

The authors state the manual is an outcome of the operation, through 18 years, of the Laboratory of Testing Materials of Purdue University and that one purpose of it is to relieve the instructor from the necessity of explaining the details of mechanical procedure.

The headings of the seven chapters and the four appendices are: General; General Instructions; Definitions; Materials Stressed Beyond the Elastic Limit; Testing and Testing Machines; List of Experiments; Instructions for Performing Experiments; Common Formulae; Strength Specifications for Steel and Iron; Standard Forms of Test Pieces.

The text is well arranged and the principal types of machines and testing instruments are well illustrated by numerous drawings. Although the manual was written primarily for students at Purdue, the list of experiments given, with slight modifications, can be performed in other well equipped laboratories. Much of the information will be found of value to the practitioner.

Sub-Aqueous Foundations.—By Charles Evan Fowler. John Wiley & Sons, New York. Cloth, 6x9 ins.; 794 pp.; illustrated; $7.50.

This is the third edition of the book, the first appearing in 1898 and the second in 1904. In the present edition, which comprises 735 pages of text and 59 pages of selections from specifications, the author has collected a large number of valuable data on foundation work, much of which can be easily applied by the practicing engineer. There is included in the present edition sub-aqueous foundations, the cofferdam process for piers, and dredges and dredging.

The new matter consists largely of practical examples of work, and of descriptions and working plans of different kinds of construction plant. New data have been added on pile-driving, on the use of concrete piles and metal sheet-piling, and a chapter has been added describing methods and equipment for the jetting of piles. Additional data are also given on the use of compressed air in caisson work, and on the bearing power of piles. The calculation of piers, retaining walls and footings has been added, as has also a chapter on cement and concrete. Entirely new matter has been given on piers and wharves; on dams, seawalls and retaining walls; on dry docks and locks; and on the cost of foundation work. The present treatise should prove of particular value to engineers engaged in foundation work for bridges.

Field Manual for Plane Surveying and Railroad Curves.—By R. C. Yeoman and E. A. Tucker. M. E. Bogarte Book Co., Valparaiso, Ind. Cloth, 5x8 ins.; 150 pp.; illustrated; $1.75.

This book combines the features of the students surveying manual and the ordinary field book for recording the data obtained in making transit surveys. It is a product of the teaching staff of the Department of Engineering of Valparaiso University. It contains sample field notes of the form usually recorded in connection with students surveying exercises. These sample field notes are followed in each case by blank pages, ruled up as in the ordinary field book, for recording the notes taken by the student in solving problems similar to those shown as examples.

The book illustrates the solution and note keeping for seven groups of problems as follows: Pacing problems, chaining problems, leveling problems, compass and sextant, transit problems, astronomical surveying and railroad curves. The book will be used in the authors' classes as supplementary to lectures and assigned reading in a standard text book on plane surveying.

The book promises to make better note keepers out of students and if it fulfills this promise it will be well worth while. Its use will be restricted to academic work. Combining the features it does it effects a saving in the student's former book bill at Valparaiso University.

Rainfall Reservoirs and Water Supply.—By Sir Alexander R. Binnie. D. Van Nostrand Co., New York City. Cloth, 6x9 ins.; 150 pp.; illustrated; $3.00.

This is not a comprehensive treatise on water supply engineering but a monograph on the design and construction of reservoirs for impounding stream flow, with additional matter covering the conveyance of the water to the point of consumption including the description of the necessary conveying, distributing and controlling works.

The book contains ten chapters of the following scope: A study of rainfall data, including average rainfall and fluctuations in the amount, run-off and evaporation and intensity of flood flows; consumption statistics, quality of raw water and its purification; sources of supply, gravity and pumping systems of supply, pumping works, catchment areas, deductions from rainfall and compensation for water used; capacity and site of reservoirs, puddle and concrete core walls and embankment base; reservoir embankments and masonry dams; works for drawing off water from reservoirs; waste works; aqueducts, service reservoirs and distribution system; valves, meters and house fittings.

This work is the outcome of the author's lectures to the students of London University but it would be grossly unjust to brand it as a book solely for the student. It contains much matter of interest to the water works engineer, particularly in the discussion of rainfall data and in the discussions on the construction of reservoir embankments and dams.

Foundations of Bridges and Buildings.—By Henry S. Jacoby and Roland P. Davis. McGraw-Hill Book Co., New York. Cloth, 6x9 ins.; 597 pp.; illustrated; $5.00.

The authors state that their aim has been to treat in a systematic manner the entire subject of foundations for bridges and buildings as represented by American practice. The 19 chapter headings and the space devoted to each of the subjects covered are: Timber Piles and Drivers, 36 pp.; Driving Timber Piles, 38 pp.; Bearing Power of Piles, 41 pp.; Concrete Piles, 58 pp.; Metal and Sheet Piles, 24 pp.; Cofferdams, 41 pp.; Box and Open Caissons, 41 pp.; Pneumatic Caissons for Bridges, 29 pp.; Pneumatic Caissons for Bridges (Cont.), 29 pp.; Pneumatic Caissons for Buildings, 28 pp.; Pier Foundations in Open Wells, 18 pp.; Ordinary Bridge Piers, 33 pp.; Cylinder and Pivot Piers, 16 pp.; Bridge Abutments, 19 pp.; Spread Foundations, 38 pp.; Underpinning Buildings, 28 pp.; Explorations and Unit Loads, 20 pp.; Pneumatic Caisson Practice, 24 pp.; and References to Engineering Literature, 36 pp. The text is followed by a brief index.

As regards the space given to the various subjects the book is not well balanced, but the text is well arranged. Numerous working drawings and construction views are given. The compiled data reflect modern practice and the book should prove of value to engineers engaged in foundation design and construction. The complete list of references to the engineering literature on the subject of foundations is a valuable feature of the book.

Suspension Bridges, Arch Ribs and Cantilevers.—By Wm. H. Burr. John Wiley & Sons, New York. Cloth, 6x9 ins.; 414 pp.; illustrated; $5.00.

It is stated in the preface that the book was written primarily to meet the author's needs in the class-room, although the tabular data should also prove of value to the practitioner. The arrangement of the book is logical and the text is clearly presented.

The scope of the book may be seen from the following brief summary: Chapter I gives a discussion of the suspension bridge, based on the assumption that the problems treated are statically determinate, the principles developed being sufficiently close to serve the purpose of approximate computations and preliminary estimates. Chapter II considers the theory of the stiffening truss with a center hinge, which makes the problems discussed statically determinate. Chapter III develops the theory of a perfectly flexible cable loaded vertically. In Chapter IV there is developed a more closely approximate theory of the stiffening truss than that previously given, based on the elastic deformation of the structure; while Chapter V considers the theory of the straight stiffening truss, based on the method of deflections. In both of these theories it is assumed that there is no sensible variation from the parabolic form of cable, or frame, under any condition of loading. The theory given in Chapters III and IV follows closely that developed by Professor Melan. Chapter VI treats of the temperature stresses in the stiffened suspension bridges; Chapter VII gives a graphic treatment of the arch rib, and Chapter VIII develops the general theory of the elastic arch according to the law of "Least Work." Chapter IX treats of the three-hinged arch rib; Chapter X of the braced spandrel arch, and Chapter XI gives a short analytic treatment of cantilevers. Two short appendices are added, one of which considers briefly the limiting spans and depths of stiffening trusses, and the other develops some formulas for reinforced concrete beams.

Theory of Arches and Suspension Bridges.—By J. Melan. Translated by D. B. Steinman, Chicago. Myron C. Clark Publishing Co. Cloth, 6x9 ins.; pp. 303; illustrated. Reviewed by H. G. Tyrrell.

Professor Melan's discussion of arches and suspension bridges, as it appeared several years ago in the "Handbuch der Ingenieurwissenschaften," is a very valuable reference work for bridge engineers, and it is gratifying to know that some one has taken upon himself the task of translating it into English.

The latter part of the book dealing with actual construction is, of course, the most interesting and valuable to practicing engineers, but the first part dealing with theory and the methods of stress computation, a translation of which is presented by Professor Steinman, is an essential part of the whole and will be valuable to students and to those who are engaged in the routine duties of computations, after the principal parts of a design have previously been determined.

While the solution of stresses in the cables of a symmetrical suspension bridge is one of the very simplest mathematical problems, it is interesting to see how many and varied complications can be discovered or imagined, and then with what dexterity the mystic stresses can be computed.

Like many other works written by the masters of engineering in Europe, this book covers the whole subject in a thorough manner. The fine plates accompanying Professor Melan's complete discussion, and the many diagrams throughout the text, offer abundant suggestion and will doubtless assist in leading the designer out into a greater degree of originality than is usually displayed in America, where bridge shop standards have, unfortunately predominated.

This book gives considerable space to the investigation of conditions in suspension bridges with several spans and with towers at different elevations, and as illustration, it gives the Easton suspension over the Lehigh River, described in Engineering News, Nov. 22, 1900. It is, of course, gratifying to American engineers, to see that this German book uses as its principal illustrations, those masterpieces of bridge engineering prepared by Mr. Lindenthal for crossing the North River at New York, and the St. Lawrence River at Quebec.

As would be expected, the bibliography is compiled chiefly from the literature of Europe, but it is, perhaps, all the more valuable to American engineers on that account, since some of our own books give references to the most important articles on these subjects in America.

Since Dr. Steinman is himself, an authority on arches and suspensions, his translation of Professor Melan's book will be of greater value. The book is clearly printed on good paper and well bound, and on the whole is an example of the best and most modern practice in American engineering bookmaking.

Good Roads Year Book, 1914.—By J. E. Pennypacker, Editor American Highway Association. Colorado Building, Washington, D. C. Cloth, 6x9 ins.; pp. 500; $1.00.

The book, which is the official year book of the American Highway Association, contains a wealth of complete and reliable information concerning the administration of highway affairs and progress made in road construction. Among the subjects treated are a history of road building, road systems in foreign countries, state aid legislation, bond issue legislation, convict labor laws, automobile regulations, types of road bridges and culverts, highway officials, road expenditures, bibliography of road literature, books and periodicals, highway educational facilities, road associations, manufacturers of materials and machinery employed in road work.

The information presented is authoritative in that it has been reviewed prior to publication by many officials concerned. The book is well indexed and will be found very useful for reference purposes by numerous executives and manufacturers.

Railway Track Hand-Book.—By Bruce V. Crandall. Spencer Otis Company, Chicago. Leather, 4x6½ ins.; 124 pp.; illustrated; $2.50.

This book contains a great deal of useful information concerning railway track, track materials and track work well arranged in a durably bound book of convenient size for the pocket. Quite a large portion of the data given, especially the diagrams for obtaining costs, is original, and it is believed that no one book contains all of the other information furnished in this volume. One of the purposes to which the book is especially adapted is the preparation of estimates of costs of railways, railway track material and railway track work. A very complete blank form for cost estimating which occupies five pages of the book provides blank spaces for the numerous items of material and labor required for constructing a railway from the first stake to the first train. Reference to this table when preparing estimates should prevent the omission of items, a matter of considerable importance. A table of current prices of new and second-hand track and miscellaneous materials for the years 1911 and 1913 with blank spaces for future years or for a certain railway is furnished to assist in preparing estimates. Another table gives the estimated costs of ballast for a mile of main track for various kinds of ballast at various prices and for various quantities per mile. A similar table gives the estimated costs for various weights of rail for a mile of track at various prices. Tables for ties, tie fastenings and joint fastenings of a similar nature, are included. A rail table giving the weight of rail required to lay one mile of track, 100 ft. of track, distance in miles or feet laid by one ton, and weight of one 30-ft. and one 33-ft. rail of practically all weights of rail in use on commercial roads, ranging from 30 lbs. to 135 lbs. per lineal yard should save much calculation. Very complete tables of weights and amounts of splice bars, track bolts, hexagon and square nuts, spikes of all kinds and weights of various substances are furnished. These tables are often quite incomplete in books on this subject.

Seven diagrams or charts show the cost of rail laid and relieved; cost of joints laid and relieved; net cost of rail relaid; and cost of spikes for relaying rail. Several pages of blank forms are provided for future or special costs for joints and renewal materials. A comparative table of the dimensions and properties of standard rail sections of the A. S. C. E., A. R. A., P. S.-Penn., P. R. R., Gt. Northern, U. P., C. & N. W., and C., B. & Q. types is one of the useful features of the book.

An interesting summation of the processes which result in various descriptions of products from iron ore to finished forms by Mr. Eliot A. Kebler, resident agent at Pittsburgh for M. A. Hanna & Co., which indicates the line of processes involved in the manufacture of iron and steel is followed by the chemical analyses and specifications for iron ore, pig iron, semi-finished steel and finished materials, including the products of iron and steel foundries. A table of approximate freight rates on rail and fastenings and structural steel from Pittsburgh, Chicago and Birmingham to the various railroad centers of the United States is given so that item of cost may be estimated.

Special information on common and screw spikes, tie plate, track bolts and nut-locks with illustrations and descriptions of those materials as manufactured by the Spencer Otis Company, is given along with the statements of advantages and economies claimed by the makers to be obtained through the use of their products. A good index completes the book which contains sufficient information of permanent value to recommend it as a supplementary text book for use in technical schools where instruction in railway engineering is given.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLII. CHICAGO, ILL., JULY 15, 1914. Number 3.

Hard Times and Hard Roads.

In the last few months work has been advertised and bids received for road contracts aggregating many millions of dollars. This work is widely distributed and varies in type from graded earth road construction to concrete and brick road building. Contracts are of all sizes to suit the capital and equipment of both large and small contractors. And yet for many of these contracts there have been no bidders and, with a few exceptions, little competition on the remainder. Judging from the dearth of bidders road contractors are all busy, or there are an insufficient number of contractors engaged in this class of work. Is either of these suppositions true? As a consequence of the so-called business depression which has existed for the past two months this work should not have gone begging. In the face of this at a recent letting of 19 contracts in Illinois there was an average of one bidder to the contract. For a number of contracts in Ohio there were no bidders, and in Maine the same condition existed. An unusually large contract in Texas was readvertised to secure bidders. New York appears to be the only state in which there seems to be much competition at the present time. This rather general lack of competition would indicate the reverse of hard times conditions.

It is quite probable that contractors have not appreciated the opportunities which the road contracting field offers. In former years country road contracts were small and long drawn out and the work was performed under political conditions of supervision distasteful to a reputable contractor. Within the last few years the average size of contracts has increased greatly and the work is almost invariably supervised by well-paid, reputable engineers. The increased use of higher types of surfacing, such as bituminous and Portland cement concrete and brick, have opened the field of country road contracting to the man formerly engaged principally in city paving work. Only in a few cases, however, have these contractors taken proper advantage of their opportunities.

Several years were required in New York before a sufficient number of road contractors were available to care for the work of that state. In Wisconsin last year most of the road construction work was accomplished with hired labor, although a large portion of it would have been contracted had a sufficient number of bids been received. There appears to be throughout the country a lack of aggresiveness in going after this work, which is unusual and which must be largely due to its newness and to the inexperience of general contractors in handling road construction.

If a general contractor desires to enter this field no better opportunity will ever exist than at the present time. Contracts both large and small are plentiful, as a rule labor is available, and interest rates are low in the midst of hard times. It looks like a good opportunity.

A Neglected Field of Engineering Authorship.

In the later years of his life George S. Morison published an essay which he entitled "The New Epoch." Mr. Morison's previous engineering writings had been extensive. His reports on the design and construction of great bridges built by him and his papers on dams and suspension spans are notable—have indeed become almost classic. We think, however, that of his writings time will rank as most distinctive, the one whose lessons are of broadest worth, the little volume that he wrote late in life. Mr. Morison offered this book almost with diffidence, as if he felt apology to be due from an engineer for permitting his pen to wander so far from the concrete problems of his profession. In this respect he reflected a feeling which is both a virtue and a fault of engineers.

Of a dozen engineering books which have come to us within a few weeks only one has been *read*. The eleven were studied; some of them were studied very carefully because it was the editor's business to know them; one only was *read* and this was the one written by an engineer of many years, to tell, as he said, the young engineer those things about tides and currents which would have been of lifelong service to himself had he heard and learned them in "the remote period when he too was young." The author claimed no purpose except to remind, for, he says, while "to youth pertains the inheritance of the ages, to youth also pertains a portentous faculty for forgetting."

Our most remote object is to exaggerate the value of this small volume of example and counsel, but the thought persists that such books as it and Morison's "New Epoch" are of the kind of writings needed to give engineering a literature, as, for example, the legal profession has a literature. Law history, biographies of great jurists, and the commentaries on justice and equity by the master practitioners of law give the lawyer a real literature of his profession. His working volumes are the bound statutes and the various manuals of legal procedure; they are a vital part of his literature but alone they do not constitute a literature. It is so of the literature of architecture and of every vocation or profession having a real literature of its own.

Many engineers write books. One often thinks that there are too many engineering books. Manuals of engineering, treatises, pocketbooks, textbooks multiply each year. There are a number of great books on engineering. Weisbach, Rankine, Wellington and other men whose names come readily to mind were great engineering writers. But all these books, or at least all but a few, are of one kind. They are books of analysis and exposition. Most of them are books of technique of designing and constructing. They are young men's books, and this is well, for young men, because the details of designing and constructing processes constitute their work, are the ones most apt in the technique of processes. Now whether one may think, as was said a moment ago, that there are too many of these books, it is, we think, certain that there are too few books on the broader problems of engineering as distinguished from the minor problems of technical processes of engineering.

The point may be made clearer perhaps by being more specific. There are few books on problems of engineering planning, for example, on the problems to be solved, not in devising processes for tunneling a ridge for railway, but in determining whether any tunnel at all is the better crossing, and if it be, which are the best elevation and length for and the best size and form of the tunnel to be built. One has to look through the books on tunneling to understand fully how little there is in them which helps to solve these larger problems mentioned, and of this little how much there is not coordinating with modern conditions. What we find in this respect to be true of tunnels is in the same respect, though perhaps not always in the same degree, true of bridges, of river improvement, of power development, of industrial plants, of water supply, of waste disposal and of engineering construction generally .

Some 30 years ago Wellington wrote a book on the Economics of Railway Location. This book has become classic; it yet stands alone in its field of railway engineering, in the breadth of its conception and discussion of problems of location. With the additional data that 30 years have made available, Wellington would today write many parts of his "Economics" differently, but it was then a great book and is yet great. River improvement, water supply, sanitation, road building, have similar large economic problems and no one, as Wellington did for railway location, has written a comprehensive study of these problems. Engineering literature lacks sadly broad treatises on economics of engineering planning.

No history of engineering development or of the development of a specific branch of engineering is easy to name. One calls to mind one or two, but stretched to the utmost the list is short. Engineering has a notable history, one as interesting in a different way as the history of architecture, but engineering has no James Fergusson.

In engineering biography there has been little since Samuel Smiles wrote, and the "Lives" of great engineers was compiled early in the last century. Memoirs of engineers are fewer. And so the record reads in engineering all through the kinds of literature not strictly and specifically the working literature of the active practitioner.

The working literature of any profession—the manuals of practice—is specifically the young man's task. In engineering he has fulfilled it well on the whole. The task of engineering writing that has not been performed is that of producing books on engineering planning and economics and on engineering history and biography, books of commentary and synthesis. Such books, except rarely, are not within the capabilities of the young engineer—he wants the perspective of years and wide experience necessary for appraising and apportioning facts and opinions. He is producing and the stress of production gives scant time from the work immediately in hand. History, biography, commentary, economics of engineering planning are the tasks of authorship of the old engineer. He should write books. The treatises, the textbooks, the manuals that line his shelves are a part of great value of the literature of the engineer, but they do not make a literature of engineering. Books of another kind are wanted to complete the records of engineering.

One has asked: who will publish these books of which you are speaking? And who, when they are published, will buy them? Frankly, we are doubtful if there will be very eager publishers or purchasers. The engineer who reads is not remarkably common. He buys books but they are chiefly such books as he can use directly in solving his immediate problem. We feel that the writers of such books as we have asked to be written must perform their labor largely for love. And truly we don't see clearly how the publisher can escape the same necessity.

The Iowa Highway System.

Perhaps the most interesting feature of the Iowa road system is its thoroughly American organization. The organization is original and, so far, appears to meet the requirements of that state. In brief, the county is an independent unit both for construction and maintenance work with a separately organized township unit for maintenance purposes. The State Highway Commission is relieved of the details connected with this work other than furnishing general directions. At the same time, it has power to enforce its regulations; summarily, where the county engineer is at fault, and through the attorney general of the state where an elective officer is remiss in his duties. The organization of the commission is compact, complete and flexible. In many respects it is similar to that of New Jersey, except that the county unit is more clearly defined. The system is worthy of study since the experience and mistakes of other states were available to the framers of the legislation which brought it into being.

A point of fundamental difference from older state organizations exists in the provision for maintenance. The road law requires that all roads shall be maintained, and makes definite provision therefor. In this respect the law is admirable.

There is no question but that in their haste for better roads many states have overshot the mark. They have constructed expensive roads without adequate provisions for maintenance. Existing roads have often been neglected with the idea of completely rebuilding them at some future time. We believe that the old neglected roads are sometimes easier to travel than the new expensive roads which are in a poor state of repair. It would seem, in the light of our present knowledge, to be folly to construct an expensive road surface without providing definitely for its maintenance and renewal.

In undertaking its work the Iowa Highway Commission has attempted to improve general road conditions and effect economies in present methods of administration and construction rather than proceed with the construction of roads radically different than those in general use. In this connection, the work of the bridge department, for which Prof. J. E. Kirkham of Iowa State College has been consulting engineer, has produced marked results. The methods of county bridge construction in Iowa, as in many other states, were woefully inefficient. No greater economy could have been effected than the reform of this work.

We wish to call attention to the spirit of co-operation which exists between the Iowa Highway Commission and the people of the state. This spirit, fostered by a wise choice of executives, has, no doubt, been an important factor in the character and cost of the work already accomplished and, if continued, will unquestionably lead to economy and efficiency in future road and bridge construction.

The Determinate Causes of Concrete Failures.

In a paper presented before the annual meeting of the American Society for Testing Materials, Mr. R. S. Greenman discussed the causes of concrete failures. As a basis for discussion he proportioned the reasons for poor concrete "as being 90 per cent due to poor workmanship, 8 per cent due to poor aggregates, and 2 per cent to poor cement." He stated further that "these reasons and the additional one of the influence of the water used are responsible for most failures in concrete." In our brief discussion we shall consider the material only—not the design of concrete construction.

From the nature of the subject under discussion it is obvious that it is impossible to set exact percentages for each contributing cause to failure which will hold true for concrete work in general, yet approximate percentages may prove useful as a basis for determining the cause of any specific failure. In fixing the percentage due to poor workmanship this value should undoubtedly be high, yet it is difficult to define what is meant by "poor workmanship" and what constitutes "good workmanship." What may be considered good workmanship for one type of structure (because it results in safe and economic construction) may not be satisfactory for a different structure. It is undoubtedly true that for most structures sufficient care has not been taken in the making and placing of concrete. When the essential element "time" is considered it is probably true that concrete has not been given a fair trial. Concrete is usually made and placed at the site, yet these two distinct operations are made to compete with the erection, only, of other types of construction. Due to this fact it has been found impracticable to give concrete construction adequate inspection from start to finish. If sufficient time and care were given to the competent inspection of concrete structures the 90 per cent weight given to "poor workmanship" would be much too high.

Under present conditions, the 8 per cent rating given to poor aggregates is probably too low. We are just beginning to realize the great difference that exists in the aggregates which ordinarily are available for concrete. As a rule, insufficient care is given to the inspection and testing of sand, broken stone and gravel. Unfortunately we still find specifications which merely require that the sand shall be "sharp" and "clean," yet tests have shown that "sharpness" is not essential and that a small percentage of foreign matter is not particularly objectionable, provided it does not form a coating on the grains. More care is needed in the inspection of broken stone and gravel, while standard methods of testing these materials are needed.

Although a careful examination of failures will usually determine the cause of failure, what is more generally needed is a better appreciation of the good results which will follow from increased attention to the selection and proportioning of concrete mixtures and to the mixing and placing of concrete. We believe that the lessons to be drawn from a study of existing failures will emphasize, mainly, the importance of factors which are (or should be) commonly understood by those now in charge of concrete construction. For example, the continued practice of specifying a definite concrete mixture, without first determining the properties of the aggregates and the proportions which will give a mixture of maximum density, is not justified by facts.

WATER WORKS

Experimental Data on the Removal of Carbonic Acid from Well Water Supply at Lowell, Mass.

The city of Lowell, Mass., draws its public water supply from two gathering grounds through driven wells. The gang of wells known locally as the Boulevard system furnishes the greater part of the supply. The other source of supply includes two gangs of driven wells known as the Cook and the Hydraulic systems. The latter are brought into service when the Boulevard system becomes inadequate, for the reason that in 1899 the State Board of Health condemned the Cook wells because of the action of their water on lead pipe and the consequent danger of poisoning. With continued use, and to a constantly increasing degree, the Boulevard system has become unsatisfactory, owing to the amount of iron and manganese in the water which renders it turbid and altogether disagreeable. Studies and investigations were recently made and reported by Mr. F. A. Barbour, consulting engineer, Boston, Mass., to formulate measures for providing the city with a more acceptable supply of water. Mr. Barbour's report, dated June 4, 1914, describes his investigations of the present supply and gives his conclusions as to the best method of providing the city with satisfactory water. The portions of the report of most interest to water works engineers are those describing the experimental work in the decarbonation of the Cook well water, the experimental work on the removal of iron and manganese from the Boulevard wells, and the description of the plant proposed for the removal of iron and manganese from the Boulevard supply. The present article relates wholly to the decarbonation of the Cook well water. A second article will relate to the deferrization and demanganization of the Boulevard supply and to the proposed works for the removal of these metallic contents.

EXPERIMENTAL WORK IN THE DECARBONATION OF THE COOK WELL WATER.

The Cook wells were driven in 1893, the Hydraulic wells one year later. For some years these wells were in continuous use but in 1897-8 complaints of lead poisoning became so frequent that after an investigation of the effect of this water on lead pipe, made by the State Board of Health in 1899, the Cook wells were abandoned for a time. The Board's report stated that the Cook water contained more carbonic acid than any other supply in the state, and stated further that the city should either remove the lead service pipes through which this water was delivered for cooking and drinking or extend the Boulevard system to supply the entire city.

As a result of this condemnation of the Cook wells, the Water Board further extended the Boulevard system and so increased the supply obtainable from this source that in 1902 and 1903 no water was drawn from the Cook wells. Since 1904, however, owing to shortage or the poor quality of the supply obtainable from the Boulevard system, it has been found necessary to draw from the Cook wells each year for short periods—the amounts so taken varying from 3.2 per cent of the total consumption in 1904 to 28 per cent in 1913.

Since 1900, when the independent station at the Hydraulic wells was dismantled, the water from this system has been drafted by suction to the Cook wells station—about 1,000,000 gals. daily being so obtained during the period when this station has been in operation. During the experimental work, the Hydraulic wells were out of service, and the investigation has been confined to a study of the supply from the Cook system. The results, therefore, apply only to the treatment of the Cook well water, in which the important factor is the high carbonic acid content, which is responsible for the corrosive action of this water on metals. The iron in this water is low —less than .10 parts per million—but in the Hydraulic well water, from .30 to .60 parts of iron were found during the years when this supply was in service, and it is reasonable to conclude that, with continued use, the amount of iron in this water would have increased in somewhat the same way as it has in the Boulevard well system. The treatment of the Hydraulic well water necessary to render it satisfactory would therefore involve a plant for the removal of iron, whereas, in the case of the Cook well supply, the removal of the carbonic acid is all that is necessary to render it acceptable. As this latter supply is capable of furnishing from 2,500,000 to 4,000,000 gals. daily, dependent on the time of year, it has an obvious value as an auxiliary to the Boulevard system, providing the corrosive action of the water on metals can be corrected, and it was in an endeavor to determine how this could most economically be done that the experiments here described were undertaken.

The most active agent in the corrosion of lead pipe is the carbonic acid in the water. Decarbonation, by the neutralization or removal of this acid, is the obvious remedy—the first by the use of lime or soda, the second by aeration.

The investigation was planned to make possible a comparison of the lead found in the untreated Cook well water after passing through 50 ft. of ½-in. lead service pipe, with that found in the water after passing through similar coils of pipe when the carbonic acid had first been neutralized to various degrees by lime or removed by aeration. Three 50-ft. coils of lead pipe were used—one for the raw water, another for the lime-treated water, and the third for the aerated water. The effect of the period of contact on the amount of lead taken up by the water was determined by taking samples after a period of four hours standing in or passing through the pipe, and also after a period of 15 hours.

The apparatus for the addition of lime consisted of a funnel-shaped saturator, 24 ins. high and 16 ins. in diameter at the top. The lime was slaked in a small receptacle at the side of the saturator and introduced as milk of lime by a pipe which led to the bottom of the funnel. Water was introduced from a calibrated nozzle into a pipe which extended vertically through the center of the funnel to the bottom, the supply of water then rising slowly through the milk of lime and overflowing at the top from a circular weir. The capacity of the saturator was such that when treating 5,000 gals. per day of water with one grain of lime per gallon, the time of passage of the lime water through the saturator was about one hour.

The lime water varied in strength more or less, but on the average contained 1,120 p. p. m. of calcium oxide, or 65.5 grains per gallon of this oxide, these figures being based on determinations of the caustic alkalinity of the lime water as it issued from the saturator.

The flow of lime water was maintained at as nearly a constant quantity as possible, and the amount of raw water was varied when it became necessary to change the quantity of lime added per gallon. The lime water was applied to the raw water without exposing the latter to the air, duplicating, as far as possible, the conditions under which, if this method of removing carbonic acid were adopted, the lime water would be discharged into the suction pipe of the pumps.

Apparatus for the determination of hardness, alkalinity, dissolved oxygen, carbonic acid and iron was provided, and these analyses were made on the ground by Mr. Clifton L. Rice, chemist. The determinations of lead were made by the State Board of Health.

In starting the investigation, such portions of the raw, lime-treated and aerated water were passed through the lead pipes during the day that four hours were required between the time of entering and leaving these pipes. At a later date, as always in the case of the 15-hour exposure, the four-hour samples were collected from water which had been allowed to stand in the pipes for this length of time. In the interim between the periods during which the water stood in the pipes, the water was passed through at a rate of about 1 gal. in five minutes, the surplus of the treated water being wasted through a bypass. In general, the scheme was intended to represent, as nearly as possible, the actual conditions of a house service pipe.

The regular routine of the experiment station involved the determination, several times each day, of the dissolved oxygen and carbonic acid of the raw and aerated waters, and of the lime-treated water at the time of collecting the four and 15-hour samples; determinations of the strength of the lime water at two-hour intervals, and always, when closing down, for the four and 15 hour samples; determinations of the alkalinity, total hardness and iron once each day during August and September, and once each week thereafter; hourly observations of pressure on aerator nozzles; and hourly regulation of the rates of flow.

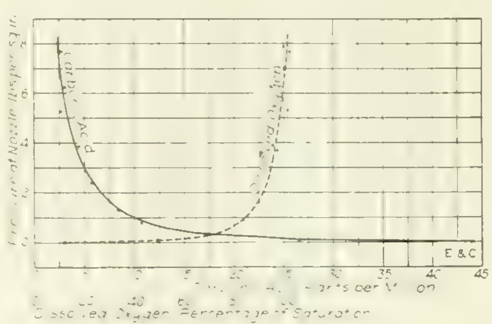
TABLE I.—RELATION BETWEEN CARBONIC ACID AND LEAD CONTENT OF UNTREATED AND LIME TREATED WATER AFTER REMAINING IN PIPE FOUR HOURS.

Date.	Untreated water: Carbonic acid, p. p. m.*	Untreated water: Lead taken up, p. p. m.	Lime treated water: Lime added, p. p. m.	Lime treated water: Lime added, gr., gal.	Lime treated water: Carbonic acid, p. p. m.	Lime treated water: Lead taken up, p. p. m.
September 2-4	37.5	[illegible]	16.9	[illegible]	16.0	[illegible]
September 9-11	40.5	5.200	18.5	1.08	19.0	1.400
September 17-19	38.0	5.200	16.7	0.98	16.0	2.200
September 23-24	35.5	3.000	16.8	0.98	14.5	1.314
September 27	....	2.800	17.2	1.00	..	0.914
October 2-3	31.5	4.000	17.4	1.02	9.0	1.828
October 7	37.5	3.600	17.7	1.04	13.0	1.600
October 10	45.0	5.200	38.0	2.22	1.8	1.400
October 17-19	41.5	6.400	35.6	2.08	0.0	0.400
October 22-23	38.0	4.800	25.8	1.51	6.0	0.714
October 27-28	42.3	5.200	26.1	1.53	0.0	0.257
November 5	39.0	7.000	29.7	1.74	0.0	0.257
November 6-8	41.5	8.000	29.1	1.71	1.8	0.943
November 12	36.0	4.600	29.5	1.73	0.0	0.400
November 13	37.5	5.200	25.5	1.49	0.0	0.800

*p. p. m. = parts per million.

The important results of the investigation are given in Tables I, II and III.

Analyses of the raw water collected from the pumps at the Cook well stations showed an average of 33.1 p. p. m. of alkalinity, 55.2 of total hardness, 22.1 of incrustants, 0.07 of iron and 44.9 of carbonic acid. The average dissolved oxygen content was 10.9 per cent saturation.



Curves Showing Relation Between Pressure on Aerator Nozzles and Carbonic Acid and Dissolved Oxygen in Aerated Well Water at Lowell, Mass.

Tables I and II indicate the effect of adding lime to the water in reducing the corrosive effect on lead pipe. It should be first noted that 0.5 parts of lead per million of water is generally considered the maximum which can be permitted in a potable supply. Referring to Table I, showing the effect of a four-hour period in the pipe, it appears that the lead in the untreated water averaged about 5 p. p. m., or practically 10 times the safe limit, and for a 15-hour period, as shown in Table II, 6.7 p. p. m., or 13 times the safe limit. Very obviously, the Cook well water, in its present condition, is entirely unfit for use.

Again referring to these tables, it appears that, by the addition of approximately one grain of lime (100 per cent available), the lead taken up by the water after a period of four hours in the pipe, is reduced to 1.5 p. p. m., and after a period of 15 hours, to 3.1 p. p. m., or respectively, three and six times the safe limit. When the lime is increased to 1¾ grains per gallon, the carbonic acid is practically neutralized, and the experiments indicated that the corrosive action on lead is so far reduced that the lead content of the treated water, after remaining in the pipe 4 or 15 hours, is approximately 0.5 p. p. m. Further, the experiments indicate that with 1.5 grains of lime per gallon, the effect of the water on lead would be sufficiently nullified as to render the supply safe under the conditions of actual use—a conclusion based on the fact that the tests represented more severe conditions than occur in the actual distribution of the water.

On the other hand, it is believed that the experiments show that a lesser amount of lime than 1.5 grains per gallon would not be successful—a rather disappointing result, as the addition of this lime will theoretically increase the hardness by 45.6 p. p. m., or would add 80 per cent to the present total hardness of the water. The total hardness of the treated water would, under these conditions, be about 100 p. p. m.—an amount too great to be satisfactory to consumers accustomed to soft New England waters.

It had been hoped, in starting these experiments, that the addition of a fraction of a grain of lime per gallon would so far reduce the corrosive effect as to make the supply safe, but the tests indicate that such is not the case. It is also of interest to note that the continued passage of the water through the lead pipes does not appear to reduce the corrosive effect by the formation of any coating in the pipes.

It has been calculated that the increased cost of soap used by a community equals 10 cts. per 1,000,000 gals. for each additional part per million of hardness which, for the increase in hardness due to the addition of 1.5 grains per gallon of lime, would be equivalent to $4.65 per 1,000,000 gals. of the treated water used in the city.

Concluding the discussion of lime treatment, it is believed that the decarbonation of the Cook well supply by lime has been proved to be undesirable and uneconomical because of the resulting increase in hardness.

REMOVAL BY AERATION.

Turning to the discussion of the experiments in removing carbonic acid by aeration, the results are more satisfactory.

Small nozzles ⅛-in. to ¼-in. in diameter, furnished by the Spray Engineering Co. of Boston, were used in the aeration test. The pressure on the nozzle was obtained by a gage tapped into the supply pipe just below the nozzle, or, for pressure too light for registration by a gage, a manometer tube was used.

Table III shows the relation between the carbonic acid in the aerated water and the lead taken up by this water after standing in the pipes for 4 or 15 hours.

TABLE II.—RELATION BETWEEN CARBONIC ACID AND LEAD CONTENT OF UNTREATED AND LIME TREATED WATER AFTER REMAINING IN PIPE 15 HOURS.

Date.	Untreated water: Carbonic acid, p. p. m.	Untreated water: Lead taken up, p. p. m.	Lime treated water: Lime added, p. p. m.	Lime treated water: Lime added, gr., gal.	Lime treated water: Carbonic acid, p. p. m.	Lime treated water: Lead taken up, p. p. m.
August 22	[illegible]	15.152	17.4	1.02	16.5	6.000
August 27-28	37.2	6.000	18.1	1.06	13.0	2.752
September 2-4	34.8	4.516	17.6	1.03	6.5	5.000
September 9-11	40.6	5.290	18.9	1.11	9.0	1.800
September 17-19	38.0	3.200	16.7	0.98	14.0	1.600
October 2-3	37.0	5.600	17.6	1.03	18.0	1.915
October 7	35.5	6.000	16.0	0.94	11.0	2.600
October 10	36.0	4.344	36.0	2.11	0.0	0.400
October 17-19	39.5	10.000	32.8	1.92	0.0	0.571
October 22-23	34.5	8.000	26.5	1.55	7.2	0.800
October 27-28	34.5	7.200	24.8	1.45	2.6	0.543
November 5	37.5	8.000	30.9	1.80	0.0	0.543
November 6-8	41.0	9.000	29.1	1.70	1.0	0.543
November 12	34.5	5.200	29.9	1.75	0.0	0.257
November 13	35.0	5.600	25.3	1.48	0.0	0.400

The accompanying diagram shows the relation between the pressure on the nozzle and the carbonic acid left in the aerated water.

One of the principal objects of the test was to determine the lowest pressure on the nozzles which would effect the desired removal of the carbonic acid. The nozzles were of a type which threw a very fine spray, and the results indicate that such a spray is very effective in the removal of carbonic acid, although the time during which the spray is in the air is short.

The tests indicate that by a pressure of 5 lbs. on the nozzle, the carbonic acid can be reduced from 45 p. p. m. in the raw water to about 3.3 p. p. m. in the aerated water, and that with this reduction the effect of the water on lead pipes is so far lessened as to render the supply safe. In stating this conclusion, the fact that, in the 15-hour tests, the lead taken up somewhat exceeded 0.5 p. p. m. is not overlooked, because it is believed that the retention of the water for 15 hours in the pipe is too severe a test, and that the four-hour period of contact more nearly represents the conditions of actual use.

The removal of carbonic acid by aeration has a great advantage over the lime treatment in that it does not increase the hardness, and further, as indicated by the tests, aeration is more effective than the lime treatment in reducing the corrosive action when an equal amount of carbonic acid is left in the treated water. This result is not what might be expected, because by aeration the dissolved oxygen, which has generally been considered a contributary factor in corrosion, is increased, but the experiments clearly indicate that, with an equal content of carbonic acid, the aerated water takes up less lead than the lime-treated water.

TABLE III.—RELATION BETWEEN CARBONIC ACID AND LEAD CONTENT IN AERATED WATER AFTER REMAINING IN PIPE FOR 4 AND 15-HOUR PERIODS AT COOK WELLS, LOWELL, MASS.

Date.	4-hour sample		15-hour sample	
	Carbonic acid.	Lead taken up.	Carbonic acid.	Lead taken up.
	Parts per million			
September 9-11	3.0	0.114	3.5	0.571
September 17-19	3.5	0.457	3.0	0.543
September 23-24	4.5	0.657	3.5	0.800
October 2-3	3.5	0.457	3.0	0.657
October 7	3.5	0.400	4.5	1.143
October 17-19	4.0	0.400	3.5	0.400
October 22-23	10.0	0.400	13.0	0.743
October 27-28	17.0	0.600	16.0	0.886
October 28	3.0	0.343	....	.....
November 6-8	10.0	0 343	9.5	0.943
November 12	22.0	0.943	17.5	1.143
November 13	3.0	0.257	...	.....

An aerating plant, if constructed, would include a pipe system into which the necessary number of nozzles would be tapped, a concrete pool, and below the pool, a small storage basin with a capacity of perhaps one hour's run of the pumps. The water, as drawn from the wells, would be forced through the aerator, and after falling into the storage well would be repumped into the distribution system. It is estimated that the first cost of an aerator, storage well and new steam turbine centrifugal pumping unit of 4,000,000 gals. daily capacity will be approximately $25,000. Assuming that the installation of the aerator will involve an additional lift of 30 ft., the cost of aerating the water will not exceed $2.25 per 1,000,000 gals., including interest, depreciation and the cost of extra pumping necessitated by aeration. This cost is less than the cost to consumers for additional soap, which would necessarily be used if the carbonic acid were neutralized by lime treatment, and it therefore follows that aeration is the more economical method.

The experimental work above described has proved that the Cook well water can be made suitable for use by aeration at relatively small cost. When thus rendered safe, it can be, in the future, utilized, if required, as an auxiliary to the Boulevard system in providing for the summer peak in the consumption curve. Aeration alone is, however, adapted only to the treatment of the Cook well supply, without admixture of water from the Hydraulic wells, from which the iron must be removed in order to render it satisfactory.

Motor surveys of 18 Ontario counties where highway systems have not yet been organized have been commenced under the auspices of the Good Roads Commission.

Design of Submerged Intake Crib and Flexible Jointed Pipe Line at Burlington, Ia.

(Staff Article.)

A new water supply intake consisting of a rock filled, submerged, timber intake crib and a 30-in. cast-iron pipe line with flexible joints will be constructed in the Mississippi River at Burlington, Iowa, during the present season. The new intake is for the water system of the Citizens Water Co. and will be installed under the specifications of Mr. Frank Lawlor, Superintendent. Some of the features of the proposed work are here described from information abstracted from the specifications.

The length of the proposed pipe line is 1,015 ft. A trench for this pipe will be excavated to an established grade line. The bottom width of trench will be not less than 5 ft. and the side slopes will be not steeper than two horizontal to one vertical. This trench will be maintained to the lines specified until the pipe is properly placed and all the joints are thoroughly bolted and caulked. As a rule five lengths of pipe will be made up on shore and the parts of a flexible ball joint placed on each end. When all the joints have been well filled with lead and thoroughly caulked bulkheads will be placed on each end and the section, which will be about 63½ ft. in length, will be launched and placed in position. The bulkheads will then be removed and the nuts on the bolts of the ball joint tightened. As a rule, the pipe will have bell and spigot ends but about 15 flexible ball joints will be furnished by the water company. An elbow and a short piece of pipe to extend up into the crib will be placed on the outer end of the last section.

After the pipe line is laid it will be inspected by a diver and if it is found that any joints have pulled apart or opened so they cannot be caulked to make a good job the pipe will be taken up and relaid. All joints will be air tight. To test for this condition, as soon as the pipe laying is completed a plug or cap will be placed over its outer end and air pressure will be applied to the line from an air compressor.

Crib.—As soon as the intake pipe is in place, a crib will be sunk over its outer end. This crib will be 14 ft. long, 8 ft. wide and about 8 ft. in height. It will be built of 8x8-in. pine timbers put together as customary in such structures and bolted at every intersection with ⅝-in. round iron bolts 25 ins. long. A ½-in. auger will be used in preparing the holes for these bolts. Ballast floors of 2x4's spaced 2 ins. apart in the clear will be placed on top of the second 8x8-in. timber from the bottom of the crib. These scantlings will be toe nailed to the main timbers of the crib with 40-dy. wire nails. On the back or west side of the crib, the lower courses of 8x8's will not be used near the center of the crib immediately over the 30-in. pipe. Accurate soundings will be made so that the bottom of the crib will conform to the bottom of the river. Before the crib is filled with stone, it must rest evenly on the bottom when its sides are plumb and its top horizontal. Any inequalities of the bottom to which the crib does not conform will be removed by a diver. As soon as the superintendent is satisfied that the crib is in proper location, it will be filled with stone, except in the space immediately around the pipe, and stone will be piled on top of the crib to a height of 1 ft. above the timber work. In placing the stone filling, care will be exercised to put the larger stones in the openings between the timbers and not allow any stones to fall into the pipe. The stones to be used for filling will be of a size that can be conveniently handled by one man.

No part of the pipe line or submerged crib will be higher than 6 ft. below low water mark, which is about 1 ft. 8 ins. below ordinary low water mark. If any ridges or plies of material are thrown up by the dredge they will be removed to a depth of 6 ft. below low water.

Abuses of and Regulations to Control Use of Fire Hydrants at New Orleans, La.

The use and waste of water through fire hydrants in the city of New Orleans during the year 1913 amounted to approximately 3,000,000 gals. per day. Observation indicates that a very large proportion of this quantity is wasted. Workmen with hydrant wrenches frequently open hydrants and draw off enormous quantities of water for reasons which are purely whimsical. Such men, moreover, are often exceedingly careless in opening and closing hydrants, putting undue strains on the moving parts and otherwise abusing the hydrant.

When water is discharged freely through an open hydrant it loses all its pressure, in excess of atmospheric pressure, as soon as it reaches the air. It then escapes by the gutters or drains with a minimum flushing effect. All the matter that the flowing water will transport is removed at once and after the first few minutes such flushing is of no use whatever, yet it is a common practice in New Orleans to let from 100 to 500 gals. per minute run into a gutter or drain for a half hour at a time. This practice is by no means peculiar to New Orleans but obtains, probably, in the majority of cities. The amount of water thus wasted if utilized through a hose and proper nozzle can often render service fully warranting its use, but in general it does not do so when used as above described.

In the past there has been much complaint in New Orleans from those having supervision of sprinkler systems, and from other water consumers as well, and much unnecessary damage has been sustained by the hydrants and the pipe system and its connections by the improper use of fire hydrants. Consequently the following instructions for the use of hydrants, with the reasons therefore, have been issued by Mr. George G. Earl, general superintendent of the New Orleans Sewerage and Water Board:

To Parties Using Fire Hydrants:

Your co-operation in seeing that the following rules for the use of fire hydrants are strictly complied with is earnestly requested:

1. Open a fire hydrant slowly, and steadily, and always enough to prevent any chattering of the main valve, even if more water is drawn than is required.

2. If the fire hydrant is to be left open, with pressure available, for self-closing or other faucet, or valve, on its outlet, for intermittent use, open it slowly, until the hydrant is full open, then release the operating nut by one turn backwards. Where a hose bib or other connection is placed on hydrant, care must be used to see that rubber gaskets are placed under the nozzle caps.

3. Close a fire hydrant very slowly, especially as the valve approaches its seat, until it comes firmly to its seat, and then release tension on valve and stem by one turn backwards of operating nut.

4. After closing a fire hydrant see that the caps are properly replaced over its outlets.

5. Do not allow a fire hydrant to run for gutter flushing after the flow in the gutter becomes reasonably clear.

6. Use no tools to operate a fire hydrant other than a close-fitting five-sided wrench fur-

nished by the Sewerage and Water Board. If wrench becomes worn, so that it injures the operating nut, turn it in and obtain a new one in its place.

The reasons why it is essential to comply with the above are as follows:

There is considerable complaint by the sprinkler supervisory authorities, and others, brought about by the improper handling of fire hydrants, and great damage and expense in repairs is being caused by their improper use.

The very rapid opening of a fire hydrant, with a free discharge, causes a sudden and momentary drop of pressure, which sends in a low pressure alarm.

The sudden closing of a fire hydrant throws a strain, in the nature of a water hammer, upon the whole pipe system in its locality, and upon all connections thereto, that is often double the static pressure in the system, and is a severe test upon all joints and connections.

The drawing of a very small stream from a fire hydrant through a main valve which is barely off its seat, sets up a vibration and noise that is equally trying upon all connections and upon the nerves of the occupants of buildings.

Any and all of the above procedures cause damage to the hydrants, and necessitate frequent and expensive repair.

The leaving of the openings of a hydrant uncapped gives opportunity for the insertion of matter into the hydrant and for damage to the threads of same, either of which will destroy or retard its usefulness when it is needed for fire service.

The use of any tool in operating hydrant, except a well-fitting five-sided wrench, furnished by the Sewerage and Water Board, very quickly wears the operating nut past further usefulness.

The leaving of a hydrant open less than full when it is supposed to be open for supplying water for building purposes, etc., applies full pressure to a drip in its base, which causes the water to escape around the base and soften same, and also to escape at the surface, thus damaging the hydrant setting and pavement.

The flushing of gutters from fire hydrants by a free discharge, without hose or nozzle, for considerable lengths of time, is, in nine cases out of ten, a waste of a very excessive amount of water after the first five or ten minutes, since water so applied has no efficiency in scouring or removing any other than the lightest matter.

Some Notes on the Installation of Gas Engines.

The increasing use of internal combustion engines in small water works pumping stations warrants a brief consideration of the best procedure in their installation. The notes here given are taken from a recent article by Mr. T. L. Hobbs, published in *The Gas Engine*.

The acceptance and unloading of gas engines are important considerations so it will be assumed that the installation of the engine begins with the receipt by the purchaser of a notice from the railroad company that the ordered engine has arrived. If the engine is heavy the car on which it is shipped will usually be placed on a siding at a point convenient for unloading to a wagon. Before paying the freight the purchaser should inspect the condition of the engine in company with the station agent. If any damage is discovered the agent should be required by the purchaser to make a full notation of the nature of the damage upon the expense bill. The freight may then be paid.

If help is needed in unloading, the purchaser has the right to call upon the agent or section foreman for this assistance, as it is their duty to unload the engine unless it is large enough to be a minimum carload by itself, in which case the purchaser must unload it. In either case the agent is to see to it that suitable timbers are supplied, such as ties or other heavy timbers. Every reasonable effort is supposed to be used to unload the engine without damage, but if damage should occur to the engine in unloading on account of faulty timbers or lack of proper assistance, where the engine is shipped local, the railroad company is liable for the damage.

An instance of this kind happened to an erecting man who was sent out to unload and install a 12-HP. stationary engine. He asked the agent for suitable timbers, and the assistance of the section men. The section men were out on the road and the timbers furnished were not good. The erector refused them, but the agent informed him that was all he had. The expert accepted the situation under protest and proceeded to unload the engine. When it was about half way from the car to the wagon one of the timbers gave way and the engine went bottom side up in the ditch. He went to the long distance telephone and called up his employer to ask for instructions. He was told that another engine would be immediately loaded and shipped by the first freight, and for him to turn the ditched engine over to the agent and wait for the new one. When the agent reported the matter to his superior he was instructed to have the section men place this engine on a car and return it free of charge to the factory, where it was rebuilt and made as good as new at the expense of the railroad company.

The next proceeding is the unloading of the engine. The first thing to consider is a suitable wagon. A strong flat top dray wagon is the most desirable conveyance. An ordinary wagon coupled short with the space between the standards filled with ties or any other strong timbers is very good. After the engine is on the wagon and fastened securely the smoothest road to the place of installation should be used, avoiding any road which is lower on one side than the other. Arriving at the plant it may be wise to dig trenches for the wagon wheels in order that the engine may be nearer the ground.

The engine room and foundation may already be provided, but a few things which should be considered in this connection will be mentioned. The following is considered an ideal installation, and the nearer the actual approaches this the better service may be expected from the engine.

The engine should be installed in a clean, dry room with plenty of light, in such a way that the foundation can be built up direct from the ground. The foundation should be twice as long at the bottom as the base of the engine and should be 2½ times as wide as the frame of the engine. There is no regular depth, but the deeper the better, even if the foundation goes to bed rock. The hole should be dug wider at the bottom if the ground will stand it. If not, a form may be used to give the desired shape. The foundation should be in the form of a pyramid, being from 2 to 6 ins. wider at the base of the engine than the bottom of the engine frame. Anchor bolts, long enough to go entirely through the foundation should be provided for fastening the engine to the foundation. On the bottom of these bolts should be large, strong washers to aid them in their task. The bolts are set in proper position so that they will, when the foundation is completed, enter the holes at the bottom of the engine frame provided for them, and should be as large as can be used. A templet is generally supplied, made of wood with holes in to match the holes in the bottom of the engine frame.

The foundation should consist of concrete in the following proportion: Three parts of small pieces of rock or gravel, two parts of fine sand, and one bag of cement thoroughly mixed with plenty of water to make it easy to work into position. It should be packed tightly with a heavy timber or other suitable rammer. Old scrap iron can be used to good advantage. Any scrap, such as old buggy springs, cast-iron wheels, pieces of chain, heavy pieces of wire, woven wire, in fact anything which will add to the strength and prevent the cement from cracking. This should be allowed to set for four or five days, the longer the better. It is then ready for the engine. The engine should not set directly on the concrete, but should have at least ½ in. of wood between them, the more wood up to 2 ins. in thickness is better. After the engine is put in place, the nuts should be put on the bolts and drawn down a little at a time, going from one to the other until they are as tight as they will stand.

It may be necessary to install the engine on a floor of wood. If this be the case a couple of long heavy timbers should be provided and bolted tightly to the floor and the engine fastened securely to this, which will make a very good foundation.

If the machinery to be driven is already in place it will be necessary to figure out where the belt wheel will have to be and to set the foundation accordingly, but if the machinery is not in place it will be a very easy matter to line it up with the engine.

The water cooling system for the cylinder is generally planned and provided by the factory ready to install and will work nicely if the proper attention is given to the draining of the pipes to prevent freezing. An ideal system, which is as near automatic as can be made, is to have the water tank under the ground, with the pump below the frost line. Just above the pump a ¼-in. hole is drilled in the pipe so that every time the engine is stopped it will be drained automatically, then by the time it begins to get warmed up the water circulation will be working.

The battery box should be placed in the cleanest, dryest place in the building, where there will be as little vibration as possible. The switch may be placed near the engine where it will be handy, and the battery box be placed in another room or up on a high shelf, any place to get it where it will be free from moisture, dirt and vibration. This will do away with the possibility of a wire being broken, or jarred loose from a connection by the constant vibration.

A wire should be run from the engine frame to the zinc or outside post of the first cell, a wire from the carbon or center of this same cell to the zinc of the next, and so on through the whole battery. At the end of the series there will be a carbon post unconnected. Run a wire from this to the center binding post of the switch (a three-point switch should be used), another wire from one side of the switch to the coil, a third from the other side or post of the coil to the igniter, and the wiring is complete. The coil may be placed inside the battery box, or at any other convenient place where it will be free from dirt, grease, dampness and vibration.

If it is desired to use a magneto with the ignition system, all that is necessary is to place the magneto on a solid foundation where the drive wheel of the magneto may come in contact with the fly wheel of the engine, run a wire from one post of the magneto to the frame of the engine, or it may be connected to the other wire which goes to the frame of the engine from the batteries, if more convenient. Then run a wire from the other post on the magneto to the unoccupied post of the switch and the magneto wiring is completed.

If liquid fuel is to be used the tank should be placed outside of the building; burying it in the ground is the best practice. The tank and piping are generally furnished with the engine, but it is generally necessary to make some changes in the piping which can easily be done by the use of a hack saw and a thread cutter for rethreading the pipes. Where it is desired to use a small quantity of gasoline for starting the engine and a different fuel for running a small tank can be placed in some convenient place to hold enough gasoline for starting.

If natural gas is to be used for fuel, a special mixer, which can be furnished by the manufacturer, is required. A small tank for the gas, or a gas bag to aid in getting the proper quantity of gas into the cylinder quickly is also necessary.

Observations of the Effect of Ozone on Algae Growths.

For the past five years the Baltimore County Water and Electric Company, under the supervision of Mr. A. E. Walden, have been using ozone as a sterilizing agent to purify their Herring Run water supply. The system as it was originally installed did not accomplish the results that were expected in many ways, so that it became necessary to entirely redesign, as little information could be obtained from any of the European plants in operation. It is not, however, the object of this article to recite the details of the work that has been done, but merely to relate the effect of ozonization upon certain types of micro-organisms, as described by Mr. S. T. Powell in his paper before the recent annual convention of the American Water Works Association.

Herring Run is a small surface water stream that flows for the most of its course through a thickly populated territory and receives more or less surface drainage, so that there is always an appreciable amount of matter carried in suspension by the supply. Before reaching the ozone plant the water is stored for about 21 days in two earthen reservoirs from which a portion, but not all, of the top soil was removed before they were placed in use. Algae and other vegetable growths have always been more or less prolific in the supply and it has been necessary to treat the water many times each year with copper sulphate to keep down these objectionable growths. Up until the present time the water flows to the ozone plant direct from the reservoirs without prefiltration and for this reason we have been afforded an opportunity to note the effect of ozone as an algaecide.

The method of mixing the ozone and water is by means of aspiration, the falling water sucking the ozone directly from the generators, and is carried through a mixing chamber and from there delivered to the suction well of the pump. This well is open to the atmosphere so that the ozone not used up during passage through the mixing chamber is allowed to escape at this point.

It was noticed that very soon after putting the plant in operation that a foamy greenish scum collected on the surface of the water, which increased in thickness the longer the plant operated and had to be removed by overflowing the well. Microscopic examination showed that the accumulation was due to algae growths to which had adhered small bubbles of ozonized air and had carried the organisms to the surface of the water. It was evident also that the ozone had a disintegrating effect on many of the organisms, especially the chlorophyceae and cyanophyceae. The analyses of samples of water before and after ozonization showed that the treatment had materially reduced the total number of organisms, as well as the amorphous matter. The percentage removal varied from day to day but not uniformly with the changes in the ozone concentration. It was evident though that although the reduction showed the extent to which the organisms were eliminated from the water, this was by no means due entirely to oxidation by the ozone but partly on account of breaking up of the more delicate organisms by the violent agitation of the water in passing through the mixing chamber, and to the formation of scum on the surface of the water in the suction well.

For this reason laboratory tests were made to determine to what extent the algae were killed by direct contact with the gas and the general effect on the water resulting from ozonization of samples impregnated with such vegetable growths, especially with reference to color and odor.

SOME RESULTS OF LABORATORY TESTS.

The experiments indicated that low concentration and long contact will usually ensure as thorough oxidation of organic matter as higher concentration and short periods of contact.

Color Removal.—Samples of swamp water were ozonized for one-half hour and then examined microscopically. There appeared to be a general disintegration of all the organisms containing chlorophyll and excepting where these growths were massed in bunches they were killed. The chlorophyll was scattered through the water but apparently the ozone had but a slight, if any, bleaching effect upon it. The diatoma, crustacea and protozoa were unaffected. After ozonization the samples were set aside for 24 hours and again examined. By this time the crustacea and protozoa were all dead but not removed; the color of the water had increased and there was a decided increase in the odor. The killing of the crustacea was probably due not to the oxidizing effect of the ozone but on account of the removal of the food supply by sterilization of the water. The color and odor increase was caused by the scattering of the chlorphyll and oil globules through the water when the organisms were broken up.

In support of this theory of the color increase a weak gasoline solution of pure chlorophyll was made up and through this ozone was passed for five hours. Even after this long period of contact there was not the slightest reducion in the color of the solution. The same concentration of ozone in 20 minutes removed 75 per cent of the color in a sample of water which had been stained by boiling a quantity of dead leaves in distilled water and then filtering.

Odor Removed.—There has been much discussion recently in reference to the removal of odors by ozonization, but practically all of the investigations have been directed to the elimination of malodorous compounds that exist in the atmosphere so that the removal of odors in solution touches a somewhat different phase of the subject. An attempt was made in this investigation to determine the general effect of ozonization in reference to odor removal from water, particularly those odors arising from algae growths. It was demonstrated from the tests that where the ozone acted upon the water containing living micro-organisms that the characteristic odors were intensified. This condition was due to oxidation and disintegration of the plant and scattering the oil globules through the solution. These oily substances were only acted upon after complete oxidation of the organic matter present. In view of this fact waters containing micro-organisms were first filtered before attempting to deodorize them with ozone. At the time of conducting these tests it was not possible to obtain samples of water giving all the characteristic odors of the various forms of algae growths, but a number of substances that impart a distinctive odor were mixed with samples of tap water and the deodorizing effect of the ozone was noted. The data obtained from the tests, although they do not give any positive evidence of the ability of ozone to remove all odors arising from micro-organisms, give some idea of the value of the gas as a deodorizer for water impregnated with such objectionable substances.

A distinctive algae odor arising from a sample of stagnant swamp water was greatly reduced in 10 minutes with an ozone concentration of 0.20 gm. per cubic meter and completely removed in 30 minutes. The same odor was entirely eliminated by a concentration of 1.69 gms. in one minute.

A faint fishy odor caused by algae growths was removed in 10 minutes with a concentration of 0.18 gms. of the ozone gas.

CONCLUSION.

The observations made demonstrate clearly that certain forms of algae are very readily removed from water by direct oxidation while certain other forms are entirely unaffected even with protracted periods of contact with the ozone. In addition to this it has been noted that there is an increase in the odor arising from direct ozonization of algae that can only be removed after complete oxidation of the organic content of the water.

As has previously been stated, these studies were undertaken not with the idea of making use of ozone as an algaecide but to determine what influence such growths would have in maintaining the efficiency of the sterilization plant, and to this extent the experiments have been of considerable value.

At practically all ozone water sterilization plants abroad the raw water supplies are filtered previous to treatment so that the influence of algae upon the efficiencies of the systems has not been studied. As the rate of filtration used at all these places is about the same as employed by mechanical filters in this country it is reasonable to suppose that practically all of the organisms are removed by the filter beds, but there is a possibility of the growth of organisms in the underdrains and conduits of the system, the presence of which will reduce the efficiency of sterilization in proportion to their abundance.

The ability of ozone to remove or reduce certain odors arising from substances dissolved in water is to the mind of the writer one of great importance. There is no doubt that ozone is a powerful deodorizing gas under certain conditions and when brought into direct contact with the substance for a sufficient period. Its deodorizing value depends on the oxidizability of the substance treated, and concentration of the gas as well as the thoroughness of the mix.

Notes on Reservoir Construction.

The following notes on the construction of concrete reservoirs and settling basins for water works service are abstracted from a paper by Mr. T. C. Hughes, city engineer of Tulsa, Oklahoma, before the recent annual meeting of the Southwestern Water Works Association.

The most satisfactory mixture of concrete to use in building reservoirs and basins, in Mr. Hughes' experience, is a 1:2:4. This meets the requirements of density and non-porosity. The cement must stand the ordinary test of the American Society for Testing Materials. The sand should be coarse and absolutely clean and the stone should be a mixture of ½ and 1 in. rock. This character of aggregate gives the most satisfactory results and with the proper care in tamping will produce a wall absolutely impervious to water without any special protective agency such as hydrated limes, waterproofing compounds, or ground clay.

Two practical difficulties occur in building water basins of this nature for municipal use. One of these is to make a satisfactory joining of the old and new concrete. This can be accomplished where the connection is to be made between the old work and that several days old by first washing the old surface of the concrete with strong soda water or lye water to take any possible grease and then washing the soda or lye water with clear water, then washing the surface again with about an eight or ten per cent muriatic acid solution, then flushing this solution off with perfectly clean water. Sprinkle on dry cement and at once place and tamp the new concrete material down. By this process an almost perfect bond will be furnished between the old and new work.

The second trouble is to make a satisfactory joint in a wall where the same becomes necessary as between a completed compartment and a new compartment. Possibly the best construction for this purpose is to make a dovetail connection and thoroughly paint the inside with asphaltum having a penetration of about 75° Dow Standard and then run the new concrete. This will usually make a water tight construction. In making the connection between old and new concrete it must be borne in mind that the "laitance" or scum that always arises on cement work must be eliminated and cleaned off before any cohesion can be obtained between the old and new work.

In beginning the construction of a reservoir or basin the foundation upon which it is to be placed should be investigated thoroughly. If this should be composed entirely of sand, as for instance in the Arkansas River bottoms, it will be necessary to drive to the solid foundation a sheet of watertight sheet piling completely surrounding the work; and before laying the floor upon this enclosed foundation of sand at least 2 ft. of thoroughly ground and puddled clay should be tamped down so as to be impervious to leakage.

In a square or rectangular structure it is perhaps the best practice to build a gravity wall such that the section would be stable by its own weight, especially if the compartments are larger than 70 ft. in any one direction.

Where cross walls occur more frequently

than this the design may be cut down such an extent as not to depend upon a gravity section in all cases. This is a matter of detail, of course, to be determined by the designer at the time of making the plans. Every wall should have a proper and sufficient footing to carry the weight of the section imposed upon it without a tendency to crack.

One of the frequent mistakes at this point is in failing to provide a sufficiently large size drainage system to empty the basin quickly and save the time of the department in cleaning out as occasion may arise when this work must be done expeditiously, therefore the draining and cleaning system of basins should be looked after with more care than is usually done by designers of water basins.

All basins should be so arranged, if a coagulant system is used, such that the passing of water will be through the basin in as nearly a solid column as practical, as the proper sedimentation will, of course, depend upon the slowness of movement of water through the basins.

During the construction work judicious care should at all times be exercised by the engineer in charge, and the placing of the reinforcing and the proper cleaning off of the surfaces and the making of good joints should be the continuous care of the inspector.

The structure after it has been completed should be very carefully gone over and inspected in detail and afterward painted on the inside with a water-proofing paint or a thin mixture of neat cement, after which it should be filled and carefully looked after during the test of filling.

All openings for pipes entering the basin should be square and in a pyramidal form with the large base towards the inside of the basin such that the pressure will tend to close up any faulty work or leakage.

ROADS AND STREETS

The Organization and Standards of the Iowa Highway Commission.

Staff Article.

A state highway commission with sufficient power to supervise all road and bridge work was created by the Iowa legislature in April, 1913. Briefly stated, the road law provides for a practically unpaid commission (compensation not to exceed $1,000 each per year) of three men who provide the necessary organization to supervise the road and bridge work of the various counties in the state. The duties of the commission are general rather than specific, although the power is vested in it to require work which conforms to standard.

A subsidiary and in a manner independent organization, although under the general supervision of the county supervisors, is provided in each township. The township trustees appoint a superintendent who has charge of the maintenance and repair of township roads, awarding contracts for dragging and when necessary supervising force account and contract work.

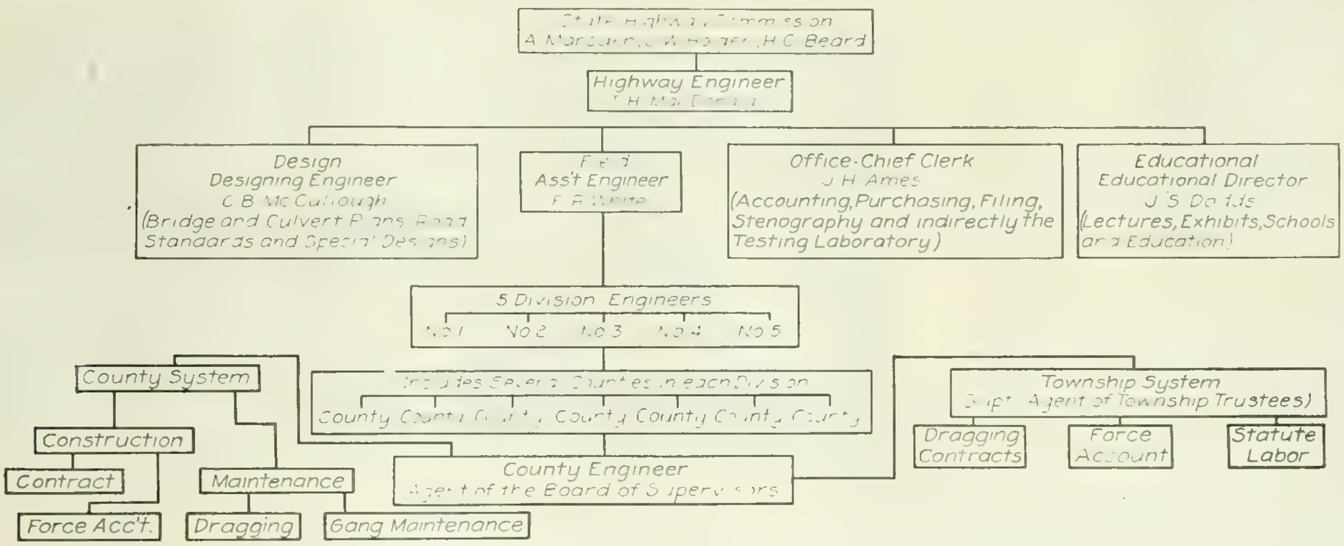


Fig. 1. Chart Showing Organization of Highway Work in Iowa.

The law further provides that each county shall be organized as a separate unit under the supervision of the commission. The county board of supervisors has direct charge of the work in the county. It is required to employ as its agent a county engineer who has direct supervision over all road and bridge work. The engineer is bonded, his term of office and compensation are fixed by the county board and he must be acceptable to the State Highway Commission, since it has specially granted power to discharge him at pleasure. From 10 to 15 per cent of the total mileage of roads outside of incorporated towns are selected by the county board of supervisors as a county highway system. This system is modified by the state commission if it sees fit and recorded as the system all work on which must be done under the direct approval of the state commission. For the upkeep of their county road system the county commissioners are held responsible to the commission. The duties of the county engineer are named and cover the entire field of engineering and supervision, both by contract and force account on road and bridge construction and maintenance.

Funds are divided for road and bridge purposes and are apportioned by the board of supervisors, with the exception of a non-divertable fund to provide for the dragging and maintenance of township roads which is apportioned by the township trustees. The law is very specific in regard to the duties of the various county officials, the distribution of funds and the quality of work to be accomplished. Provision is made for the extension of the county road system after the 10 to 15 per cent of the total mileage of road first designated is completed.

The organization perfected by the commission to carry out its work is shown graphically by Fig. 1. It will be seen that the county is adopted as a unit, the state commission acting in an advisory capacity with power to enforce its regulations when necessary.

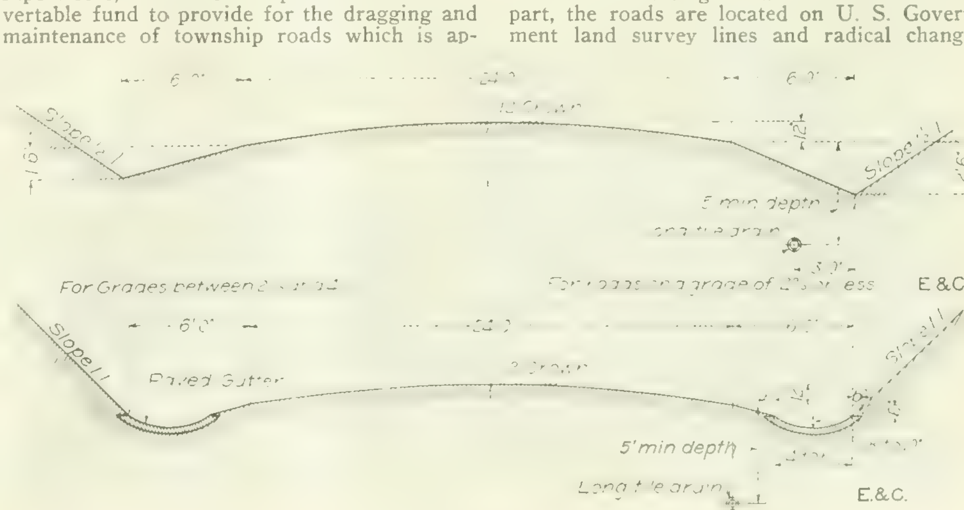


Fig. 2. Cross Sections for Earth Roads.

General Conditions.—The population of Iowa is to a large extent rural. It is an agricultural state of great wealth. For the most part, the roads are located on U. S. Government land survey lines and radical changes in alignment are difficult. The roads are generally 66 ft. wide. Little expensive construction exists, the present roads being of the graded earth type with little attempt to conform to established grades. Over a large portion of the level section of the state the soil is of black, sticky nature rich in organic matter and in need of drainage, but yielding

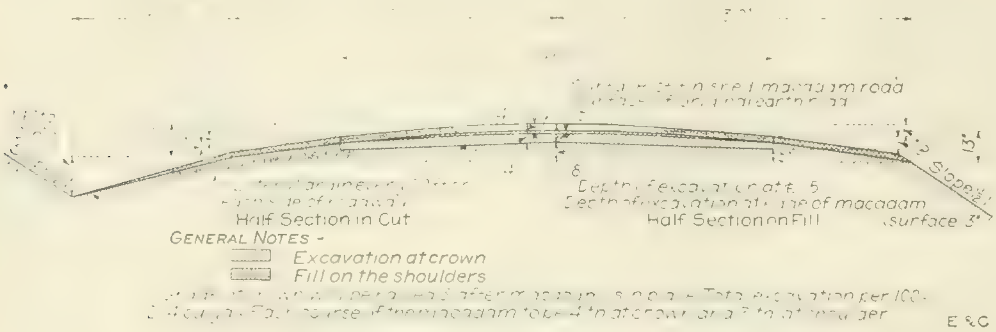


Fig. 3. Cross Section for Gravel and Macadam Roads.

readily to continuous dragging. Materials for constructing all types of roads are available throughout the state, but the most urgent need is the drainage, grading and maintenance of the present earth roads in a passable condition and the construction of a more substantial type of waterways.

Cross Sections.—The cross sections recommended by the commission for use on earth roads, especially in prairie sections, are shown by Fig. 2. It will be noted that the depth of ditch is varied with the grade and on grades exceeding 4 per cent a paved gutter is used. This provision is of value in easily eroded soils and in level sections where the road

TABLE I.—TABLE OF SPECIFICATIONS FOR SHEET METAL CULVERTS.

Diameter.	Min. sheet thickness. In.	Min. rivet. In.	Max. rivet spacing. Ins.
*12-in	1/4	1/2	6
15-in	1/4	1/2	6
18-in	1/4	1/2	6
24-in	1/4	1/2	6
30-in	1/4	5/8	6
36-in	3/8	5/8	6
42-in	7/16	5/8	6

*Farm crossings only.

ditches are in effect drainage canals for large areas abutting the road.

Figure 3 illustrates a cross section used when a gravel or macadam surface is applied to a graded earth road. The type of gravel generally found is sandy in nature, containing only a small percentage of large aggregate and difficulty is sometimes experienced in bonding it. In Fig. 5 is shown a gravel road finished to standard cross section. Figure 6 illustrates cross sections for macadam and concrete roads.

supervising construction. Plans show all important features, including in all cases the drainage area of waterways, and are prepared in a standard manner, Fig. 7, upon sheets 22 ins. by 34 ins. in size.

County road maps prepared by the county engineer showing the location of the roads designated as the county system with information concerning them, are filed with the commission. In Fig. 8 (p. 58) is shown a standard type of county road map.

Waterways.—One of the most important features of the work of the commission is the preparation of plans for waterways and the supervision of their construction. The methods and standards employed in the construction of pipe and box culverts are given here. A discussion of steel and concrete bridges in use will appear at a later date in the Bridge Section of ENGINEERING AND CONTRACTING.

Fig. 5. Rolling Gravel Road Built to Standard Cross Section.

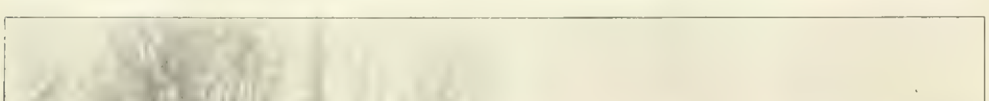


Fig. 4. A Problem Frequently Encountered in the Relocation of Old Roads.

Road Plans and County Maps.—Road plans are prepared by the county engineer and submitted to the commission for approval. The division engineer co-operates with the county engineer in the preparation of plans and in the subsequent work of letting contract and

Loads.—For concrete structures the minimum concentrated load is assumed as a 15-ton traction engine, the load being distributed as follows: 20,000 lbs. on rear wheels, 10,000 lbs. on front wheels, 11 ft. between axles, 6 ft. between center of back wheels, width of wheels 22 ins. Each back wheel load is assumed as distributed over an area 6 ft. wide and 5 ft long. For thin slabs on girders the area of distribution is 4 ft. by 4 ft. in size. An alternate uniform live load is assumed at 100 lbs. per square foot of roadway and sidewalks.

Roadway.—In figuring the length of culverts side slopes are taken at 1½ to 1 and the minimum lengths recommended are as follows:

Item.	Span, ft.	County roads, ft.	Township roads, ft.
Pipe culverts	1—4	*	*
Box culverts	2—16	24	20
Slab bridges	16—25	20	18
Girder bridges	16—60	20	18

*To meet side slopes of fill.

Slab and girder bridges are cambered 1/20 in. per foot of length.

Permissible Stresses.—The following stresses were used in designing small concrete structures, including slab and girder bridges:

Concrete: Compression, 600 lbs. per square inch; tension, none; shear, 100 lbs. per square inch (diagonal tension). Steel: Tension, 16,000 lbs. per square inch; compression, 15 times surrounding concrete.

Footings.—Footings are ordinarily carried down 4 ft. except for spans under 16 ft., which may be built without footings, but with a heavily reinforced floor distributing the pressure on the foundation. Under heavy fills floors are reinforced longitudinally the full length of the structure. The foundation pressure is not ordinarily to exceed 2 tons per square foot. In drainage ditches on flat grades the top of the footing is level with the ditch grade line. Curtain walls at each end of the floor are used and a curtain wall used across the end of the wings on the down-stream side. Where used, piles are ordinarily driven 3 ft. on centers, with 12 ins. to 18 ins. of unbruised wood projecting into the footing, and are computed to carry the full load.

Concrete.—A 1:2:4 concrete is used throughout except in footings, where a 1:2½:5 concrete may be used. The maximum size stone used are 1½ in. for 1:2:4 and 2½ for 1:2½:5 concrete. Rubble stone are permitted in concrete sections 2 ft. thick or over.

The following tabulation shows the quantities of material specified for various sand contents. These quantities are varied slightly to fit conditions:

For 1 cu. yd. of concrete	Cement, bbls.	Sand, cu. yds.	Stone, cu. yds.
1:2:4	1.46	0.44	0.89
1:2½:5	1.19	0.46	0.91
1:3:6	1.00	0.46	0.92
1:2:3½	1.68	0.47	0.83
Hand rail	2.84	1.00	....

For gravel concrete the proportions are as follows:

Sand passing ⅛-in. screen, per cent.	Cu. ft. of pit run gravel required for 1 sack of cement. 1 : 2 : 4 concrete, cu. ft.	1 : $2\frac{1}{2}$: 5 concrete, cu. ft.
42	4.50	5.5
45	4.00	5.0
55	3.50	4.5
65	3.00	4.0
75	2.50	3.5
85	2.25	3.0
95	2.00	2.5

Extensive tests are now being made to determine the value of this table, and it is subject to correction after tests are completed.

TABLE II.—TABLE OF SPECIFICATIONS FOR CORRUGATED CULVERTS.*

Diameter.	Min. gage.	Min. rivet. In.	Max. rivet spacing.	Remarks.
12-in...	16	3/8	1 rivet in each corrugation.	This size for farm driveways only.
15-in...	14	[illegible]	1 rivet in each corrugation.	
18-in...	14	3/8	1 rivet in each corrugation.	
24-in...	14	3/8	1 rivet in each corrugation.	
30-in...	12	[illegible]	1 rivet in each corrugation.	
36-in...	12	3/8	1 rivet in each corrugation.	Not to be used where fill over culvert exceeds 6 feet.
42-in...	12	3/8	2 rivets in each corrugation.	Not to be used where fill over culvert exceeds 4 feet.

Note.—Corrugated pipe for use on township roads and farm driveways only, or for temporary repairs on county roads.

*Adopted for temporary use pending results of thorough field examination.

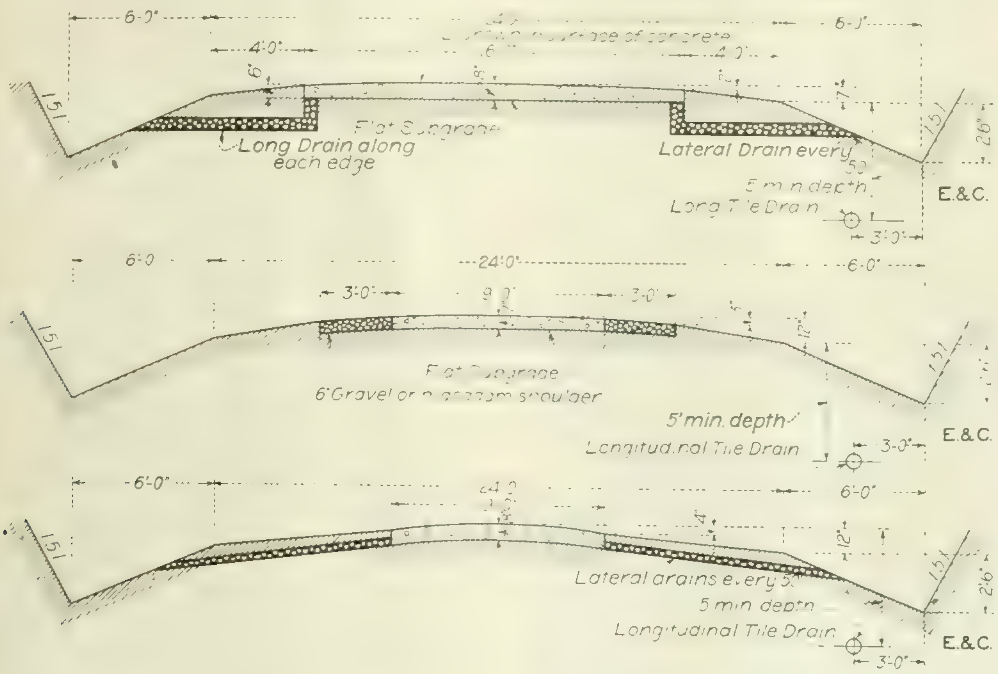


Fig. 6. Cross Sections for Concrete Roads.

Pipe Culverts.—Tables I and II give the specifications under which sheet metal and corrugated culverts are used. Figure 9 (p. 59) gives the details of headwalls. Note the difference in type of headwall when used under a heavy fill.

The standard designs for reinforced concrete pipes are shown in Fig. 10 (p. 60), the dimensions of headwalls being the same as those for corrugated pipe culverts. Wall thickness and reinforcing are subject to change after the completion of tests.

Monolithic pipe culverts are shown in Fig. 11 (p. 61). Figure 12 gives a typical staking plan and illustrates the method of cutting reinforcing steel for a 42-in. culvert. Table III (p. 60) gives the quantities for this culvert and the dimensions and quantities for other sizes of circular openings.

Box Culverts.—In Figs. 13 and 14 (p. 62, 63,) are shown the standard type of box culverts. Two types of wing wall are used, the straight wall being used generally with a slight fill and the flared type under a heavy fill. In Table IV. (p. 61), the quantities of concrete materials and bill of reinforcing for the culverts illustrated in Fig. 13 are given.

Work of the Commission.—Up to the present time the efforts of the commission have been directed in a large measure toward perfecting an efficient organization in the various counties and the standardization of bridge and culvert work. In addition to this a large amount of work consequent on the selection and approval of the various county road systems has been accomplished. In studying the various county road systems much care was necessary to see that they connected up properly and to secure a thorough distribution of main arteries over the various counties.

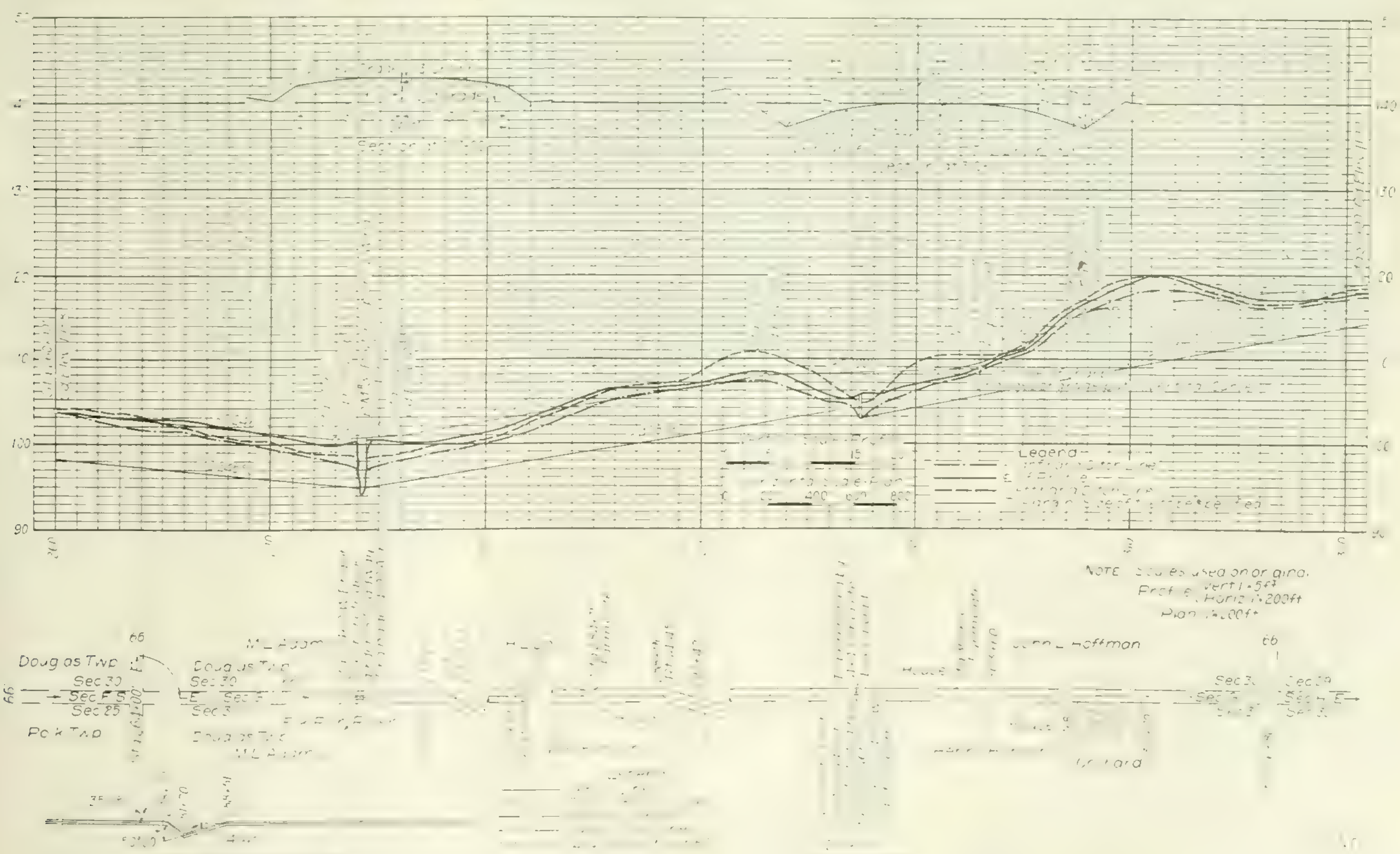


Fig. 7. Typical Iowa Road Plans Prepared by County Engineer.

An important work which has been undertaken and in which good results appear to have been secured, is the abolition of grade crossings. One field engineer and Mr. Beard of the commission, who is an attorney-at-law, have been actively engaged in this work.

We are indebted to T. H. MacDonald, state highway engineer of Iowa, and his assistant, J. S. Dodds, who supervises the educational work of the commission, for the information contained herein.

The Width, Building Line and Frontage Requirements of City Streets.

The consideration of width, building line and frontage of city streets is often complicated by local conditions which make it difficult to establish definite methods and rules to follow in designing pavements and drawing building regulations. A study of the economic factors involved and an analysis of an ideal arrangement is of value in establishing principles. Such an analysis was made by Raymond Unwin in a paper before the Institution of Municipal Engineers and reprinted in the London "Surveyor" and is given here in part.

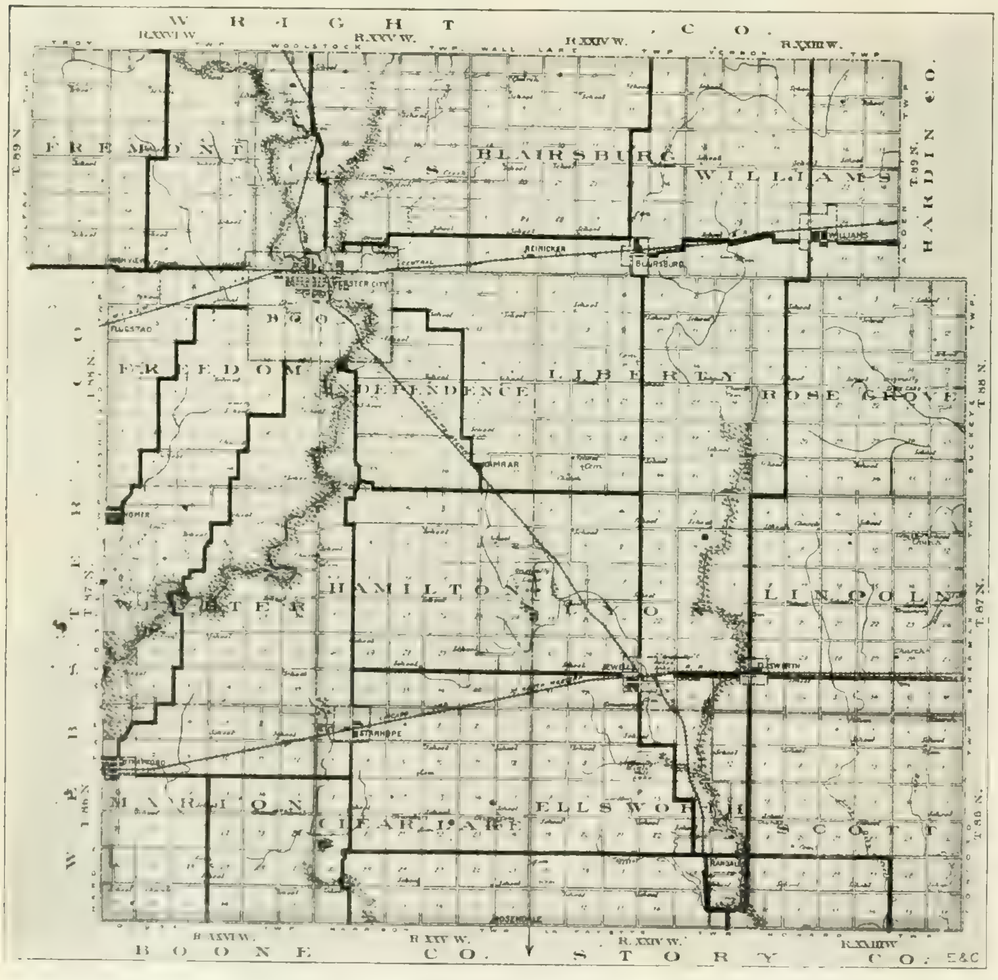


Fig. 8. Typical County Road Map Prepared by County Engineer. Heavy Lines Indicate County Road System. Scale of Original ¼ In. = 1 Mile.

STREETS.

Streets accommodate traffic and afford frontages for buildings. Their treatment depends largely on the relative importance of the two functions: those that are required to serve as main roadways for traffic should be planned primarily for the convenience of traffic, and their junctions should be considered from this point of view first; those, on which the traffic is mainly concerned with the buildings fronting upon them, should be considered primarily from the point of view of affording economical, serviceable, and therefore valuable building frontages.

As far as possible residence streets should either follow approximately the lines of the contours or should travel at right angles to those lines, so that the plane of the land, in whichever direction it may slope, may be nearly square with the building. Both the satisfactory placing of the building on the ground and the convenient treatment of lawns depend considerably upon this. Not only is the importance of traffic secondary in these roads, but it is even desirable that they should not be planned so as to afford short cuts for main lines of traffic.

The only way in which it is possible, without great total cost, to secure adequate width for main arterial roads is to design the minor roads so that they cannot become trafficways; the actual width of the carriageways and footways in these may then safely be reduced to a minimum, and the saving in land and cost thus effected can be set off against the extra width and cost of the main traffic highways. This does not mean, of course, that the width between the building lines on minor roads should be reduced, nor even that the width dedicated to the roadway should always be reduced. There will be cases in which it may be desirable either to provide space for the decoration of the roadways with grass margins, shrubberies, or trees, or to provide space for possible future widening of footways or carriageways, where, owing to the planning of the roads, an increase of traffic seems possible.

It is hardly necessary to emphasize the degree to which the health and attractiveness of dwelling houses depend on the streets. The spacing influences the frontage given to the houses, and the area of open ground available to provide air, light and outlook for the windows. Their adjustment to the nature of the slopes of the ground will have a marked effect on the cost of street per house; careful planning will preserve the residential roads from through traffic, thus reducing both noise, dust and danger, and will, moreover, keep open many attractive views of open spaces and distant objects of interest or beauty. The orderly arrangement of the houses themselves and the decoration of the streets with trees and shrubs will add to the pleasantness of the dwellings.

Where there is a choice between cutting and filling, it may be taken as a general rule that filling is most objectionable in streets where dwelling houses are to be erected and cutting in streets for business premises. Dwelling houses are much better built above the road than below it. On the other hand, business premises must be entered on a level with the street, and can usually employ a basement story profitably; this means that when the street is in a cutting, heavy expense must be incurred in excavating the site, and sometimes also in sinking cellars still further below the level.

The main consideration on the arterial streets, however, must be the traffic. Every needless junction or crossing on a main artery reduces its efficiency as a traffic carrier. Moreover, when arterial streets are broken into by too many minor streets, the facades of the buildings are apt to become ragged, to lack sufficient breadth and dignity. In the minor residential streets the variations which will give further adjustment of the streets to the site, and the limited vistas which will be produced by so planning the streets that, while they divide up the land conveniently, they do not form tempting short cuts, will help the architectural effect with the type of domestic building for which they are intended. A straight approach on an axial line to an important building, and the development of a group of public buildings on such an axial line, will be found to give a sense of order and dignity. Certain types of building are usually characterized by more handsome architectural treatment; it is desirable that the most should be made of these opportunities, and that these buildings should, as far as possible, be placed where they will serve as central features for architectural groupings from several points of view.

In most localities there are features connected with the history of the city's development in commerce, civilization, or culture, around which there cling the most cherished associations of the people, or which keep alive the memory of heroic citizens of the past, which should be considered.

NUMBER OF HOUSES PER ACRE.

I am inclined to think that the best arrangement for limiting the number of houses to the acre is to fix units of reasonable size, taking, as far as possible, areas that are defined on the town plan by main roads, railways, streams, or other definite features not likely to be changed. There should, then, in connection with each unit, be fixed a maximum average density for the whole of the unit, and a maximum density for any one acre within the unit.

There is a pressing reason for adjusting the area of land to the size of the house, and this is the increasing ratio which the cost of the site bears to the cost of the house as that cost decreases. In a house costing $7,000, the cost of the site, including the cost of street paving, will usually be no more than one-sixth of the value of the house; in the case of the $800 cottage, the value of the site will not uncommonly be found to represent one-quarter or even one-third of the value of the house. This means that, relatively, the smaller the house the greater is the expense of providing it with a site. If, then, we fix a given limit to the number of houses without reference to their size, we shall add still more to the relatively higher cost of site for smaller dwellings. This is not desirable, for already there is a strong inducement to the builder to build on each plot as big a house as possible, because the bigger house will carry a little more ground rent or site value than a small one, even for the same actual area of ground.

For all these reasons, therefore, it seems to me of importance that the number of houses to the acre should be limited by means of a schedule, which should fix the number in relation to the size of the house, and for the purpose of determining the size of the house one needs to take the effective size, and to eliminate accidental matters, such as the difference between a high-pitched roof or a low-pitched roof, or the difference between a house that has a large cubic space occupied by foundations and one that has a minimum of foundations.

On the whole, therefore, I think the best plan is to fix the number according to the cubic contents of the building, measured to the outside of external walls, to the centre of party walls, and excluding from the measurements all unused space above the level of the highest ceilings and below the level of the ground floor of the building. The schedule would then be somewhat as follows, the actual numbers being adapted to suit the individual circumstances:

Houses containing not more cu. ft. than—	Maximum average to the acre.	Maximum on any one acre.
5,000	[illegible]	[illegible]
6,300	14	19
[illegible]	[illegible]	18
10,000	12	16
12,000	10	14
11,000 and over	8	12

BUILDING LINE.

The building line has considerable bearing upon the building of houses and the architectural effect of the streets, and it is not easy to frame regulations that will work satisfactorily in all cases. There are existing roads on which it may be desirable to fix a building line for the purpose of securing the possibility of road widening at some future date should the traffic on the road develop sufficiently to require it. A line that is fixed for this purpose must, if it is to be effective, apply to all buildings, and must not be subject to exceptions on corner sites, or for the purpose of giving a sense of enclosure to the street view at certain points. On the majority of streets that are mainly used for building purposes, and have only to accommodate local traffic, and on new streets laid out of ample width, the question of street widening beyond the limits of the street boundaries hardly arises. Here the building line is fixed for the purpose of providing adequate width between the rows of houses to secure light and air, and the amenity of the streets; for this last purpose it is desirable that there shall be some opportunity for varying the building line. In some schemes the minimum amount of set-back which could be accepted for any building is all that has been secured, leaving anything further entirely to the option of the builder; in other schemes an attempt has been made to secure a more generous depth of forecourt over the general frontage of the street, but at the same time to give opportunities both to bring forward certain limited lengths of frontage here and there, and also to bring forward, to some extent, the houses at the corner of two streets, so that the amount of paving which must be wasted at street corners may not be unduly increased. While it is desirable that there should be a settled building line it is not desirable that houses should be dotted down along the road irregularly, just where each individual builder may think fit. For this reason, perhaps, the best solution of the building line difficulty is to require a minimum length of building frontage.

Where the land slopes steeply it is not at all desirable that the buildings on each side should have to be an equal distance from the center of the road; generally speaking, and where land in several ownerships has to be dealt with, the building lines may need to be set out equidistant from the center of the road; but provision should be made for cases where it is desirable that houses should be set much nearer the road on one side, generally where the ground is falling from the road. This may be compensated for by setting back the houses on the higher side of the road, the distance between the two building lines remaining the same.

Adequate frontage should be provided for each house, and long projecting wings should not be permitted in such a way as to block proper access of light and air to the windows. A simple regulation should require that all the living room windows have a certan angle of light vertically and horizontally; such angles should not be less than 60 degrees. This would allow reasonable projections to buildings, but would prevent their undue extension in length, or their undue proximity to each other in the manner that is so objectionable in a very large number of buildings erected under the ordinary building laws.

Where an attempt is made to fix the height of buildings, or to limit the number of stories, care must be taken that this is not effected in such a rigid manner as to require on a sloping road the stepping down of every building. Provision should be made that a level roof may be carried over a group of four or six buildings, and the extra height of the lower buildings utilized for an additional story.

Dust Prevention Methods on District of Columbia Suburban Roads.

There are from 50 to 75 miles of country roads within the District of Columbia which are subjected to heavy traffic and are treated for dust prevention during the spring and summer. For the most part these roads have a trap or limestone macadam surface. The methods employed in dust prevention were described by L. R. Grabill, superintendent of suburban roads, District of Columbia, during a discussion at the last meeting of the American Road Builders' Association.

Dust prevention work is started in the spring, about April 15, with the application of a heavy asphaltic oil. The work is accomplished entirely by force account. Two means of distribution are used: (1) a Johnson sprayer attached behind a tank wagon drawn by a steam roller; and (2) a Monarch spraying wagon which distributes directly, pressure being obtained from a pump geared to the wheels. It is found that the latter appliance covers as much ground as the former but uses a little more oil so that it is not quite as economical.

The roads have been treated for several years. There is applied at each application about 1-5 gal. per square yard of asphaltic oil or light coal tar heated to a temperature of 100 to 120°. Both oil and tar are used and they are both good. Oil and tar are bought by contract, and the product varies slightly, but as a rule is fairly uniform.

The distributing wagon pulled by the road roller is supplied by wagons which bring the material from the cars, and as fast as one wagon is empty it is dropped and another put in, and the roller proceeds. The roller will cover about [illegible] sq. yds. per day in an average day's work of 8 hours. If oil costs 5 cts. per gallon, which is about the average price, 1-5 gal. costs one cent for oil. Oil is covered immediately with sand. One-half of the width of the road is oiled at a time, covered with sand immediately and thrown open to travel while the other half is being covered.

Before the application the road is swept thoroughly with horse brooms, and if there are any rough spots or small dirty spots left, they are cleaned up with hand brooms.

In covering with the sand, a coarse torpedo sand, as coarse as obtainable, is used, which costs about $2.00 per cubic yard at the job, and 1 cu. yd. to 200 sq. yds. of the surface is applied. The sand costs approximately 1 ct.

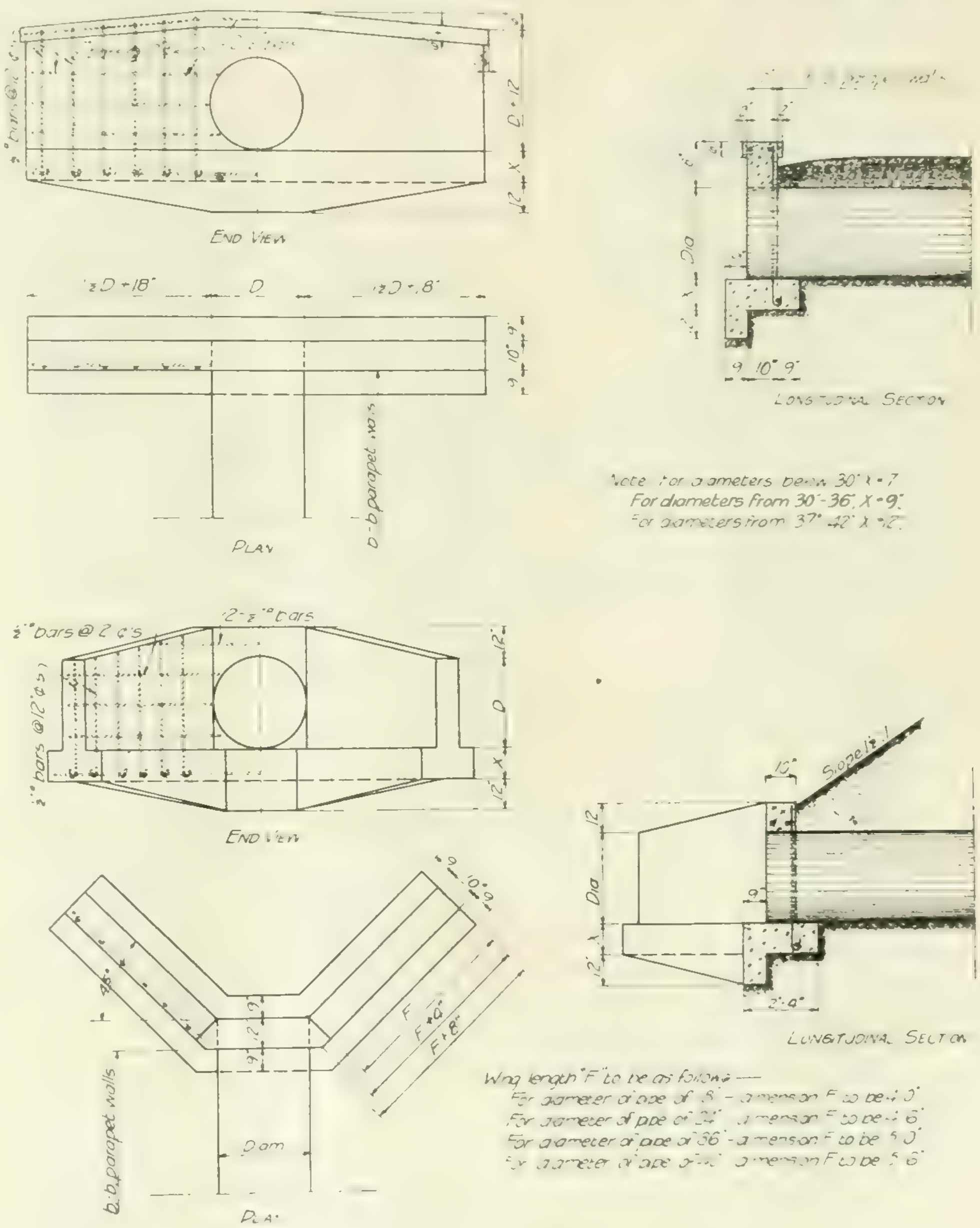


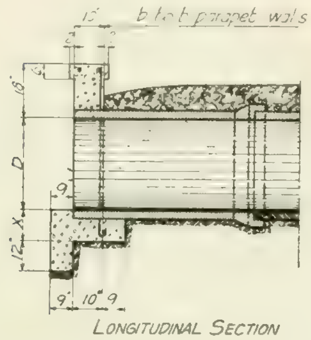
Fig. 9. Headwalls for Metal Pipe Culverts.

per square yard. The roller and the outfit which applies the oil costs about $35 a day, and 10,000 sq. yds. applied gives a cost of labor for application of oil and cover, usually, somewhat less than ½ ct., so that the whole cost of one application is about 2½ cts. per square yard.

It is usually necessary to make two applications per season to keep the roads in good condition. In a few places more are required, and there are very few places that require less, so that the cost for the season is about 5 cts. per square yard. That applies either with asphaltic oil or with the light tars, such as Tarvia B.

Where the treatment of the same road with the same class of material is continued for several years, it is found that as low as about 0.1 gal. per square yard can be used for a moderately traveled road. That is probably the lowest cost yet reached for moderate travel; for, if oil or tar costs 5 or 6 cts. per gallon, and only 0.1 gal. per square yard is used, the material costs little. The cost of labor for such applications is about the same as where more is applied.

As a rule it is found the Tarvia B treatment requires, on the moderately traveled roads, only one application per year where possibly the oil treatment would require two.



Note - For diameters below 30", X=7"
For diameters from 30"-36", X=9"
For diameters from 37"-42", X=12"

This point to be plainly marked on outside of pipe so that it may be placed at top of ditch.

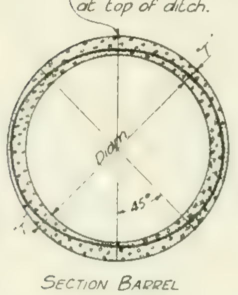


TABLE OF DIMENSIONS FOR CIRCULAR PIPE

Diameter	Thickness of Shell = T	Reinforcing A S & W Co's Δ Mesh or other mat'l of equivalent section
15 in	2.25 in.	A S & W Co's Style 6 = .058" per ft width
18 in	2.50 in	" 5 = .077" "
24 in	3.00 in	" 4 = .102" "
30 in	3.50 in	" 42 = .151" "
36 in	4.00 in.	" 23 = .170" "
42 in	4.50 in	" 32 = .225" "

Joints to be standard bell and spigot or other standard type to be approved by the Highway Commission

Fig. 10. Dimensions of Reinforced Concrete Pipe.

TABLE III.—DIMENSIONS AND QUANTITIES FOR CIRCULAR PIPE CULVERTS.

Dimensions.

Size of opening Ins.	X	Y	S	T	H	K
18	3' ½"	4' 0"	5½"	6"	1' 6"	1' 4½"
24	3' 8"	4' 6"	6"	7"	2' 0"	1' 5"
36	4' 6"	5' 0"	7"	9"	3' 0"	1' 5½"
42	See Fig. 12					

Quantities.

Concrete.

18-in. opening. Concrete materials. Total concrete, 5.4 cu. yds.

Item.	Quantity required for culvert 20 ft. b.-b. parapet walls.	Additional amount per foot additional length of barrel.
	If sand and stone are used.	
Sand	2.38 cu. yds.	.055 cu. yds.
Stone	4.80 cu. yds.	.112 cu. yds.
Cement	7.9 bbls. or 31.6 sacks.	.182 bbls. or .73 sacks.
	If gravel alone is used.	
Gravel	6.2 cu. yds.	.125 cu. yds.
Cement	10.25 bbls. or 41.0 sacks.	.238 bbls. or .95 sacks.

Steel.

Bill of reinforcing for culvert 20 ft. b.-b. parapet walls.

Mark.	Position.	No.	Size. In. sq.	Length. Ft.	Length. Ins.	Wt. Lbs.	Remarks.
A	Transverse barrel	22	½	4	10	90	
B	Longitud. barrel	4	½	28	6	97	Continued into wings.
E	Longitud. barrel	1	½	21	6	18	Center bar.
C	Transverse floor	12	½	2	3	23	
D	Longitud. floor	4	½	23	0	78	
E	Vertical wings	8	½	5	6	38	Cut in field.
F	Horizont. parapet	2	½	5	6	9	Continued into wings.
G	Horizont. footings	4	½	4	0	14	

Total reinforcing steel 367 @ 12.3 lbs. per ft. barrel.

24-in. opening. Concrete materials. Total concrete, 7.6 cu. yds.

Item.	Quantity required for culvert 20 ft. b.-b. parapet walls.	Additional amount per foot additional length of barrel.
	If sand and stone are used.	
Stone	6.75 cu. yds.	.163 cu. yds.
Sand	3.35 cu. yds.	.08 cu. yds.
Cement	11.1 bbls. or 44.5 sacks.	.267 bbls. or 1.07 sacks.
	If gravel alone is used.	
Gravel	7.6 cu. yds.	.183 cu. yds.
Cement	14.4 bbls. or 57.5 sacks.	.347 bbls. or 1.39 sacks.

Bill of reinforcing for culvert 20 ft. b.-b. parapet walls.

Mark.	Position.	No.	Size. In. sq.	Length. Ft.	Length. Ins.	Wt. Lbs.	Remarks.
A	Transverse barrel	22	½	6	4	118	
B	Longitud. barrel	1	½	22	0	19	
B	Longitud. barrel	4	½	29	4	100	Continued into wings.
C	Transverse floor	12	½	2	9	28	
D	Longitud. floor	4	½	23	0	78	
E	Vertical wings	10	½	6	10	58	Cut in field.
F	Horizont. parapet	2	½	8	0	14	Continued into wings.
F	Horizont. parapet	2	½	4	0	7	Continued into wings.
G	Hor. footing wings	4	½	4	6	15	

Total reinforcing steel 437 @ 14.2 lbs. per ft. barrel.

36-in. opening. Concrete materials. Total concrete, 12.2 cu. yds.

Item.	Quantities for culvert 20 ft. b.-b. parapet walls.	Additional amount per foot additional length of barrel.
	If sand and stone are used.	
Stone	10.88 cu. yds.	.285 cu. yds.
Sand	5.36 cu. yds.	.141 cu. yds.
Cement	17.8 bbls. or 71.2 sacks.	.47 bbl. or 1.88 sacks.
	If gravel alone is used.	
Gravel	12.2 cu. yds.	.32 cu. yds.
Cement	23.2 bbls. or 92.8 sacks.	.61 bbl. or 2.44 sacks.

Bill of reinforcing for culvert, 20 ft. b.-b. parapet walls.

Mark.	Position.	No.	Size. In. sq.	Length. Ft.	Length. Ins.	Wt. Lbs.	Remarks.
A	Transverse intrad	22	½	9	0	169	
B	Longitud. intrados	6	½	31	0	157	Continued into wings.
B	Longitud. intrados	1	½	21	6	19	Center bar.
C	Transverse floor	12	½	4	0	41	
D	Longitudinal floor	4	½	23	0	78	
E	Vertical wings	10	½	9	2	78	Cut in field.
F	Horizont. parapet	2	½	10	6	18	Continued into wings.
F[1]	Horizont. parapet	2	½	5	6	10	Continued into wings.
G	Hor. footing wings	4	..	5	0	17	

Total reinforcing steel 587 @ 18.75 lbs. per ft. barrel.

Note.—Bars F and F[1] cut from 16' bar.

42-in. opening. Concrete materials. Total concrete, 16.2 cu. yds.

Item.	Quantities for culvert 20 ft. b.-b. parapet walls.	Additional quantities per ft. additional length of barrel.
	If sand and stone are used.	
Stone	14.40 cu. yds.	.392 cu. yds.
Sand	7.12 cu. yds.	.194 cu. yds.
Cement	23.6 bbls. or 94.4 sacks.	.643 bbl. or 2.57 sacks.
	If gravel alone is used.	
Gravel	16.2 cu. yds.	.44 cu. yds.
Sand	30.8 bbls. or 123.2 sacks.	.835 bbl. or 3.34 sacks.

Bill of reinforcing for culvert, 20 ft. b.-b. parapet walls.

Mark.	Position.	No.	Size. In. sq.	Length. Ft.	Length. Ins.	Wt. Lbs.	Remarks.
A	Transverse intrad	22	½	11	0	206	
B	Longitud. intrados	1	½	21	6	18	Center bar.
B[1]	Longitud. intrados	8	½	31	10	216	Continued into wings.
C	Transverse floor	12	½	4	6	46	
D	Longitud. floor	4	½	23	0	73	
E	Vertical wings	12	½	10	7	108	Cut bars in field.
F	Horizont. parapet	2	½	16	0	27	Cont. into wings. (See note)
G	Horiz. footing wings	4	½	5	6	19	

Total reinforcing steel 718 @ 22.2 lbs. per ft. barrel.

Note.—Bars "F" consist of two bars, one 6' and one 10' in each parapet; these to be cut from 1-16' bar.

Note.—For each additional foot of barrel use one additional bar "A," one-half bar "C," and add one foot length to bars "B," "B[1]" and "D."

Note.—The above table gives theoretical quantities required. To allow for shrinkage add 20 per cent to the volume of sand and stone or gravel given in the table.

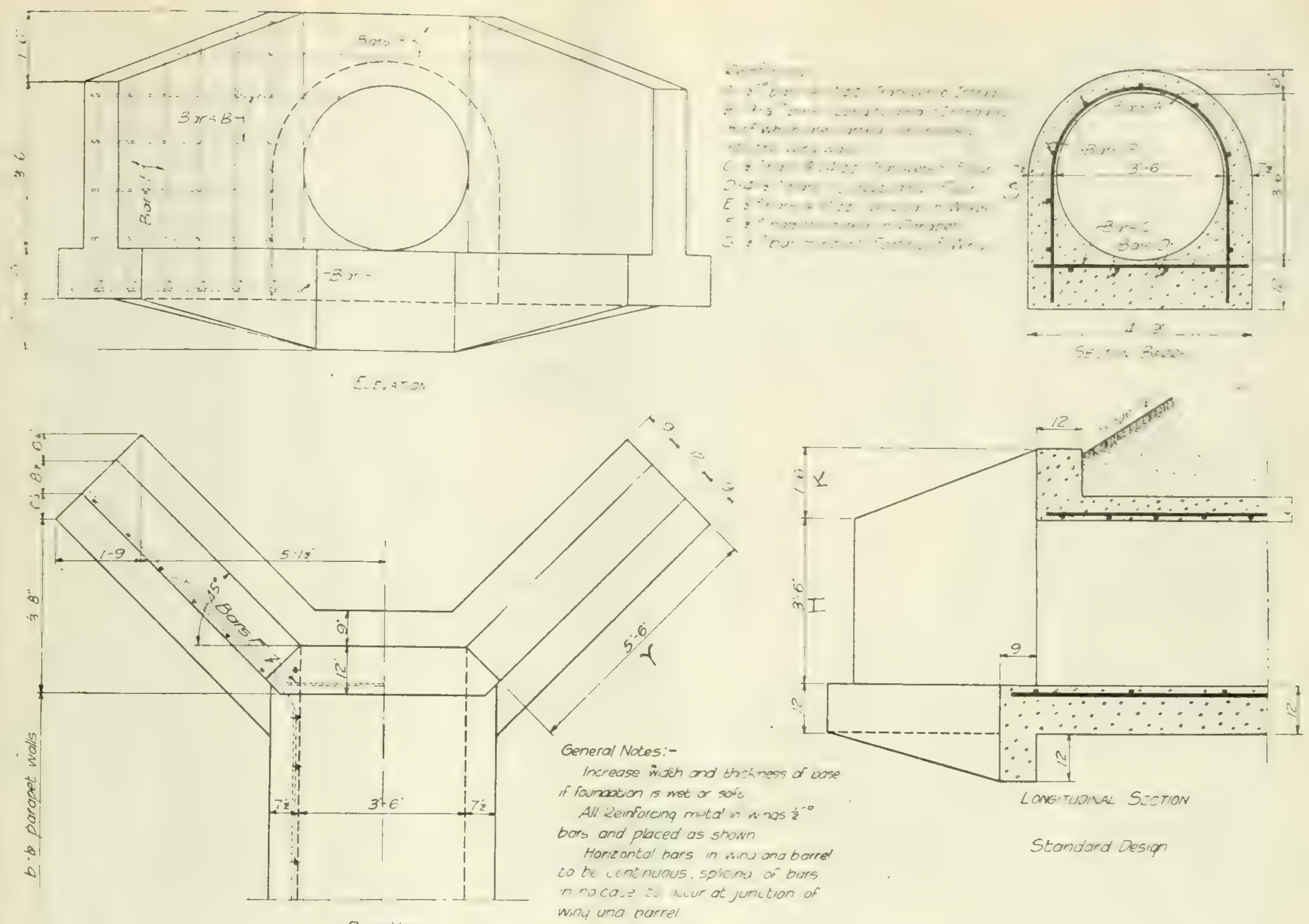


Fig. 11. Standard Design for Monolithic Concrete Pipe Culverts.

TABLE IV.—BILL OF REINFORCING AND QUANTITIES OF CONCRETE MATERIAL IN 6 FT. BY 6 FT. BOX CULVERTS.

Item.	Flared wing walls. Quantity required for culvert 20 ft. b.-b. parapet walls.	Straight wing walls. Quantity required for culvert, 20 ft. b.-b. parapet walls.	Straight wing walls. Additional amount per foot additional length barrel.
If sand and stone are used—		If sand and stone are used—parapet walls.	
Sand	16.0 cu. yds.	16.5 cu. yds.	0.45 cu. yds.
Stone	32.5 cu. yds.	33 cu. yds.	0.90 cu. yds.
Cement	53.5 bbls. = 214 sacks	54 bbls. = 216 sacks.	1.5 bbls. equals 6 sacks.
If unscreened gravel alone is used—		If unscreened gravel alone is used—	
Gravel	36.5 cu. yds.	37 cu. yds.	1.0 cu. yd.
Cement	69.5 bbls. = 278 sacks	70 bbls. = 280 sacks.	2 bbls. = 8 sacks.
Total concrete, 36.4 cu. yds.		Total concrete, 36.8 cu. yds.	

Bill of Reinforcing for Culvert, 20 ft. b.-b. Parapet Walls.

Mark.	Position.	Flared wing walls. No.	Size, sq.	Length, ft.	ins.	Wt., lbs.	Remarks.	Straight wing walls. Length, ft.	ins.	Wt., lbs.	Remarks.
A	Transverse in slab....	60	½	7	0	357		7	0	357	
B	Corner bars in slab...	120	½	4	0	[illegible]		4	[illegible]	[illegible]	
C	Longitudinal in slab ..	4	½	21	6	73		21	6	[illegible]	
D	Vertical in wall.	46	½	[illegible]	3	[illegible]		8	3	[illegible]	
E	Horizontal in wall....	12	½	[illegible]	0	[illegible]	Bent around into wings.	34	0	[illegible]	Bent around into wings.
F	Transverse in floor...	23	½	[illegible]	[illegible]	176		9	0	176	
G	Longitudinal in floor.	4	½	21	6	73		21	6	[illegible]	
H	Vertical in wings.....	14	½	16	0	[illegible]	To be cut in field.	19	0	[illegible]	To be cut [illegible] in field.
I	Vertical in parapet...	10	½	1	6	[illegible]		1	6	[illegible]	
J	Footing bars in wings.	4	½	8	0	[illegible]		8	0	[illegible]	
K	Horizontal in parapet.	2	½	7	0	[illegible]		[illegible]	0	[illegible]	
	Total reinforc. steel.	..	..	2,364	0	2,003	@ 85 lbs. per foot.	2,420	6	[illegible]	[illegible] lbs. per foot.

Note.—The table gives the theoretical quantities required. To allow for shrinkage add 20 per cent to the volume of sand and stone and gravel given in the table.

General Note.—For each additional foot of barrel use three addition bars A, six additional bars B, one additional bar F, two additional bars D, and increase bars C, E and G each by one foot.

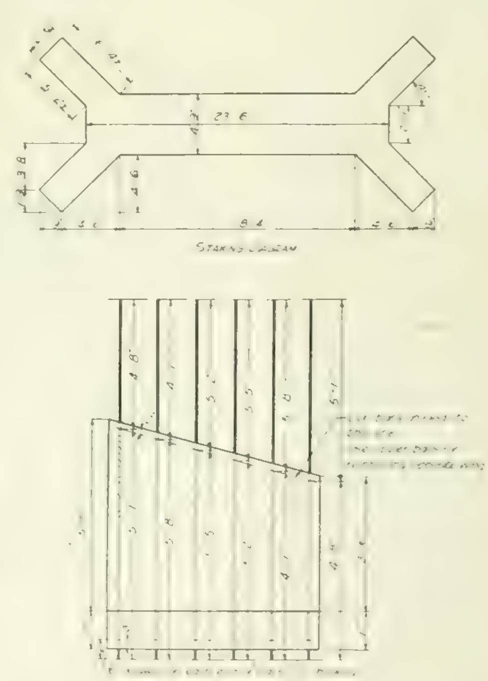


Fig. 12. Typical Plan for Cutting Reinforcing Steel and Staking Out 42-in. Pipe

Municipal Paving Plant.—A city paving plant is to be established at Edmonton, Alta., at a cost of $25,000. It is planned to lay at least 100,000 sq. yds. of paving before the close of the season. This will provide work throughout the summer and fall months for a large number of men now unemployed. The entire paving program for this year involves a total expenditure of more than $1,000,000, part of which will be financed by the Hudson's Bay Co. The city will pave several streets on the Hudson's Bay reserve, in the northern and eastern portion of the city, where more than $3,000,000 worth of business and residence property was sold two years ago.

Cost of Asphalt Paving Repair in St. Paul, Minn.

The equipment used, operating force and cost of asphalt paving repair by force account work is given in recent reports of the city of St. Paul. The city owns an asphalt plant consisting of a Warren Brothers portable asphalt plant, one 8-ton asphalt steam roller, one scarifier, one Lutz surface heater, one fire wagon, one gyratory stone crusher, two portable melting kettles, six 2-cu. yd. steel lined asphalt wagons, four ¾-cu. yd. concrete spreaders, one set of curb cutter's tools, nine asphalt rakes, testing scales and the necessary small tools.

The plant was operated during the season of 1912, a total of 92 days, and 19,428 sq. yds. of asphalt pavement were turned out, 15,040 sq. yds. of this being cut out work and 4,388 sq. yds. burner work, also 5,458.63 sq. yds. of asphalt were put in for paving contractors in repairing asphalt pavements under guarantee; of this 2,363 sq. yds. were cut out work and 3,095 sq. yds. burner work. The total cost was $6,012.96 for repairing guaranteed work.

All asphalt paving repairs for the year 1913 were made by the municipal asphalt plant. The plant was put in operation March 30 and during the season worked 178 days, and 44,193.71 sq. yds. of asphalt paving were turned out, 43,296.36 sq. yds. of this being cut out work and 897.35 sq. yds. burner work; also 16,832.42 sq. yds. of asphalt were put in for the Barber Asphalt Paving Co. on streets still under guarantee; of this 16,039.51 sq. yds were cut out work, and 792.91 sq. yds. were burner work at a total cost of $21,613.34.

A total of 7,370.13 sq. yds. of asphalt were put in for the City Ry. Co., the cost being $11,031.76. 1,249.78 sq. yds. of asphalt and 147.79 sq. yds. of concrete foundation were put in for public service corporations; 246 sq. yds. were put in on bridges on which the city maintained the surface, which cost $430.49, or $1.75 per square yard; also 8.2 sq. yds. were placed in openings in the streets made by the sprinkling department, which cost $14.35, or $1.75 per square yard.

The operating crew at the plant consisted of one foreman, one engineer, one tank man, four laborers and a night watchman; four teams were employed hauling asphalt from the plant to the work.

The street crew was made up of one foreman, one timekeeper, one roller man, two rakers, two tampers, one smoother and one cement man laying the new pavement; and two shovelers, six scrapers and two teams removing and hauling the old paving. The total expense was divided as follows:

Item.		Cost
Operation of plant, labor		$ 5,889.02
Fuel		1,024.17
Hauling material		1,559.18
Superintendence, livery, watchman, etc.		3,164.21
Repairs and supplies		1,658.05
Material		26,876.59
Street crew labor		8,206.66
Hauling material to street		5,068.40
Engineer and watchman		1,391.[illegible]
Tools, repairs, etc.		790.05
Total		$55,628.28
Total labor	$25,175.66	
Total material	30,452.62	
		$55,628.28
Charged to outside parties	34,194.23	
Charged to bridges	430.49	
Material on hand	2,512.71	
		37,137.43
Total cost of city work		$18,490.85

The following shows cost and data regarding asphalt repairs for the year 1913:

Total area of pavements on which repairs were made, in sq. yds.	222,327
Area of repairs, in sq. yds.	18,733.18
Per cent of area repaired	8.42
Cost of repairs	$18,921.34
Average cost per sq. yd. of total area.	0.085

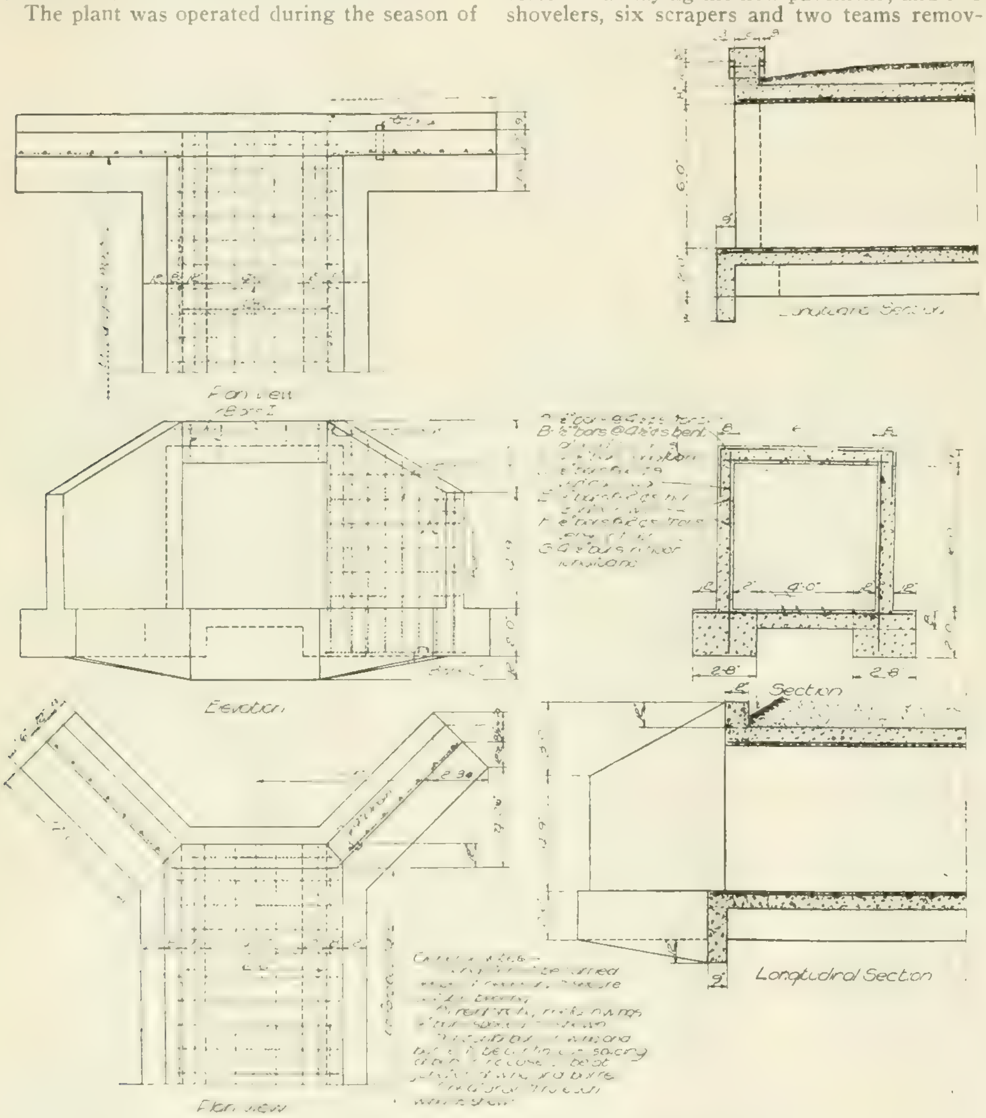


Fig. 13. Standard Design for a 6-ft. by 6-ft. Box Culvert with Straight and Flared Wings.

Division Organization for Road Maintenance in Massachusetts.

The organization for maintenance in one division of the Massachusetts highway system is given by F. C. Pillsbury in a paper before the American Road Builders' Association. In this division ordinary maintenance is separated from reconstruction. This was done because reconstruction involved so many special problems of traffic and special surfaces that it seemed advisable to have a special engineer to superintend this work. Much of the reconstruction has been accomplished by hired labor directed by foremen in the employ of the commission. This method was adopted because it was found that contractors bidding were not sufficiently skilled in new methods to accomplish the work economically. This method of work has been satisfactory and may be recommended for comparatively small areas.

Ordinary maintenance includes the care of trees, sidewalks and supervision of everything within the highway location, including underground and overhead public or private structures, and this has required a subdivision of the ordinary maintenance department. The organization of the maintenance department is as follows for ordinary maintenance under an engineer-superintendent.

First subdivision:

(1) Assistant in charge of permits, including supervision of all overhead and underground structures other than those pertaining to the structures of the highway commission on the highway itself. (Under this assistant are constantly employed one or more inspectors, who in number correspond with the volume of the work and are obtained by temporary employment, or by transfers temporarily from other departments.)

Second subdivision:

(1) An assistant, being an engineer, who has charge of special light oiling work, as well as giving assistance wherever possible when he has the time, and when necessary.

(2) Local maintenance foremen or repair agents, as we have called them, who have charge of from one to several different sections of roads.

(3) Patrol gangs or section gangs, consisting of a single team, driver working with or without one or more helpers, in charge of stated sections of road, length varying according to the quantity of work.

(4) Two or more special repair gangs doing the heavy repair work requiring the use of a steam roller, sprayer, etc. (These gangs are movable; they may do any work which would not require a regular reconstruction gang, and are moved to a road when its condition becomes so bad that the small section gang cannot put it in proper condition.)

Bituminous Concrete.—Concerning bituminous concrete the following conclusions by Linn White, civil engineer for the South Park Commissioners, Chicago, are interesting: (1) Bituminous concrete and bituminous macadam of equal thickness require the same amount of bitumen per square yard. (2) Considering the total thickness of wearing surface and base, and disregarding first cost of plant, the only difference in cost between bituminous concrete and bituminous macadam is the difference in labor, amounting to generally less than 10 cts. per square yard. (3) Old macadam is a difficult and uncertain proposition to penetrate with poured bitumen on account of dirty stone, and generally requires a new top layer of stone to produce a successful bituminous macadam, but if of substantial thickness may be utilized as a base for bituminous concrete. (4) The mixing and laying of bituminous concrete may be carried on during damp and cool weather when it would not be practical to construct bituminous macadam. (5) Bituminous concrete is of even thickness and even composition and consequently wears more evenly than bituminous macadam, no matter how well made, and costs less for repair and maintenance. In all these respects the advantage of comparison is in favor of bituminous concrete, except in the one of first cost.

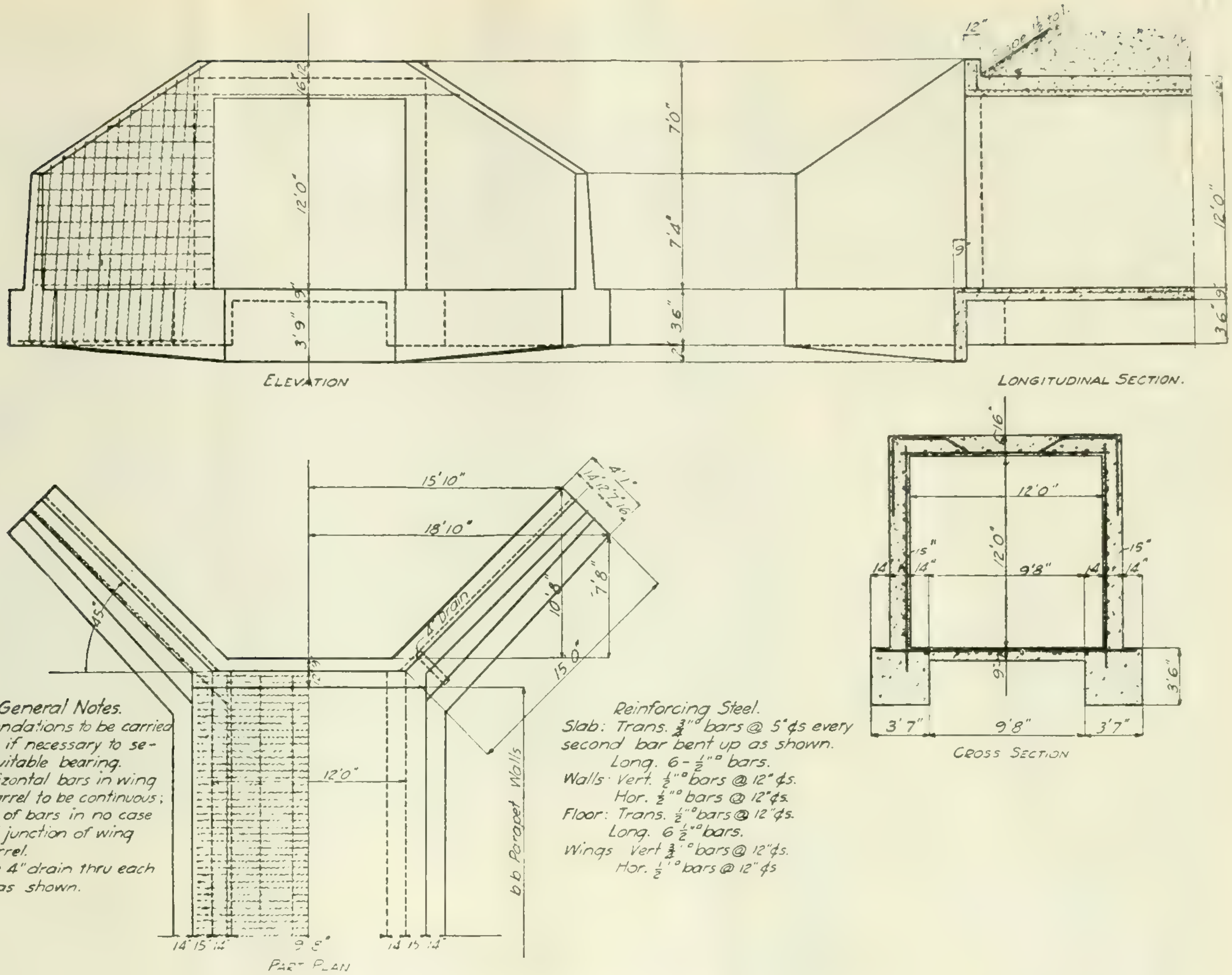


Fig. 14. Standard Design of a 12-ft. by 12-ft. Box Culvert

BRIDGES

Design and Construction Features of the Bietschtal Viaduct in Switzerland.

It is instructive to note how difficult problems of bridge design and erection are solved by the engineers of other countries. The following article describes the design and erection features of an unusual type of bridge, which spans a deep canyon in Switzerland, the data being taken from a paper by Adolph Herzog, in Schweizerische Bautzeitung.

The Bietschtal Viaduct is the latest addition to the many bridges on the Lötschberg Ry., in which variety of construction and boldness of execution are salient features. This structure spans the canyon of the Bietschbach, a stream which reaches the valley of the Rhone at the ancient village of Raron. The accepted design is the result of an advertisement for bids for the design and construction of the viaduct, the bids being asked for in June, 1910, by the general manager of the railroad company. The structure is on a grade of 2.22 per cent, and its center line is on a curve having a radius of 983 ft. It consists of a steel arch, with an adjacent deck truss span at each end, one end of the latter resting on the arch and the other on concrete abutments (see Fig. 1).

DESIGN FEATURES.

It was stated in the specifications that the drawings (a plan showing the location of the center line of the bridge and track, and a longitudinal section along the center line of the bridge) were subject to alteration, and that the bidders were at liberty to submit their own plans, provided they conformed with the characteristics of the locality and the government specifications. It was not permitted, however, to change the location, the curvature of the track, or its elevation. In making the stress calculations the government specifications of Switzerland for the "Calculation of Steel Bridges and Roof Construction," dated Aug. 19, 1892, were used, the rolling load being increased 20 per cent. After considering the various plans and proposals submitted, the contract for the delivery and erection of the steelwork was let to A. Buss & Co., of Basel, who were to make the calculations and the detailed drawings subject to the approval of the Department of Railroads of Switzerland. The final design, for which the contractors received the government's approval on Dec. 8, 1911, provided for a three-centered steel arch of 95 meters (about 311.7 ft.) span, built to carry two tracks. For the present only a single track is used, and this track is carried at each end of the structure on steel trusses of 35.5 meters (about 116.5 ft.) span. For the intended double-track construction, another single-track truss span will be placed alongside each existing span. The distance between track centers is 3.60 meters (about 11.8 ft.).

The supervising office laid particular emphasis on the probability of the use of heavier rolling loads by the time the road would be opened for traffic. It was also required that a liberal allowance should be made for the various parts of the structure, in order to insure that an overstress should not immediately take place.

Figure 1 shows a side elevation of the bridge and of the falsework used in its erection. This drawing indicates the type of construction and gives the general dimensions. Figure 2 shows a plan, a side elevation and an end elevation of the hinged ends of the arch span. These drawings show clearly the construction of the hinges and shoes.

The resultant at the hinge, due to dead load, live load, centrifugal force, braking effect, wind pressure and temperature variations, amounts to about 2,424,000 lbs. This force is transmitted from the chords by means of a cast-iron hood, which is fitted into a milled head (see Fig. 2). The hood is shaped like a pan to receive a forged steel compression piece, which transmits the pressure from the hood to the cast-steel milled head. Between this bearing and the cast-iron masonry plate four pairs of wedges are inserted. By means of these wedges, which are 100 mm. wide and 55 mm. apart, the ball-shaped bearings can be adjusted to the correct position. Horizontal displacement of the hinge plate on the mountain side is prevented by transverse wedges, while the omission of these wedges on the valley side permits the horizontal movement caused by temperature changes in the transverse connections (as far as the temperature forces can overcome the friction between the wedges acting in the other direction). To diminish this friction as much as possible the wedges were planed, polished and greased. The bearings were constructed in such a manner that four hydraulic pumps of 100 tons capacity could be attached between the bed plate and the hood, in order to effect an easy regulation of the wedges.

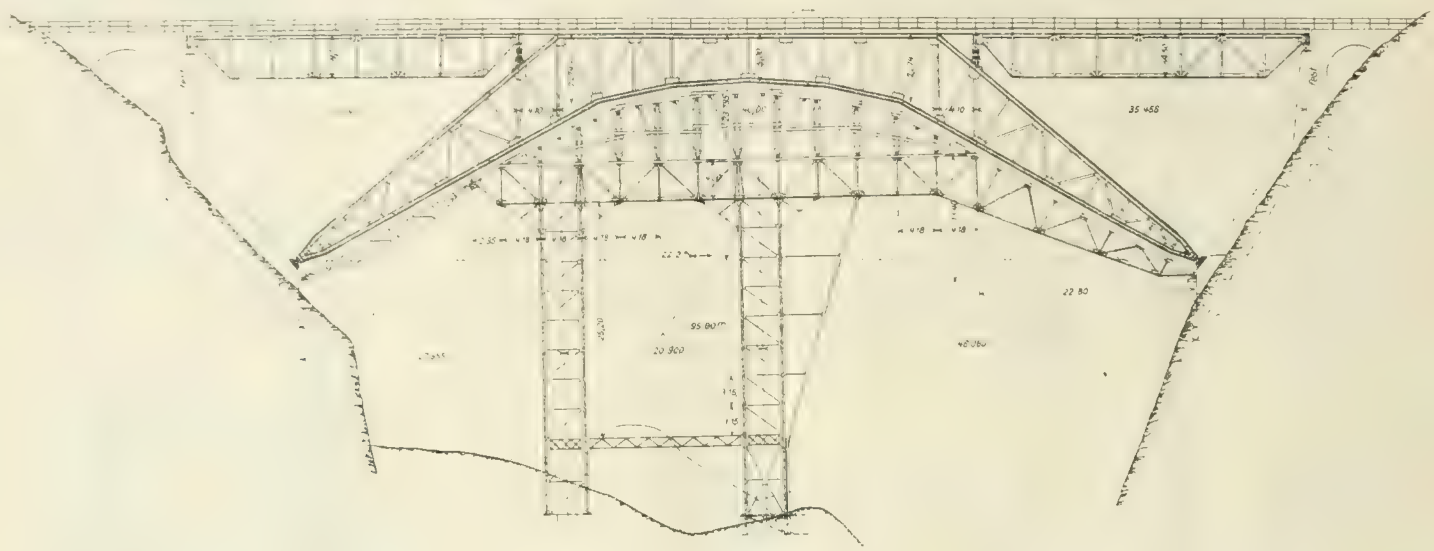


Fig. 1. Plan and Elevation of Bietschtal Viaduct, Switzerland, Showing Type of Construction of Viaduct and Falsework.

Figure 3 shows a cross-section at the center of the arch span. This drawing shows the type of bracing used and the floor construction for the single track which is to be constructed at this time.

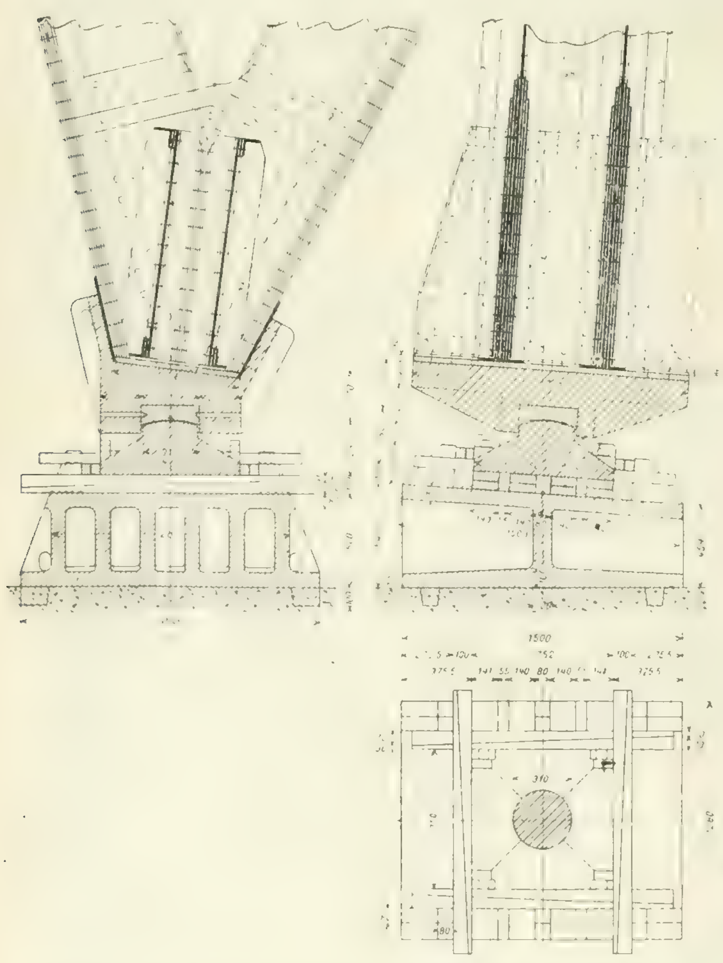


Fig. 2. Plan and Elevation of Hinged End and Shoe of Bietschtal Viaduct.

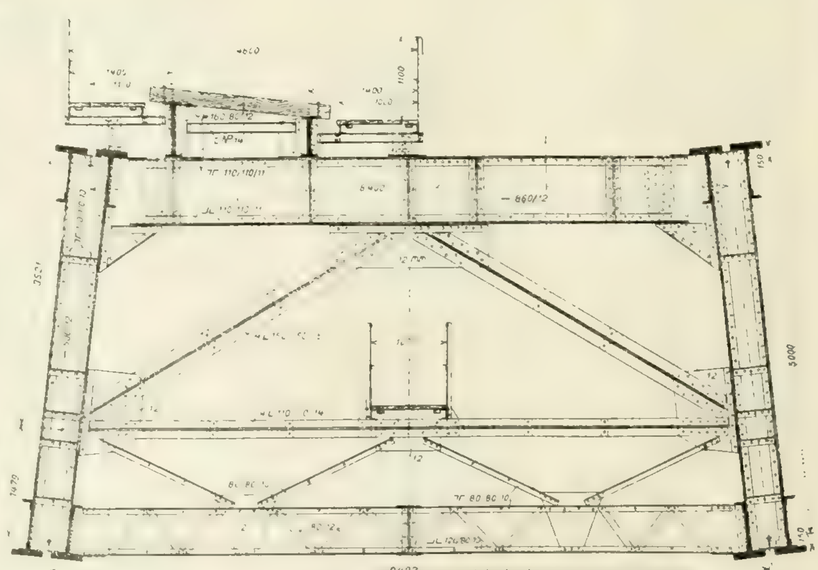


Fig. 3. Cross-Section at Center of Arch Span of Bietschtal Viaduct. One Track Provided at This Time.

CONSTRUCTION FEATURES.

The arch was assembled on rigid scaffolds (see Fig. 1.). Both on account of the great height above the ground and to lessen the settlement (which is considerable with high wooden scaffolds) steel centering, with timbering above it, was used. This arrangement proved to be very satisfactory, as was especially evident in the erection of the final arch members where openings of only 20 mm. (about ¾ in.) width resulted. These openings were closed by the adjustment of the wedges. After the difference in elevation between the arch bearings and the top chord of the centering had been determined by leveling, the height of each splice of the lower chord of the arch above the top chord of the centering was computed. In all probability such satisfactory erection results would not have been obtained by the use of an entirely wooden falsework.

As far as the author knows this is the first time that high derricks have been used for bridge erection, derricks with a reach of 34 meters (about 111.5 ft.) and a capacity of 6 metric tons (about 132,000 lbs.) being set up on each side of the valley. These derricks were operated by steam engines, and they proved to be very efficient. They were used principally to transport the steel from its storage site near the abutments to the platform on the falsework, or to the traveling portal-crane which ran over the horizontal part of the steel falsework. Figure 4 shows a view of the site after the spans were erected but before the floor system was placed. This view indicates the type of construction adopted for the falsework and the traveler, and shows the difficulties which those in charge had to overcome in bridging the canyon. The derricks were also used to place the so-called arch legs, which are on an inclined plane. They also gave excellent service in the erection of the scaffold itself, the latter being built as a tower without any falsework. The outer parts of the falsework and the lower chords were handled as complete units, and, by means of these derricks, were swung out over the canyon and placed in position.

The field riveting was done with compressed air riveters, the air pressure being about 100 lbs. per square inch. Compressed air was also used for the field forges. A cave was blasted in the rock for the engine house and repair shop. The feed water, cooling water, and drinking water were obtained from a spring, which was located in the railroad tunnel, about 650 ft. distant.

As it was impossible to obtain lodging in this wild and barren country, a camp was established with accommodations for about 40 men. There were ten airy and well-lighted rooms for the chief engineer and the workmen, in addition to a big kitchen. The contractors provided furniture and bed linen, but the men, under the direction of the chief engineer, provided the food. The meals cost about 36 cts. per day per man.

No serious accidents occurred during the construction of the bridge, although some of the erection was very difficult.

The foundations for the steel scaffolds were begun in September, 1911, and the work was pushed so rapidly that it was possible to start the erection of the derricks and the scaffolds themselves by the middle of October. The wooden top of the falsework (see Fig. 1) was completed early in May, 1912, and on May 18 the bearing plates of the arch on the Goppenstein side were placed. The inner sides of the lower chords of the arch were connected with the wind bracing on June 16. The erection of the arch had progressed so rapidly that it was possible to start the erection of the outer spans on the Goppenstein side at the beginning of September, the abutment being completed and ready for the steel. On October 1, 1912, the falsework was moved to the Brig side, and this side was completed during December. The finishing work and the riveting of the wind bracing, lateral bracing and railings, including the replacing of rivets condemned by the inspectors, had progressed sufficiently by Feb. 13, 1913, that the connections between the arch and the falsework could be removed.

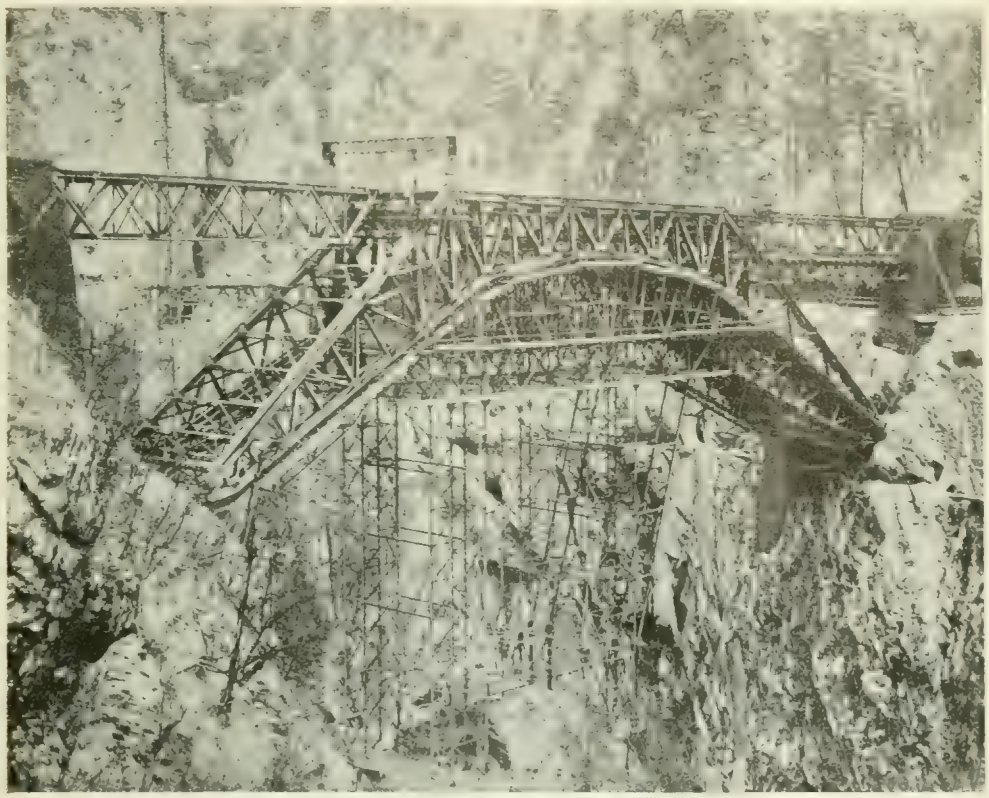


Fig. 4. View of Bietschtal Viaduct and Site—Falsework Still in Place—Track Construction and Railing Not Yet Placed.

QUANTITIES OF MATERIALS.

The following quantities of materials were required:

Double-track steel arch, with stringers for one track only, tons	880
Members at position where the other spans rest on the arch, including the transverse beams, tons	33
Two outer spans, for one track only, tons	187
Steel falsework, tons	363
Wooden top of falsework, with floor and railing, ft. B. M.	92,200
Number of shop rivets	75,000
Number of field rivets	50,000

Cost of Labor and Materials Used in Painting the St. Louis Municipal Bridge.

Contributed by R. D. Spradling, Webster Groves, Mo.

The Municipal Bridge, which spans the Mississippi River at St. Louis, Mo., when completed will consist of three main double-deck river spans and a long approach at each end of the structure. The river spans have been completed for some time, but the approaches are still uncompleted. Each of the three main spans has a length, center to center of end pins, of 668 ft. The piers supporting the center span are spaced 677 ft., center to center, while those supporting the end spans are spaced 672 ft. 6 ins., center to center.

Figure 1 shows a view of the three main spans. It will be noted, by referring to this view, that the panel lengths are unequal, the lengths increasing from the end to the center of the spans. This was done to make the diagonal web members approximately parallel throughout each span.

Figure 2 shows a cross-section of the floor system of the river spans. It will be noted that the bridge is a double-deck structure, the distance from base of rail of the railway deck to lowest point of the highway deck being 22 ft. The trusses are spaced 35 ft. apart, center to center, and the distance from the center of the top chord to the center of the highway deck (at the middle of the span) is about 110 ft. The upper deck has a 30-ft. clear roadway and two 6-ft. clear sidewalks. The distance between track centers for the lower deck is 13 ft. By referring to Fig. 2 it will be noted that the upper deck has a reinforced concrete and I-beam floor; main floorbeams with 48-in. webs being placed at the intermediate posts and intermediate floorbeams with 47-in. webs between the panel points. The construction of the floor system for the lower deck is indicated by the drawing. A detail of the railing used for these spans is also shown in Fig. 2.

PAINT AND PAINTING.

The specifications required that the shop coat of red lead should be retouched where necessary, and that in doing this all rust and dirt should be removed by scraping thoroughly and brushing with wire brushes. After the retouching was finished a coat of graphite paint was applied, and after about three days another coat was applied. At first, no drier (except that used in mixing the paint) was allowed. Later, however, when on two or three occasions sudden rains had washed off the fresh paint, the contractor was permitted to use a small amount of Japan drier. The specifications required that the red lead should be 94 per cent pure and that 30 lbs. of red lead should be used to each gallon of linseed oil. This made a very thick paint, and it proved to be excellent for retouching rusty places. The first coat was of brown graphite and the second was black, which made it easy to determine whether the structure had been thoroughly covered with two coats. The paint was received at the work ready mixed, and required only a small amount of stirring before its application.

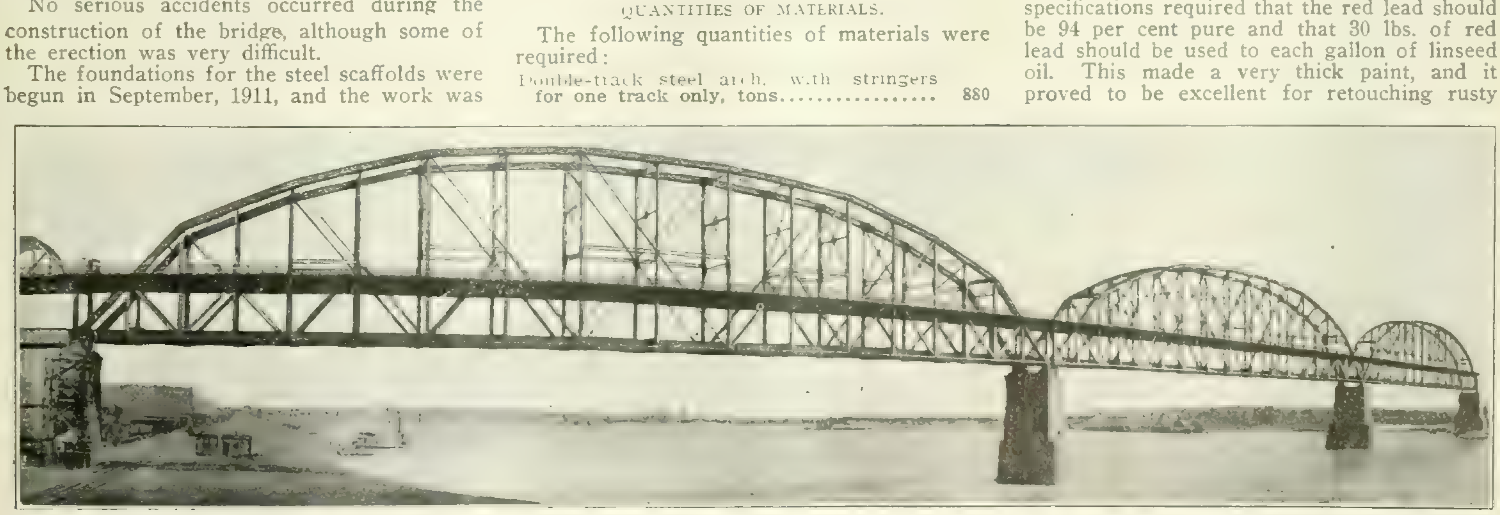


Fig. 1. View of Three 668-ft. River Spans of St. Louis Municipal Bridge Showing Type of Construction.

Very little difficulty was experienced in reaching all parts of the structure. In all cases where scaffolds were swung two men worked on a scaffold, and these men were able to move it without assistance. One man was detailed to keep the men supplied with paint, although at times two men were required for this work. The most difficult part of the work consisted of painting about 12,000 sq. ft. of cast-iron grating on the sidewalk. As the apertures in this grating were about 1-in. square, the use of brushes proved unsatisfactory and small swabs were substituted for the brushes. In general the weather conditions were excellent.

LABOR AND MATERIALS COST DATA.

Table I gives the length of time required, and the labor and material costs for painting each of the three river spans.

COMMENTS ON THE DATA.

By referring to Table I it will be noted that the cost of labor and of paint materials is not far from the same—$5,989.90 for labor and $5,812.60 for paint materials, a total of $11,802.50 for the 13,775 tons of steel in the three spans. This cost does not include any overhead expense, nor does it include the cost of paint brushes and miscellaneous items. The contractor used 450 paint brushes, at $1.60 each, the total expense for this item being $720. The estimated cost of miscellaneous items was $100.

The great variation in the quantity of materials used and in the cost of painting span No. 1 and spans Nos. 2 and 3 was due to the fact that span No. 1 was erected first and had been subjected to wear incident to the erection of the other two spans. On spans Nos. 2 and 3 the painters were all Greeks, and they proved to be very efficient workmen. They were cautioned to brush the paint out thoroughly, both to obtain a good surface and to insure economy of paint.

Discipline on an English Railway.—A bulletin showing the list of fines imposed on various mechanical employes on the London & Northwestern, dated Aug. 25, 1892, and signed by F. W. Webb, superintendent, shows how careful this railway was in maintaining its discipline. Enginemen, firemen and cleaners were fined nominal sums for various causes from passing signals set at danger to smoking in the shop. The maximum fine was about $5 for passing a signal at danger and causing a mishap. The smallest fines were two shillings and sixpence, or about 60 cents; these latter fines were levied for causing smoke nuisance, smoking a pipe in a shed, causing damage to lubricator, being late out of the roundhouse, causing delays to trains, overrunning passenger station platforms and for derailments due to carelessness. Other employes were dismissed for disregarding signals, for damaging the paint on an engine with a penknife, etc.

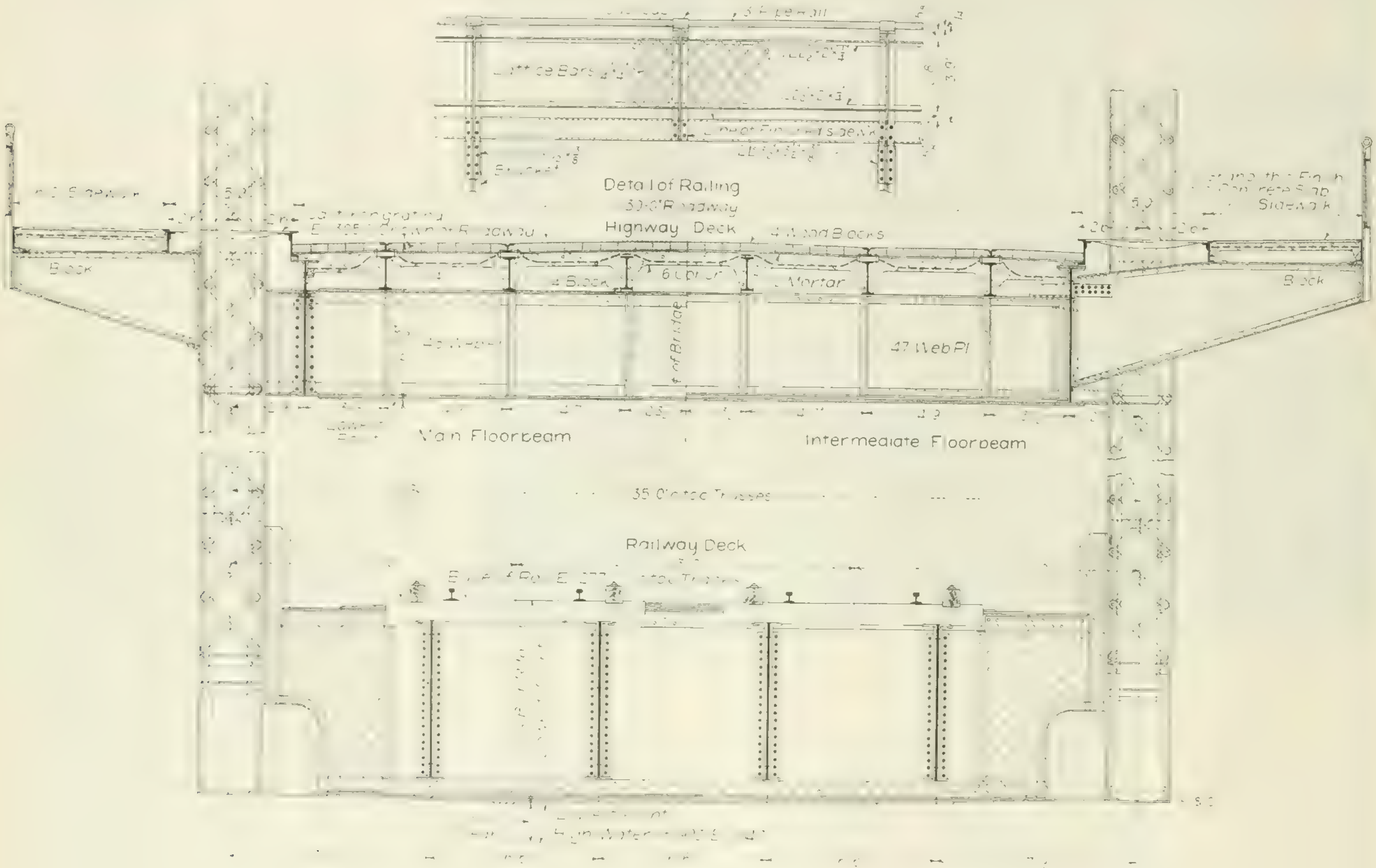


Fig. 2. Cross-Section of Floor System of St. Louis Municipal Bridge and Detail of Railing.

TABLE I. TIME REQUIRED AND COSTS OF LABOR AND MATERIALS FOR PAINTING ST. LOUIS MUNICIPAL BRIDGE.

Item.	Date started.	Date finished.	Time: Elapsed time, days.	Labor: Elapsed time, hours.	Hours per ton.	Total cost at 35 cts. per hour.	Cost, cts. per ton.	Materials: Quantity used.	Unit cost.	Total cost.	Cost, cts. per ton.
				Span No. 1.							
								7,000 lbs. red lead	$0.055	$385.00	
Retouching	Apr. [illegible], 1914	May [illegible], 1914	25	1,350	0.294	$472.50	10.3	282 gals. linseed oil	0.75	211.50	13.0
First coat	May 20, 1914	June 14, 1914	23	[illegible]	0.770	1,242.85	26.9	747.5 gals. graphite paint	1.30	971.75	21.2
Second coat	June 13, 1914	July 12, 1914	23	2,281	0.498	798.35	17.5	557.5 gals. graphite paint	1.30	724.53	15.8
				Span No. 2.							
								4,520 lbs. red lead	$0.055	[illegible]	
Retouching	June [illegible], 1914	June 27, 1914	18	893	0.195	$312.55	6.7	188 gals. linseed oil	0.75	141.00	8.5
								557 gals. graphite paint	1.30	724.74	
First coat	June 15, 1914	July 16, 1914	24	2,103	0.457	736.05	15.9	25 gals. linseed oil	0.75	18.75	16.2
Second coat	July 10, 1914	Aug. 1, 1914	21	2,296	0.500	803.60	17.5	527 gals. graphite paint	1.30	685.10	15.0
				Span No. 3.							
								[illegible] lbs. red lead	[illegible]	$207.90	
Retouching	June 28, 1914	July 31, 1914	28	761	0.166	$266.35	5.8	159 gals. linseed oil	0.75	119.25	7.1
								[illegible]29 gals. graphite paint	1.30	687.70	
First coat	July 17, 1914	Aug. 7, 1914	18	1,855	0.405	649.25	14.0	25 gals. linseed oil	0.75	18.75	15.4
Second coat	Aug. 2, 1914	Aug. 21, 1914	16	2,024	0.442	708.40	14.6	514.33 gals. graphite paint	1.30	668.63	14.5
Totals				17,114		$5,989.90				$5,812.60	

SEWERAGE

Data and Discussion on the Design and Maintenance of Inverted Siphons in Sewers.

Contributed by Frank H. Carter, Assoc. Mem. Am. Soc. C. E., Designing Engineer, Cambridge, Mass.

The recent proposed location of an interurban railroad under some of the main streets of an eastern city apparently cut the sewerage system of the city in two with no remedy except the construction of numerous inverted siphons. In advancing all possible reasons against the location of the railroad under the streets of the city the claim was made by the objectors that inverted siphons were impracticable. The writer had as one of the engineers for the petitioners previously designed several inverted siphons among which were four built under similar conditions by the city of Cambridge, Mass., for the Boston Elevated Railway Co., due to the construction of the Cambridge Main Street Subway in Cambridge, Mass.

One unavoidable difficulty in connection with the proposal to build these inverted siphons, as was the case with the ones constructed by the Boston Elevated Railway for the Cambridge Main Street Subway, was the fact that the inverted siphons must be built in old sewers with no head available to overcome the extra loss of head arising from the construction of the inverted siphons.

In seeking for data concerning inverted siphons built by the city of Boston, it was discovered that most of these inverted siphons were built into old sewers with no head available other than that produced by the backing up of the sewage to produce the proper head necessary to overcome the loss of head due to the introduction of the inverted siphon.

In designing the inverted siphons for the Boston Elevated Railway Co. for the Cambridge Subway, the writer adopted the "two pipe method" for combined sewers with the smaller pipe at the lower elevation. When the smaller pipe is filled the sewage rises in the siphon chamber to fill the other pipe which then comes into play. For storm sewers the one pipe siphon was used.

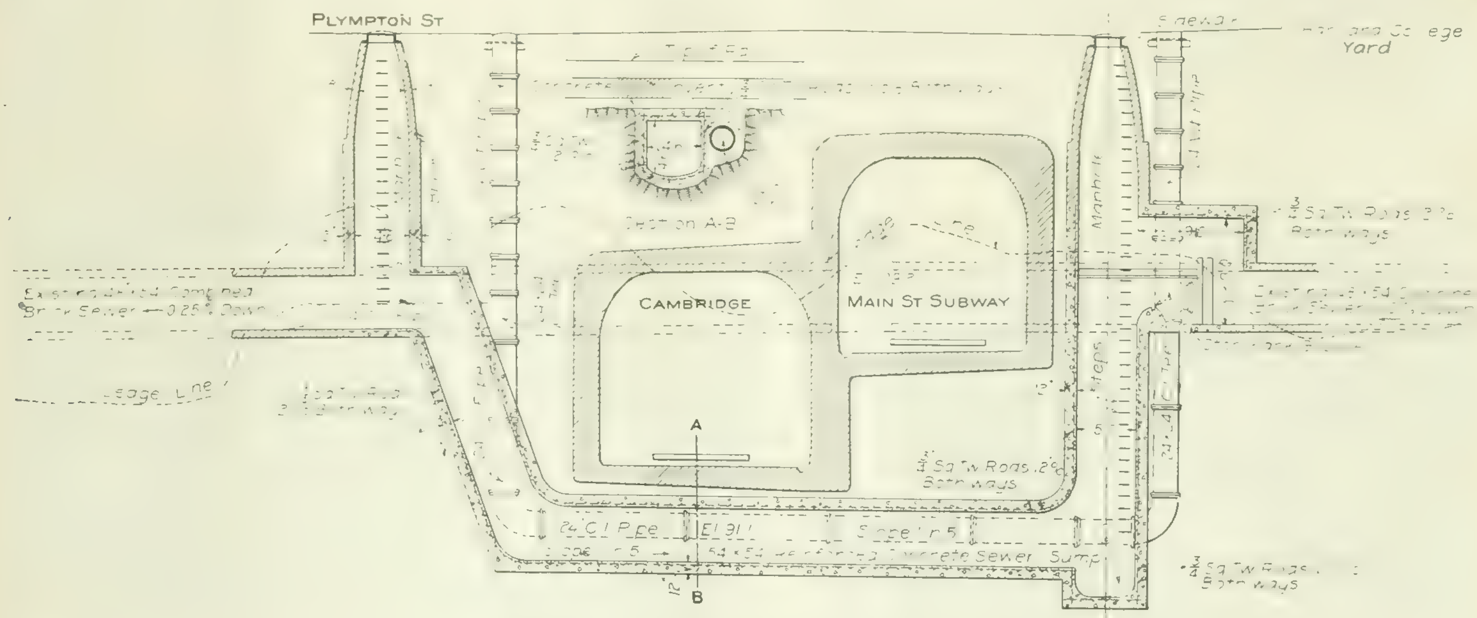
As to the proper ratio of area of the inverted siphons with respect to the area of the sewer itself, due consideration must be given to the velocity in the siphon to provide proper cleansing velocities. Where siphons are introduced into a new sewerage system, it is in general not difficult to provide the proper velocities especially where the proper loss of head to overcome the increased flushing velocities is obtainable. Where, however, inverted siphons are introduced into old sewerage systems from one cause or another, due consideration must be given to the fact that every increase in velocity through the siphon pipe or pipes is attended with the formation of a pool above the siphon due to the loss of head rising from the passage of the sewage through the siphon at the higher velocity. It is, however, no doubt easier to cleanse a deposit from the sewer at this point than to remove a deposit from the siphon itself.

From the standpoint of removal of sludge deposit, the siphon pipe should be as large as possible as contrasted with the demand for a high velocity that the pipe be as small as consistent with the allowable loss of head necessary to pass the sewage through the siphon pipe.

In general, too, it is well to build the upstream leg of the inverted siphon as a vertical manhole perhaps rather larger than the pipe section of the siphon to reduce the velocity in this part of the siphon and thereby to cause whatever deposit will occur, if any, to take place in this upper leg manhole. Then the part of the inverted siphon which is nearly horizontal should preferably have a decided pitch towards the upper leg manhole for drainage towards this manhole.

Another desideratum is that the rising or down stream leg of the inverted siphon be built on an incline that the sewage in solution which is not deposited in the larger section of the upstream deposit manhole may be more readily carried along with the flow of sewage through the siphon.

Time may be profitably spent in study of the hydraulic features of the location of the two pipes of a double siphon above the invert of the sewer in the upstream manhole, such as entry head, etc.



Longitudinal and Transverse Sections of the Inverted Sewer Siphon Under the Cambridge Main St. Subway, at Massachusetts Ave. and Plympton St., Cambridge, Mass.

The 4 ft. 6 in. by 4 ft. 6 in. combined sewer will serve eventually as a storm water sewer. The 24 in. sewer will eventually serve as a sanitary sewer and was designed for present dry weather flow.

TABLE I.—INVERTED SIPHONS ON THE METROPOLITAN SEWERS IN THE VICINITY OF BOSTON, MASS.

Name of siphon.	Length, ft.	Diameter.	Depth, low point, ft.	Outlet lower than inlet, ft.	Quantity, cu. ft. per second.	Velocity, ft. per second.
Shirley Gut	[illegible]	6 ft. 1½ ins.	20.06	0.73	56.6-79.4	(1.92-2.70)
Chelsea Creek, East Boston	515±	5 ft. 7½ ins.	27.32	5.17	57.5-80.8	(2.31-3.25)
Malden River	[illegible]	3 ft. 5 ins.	7.94	0.70+	10.9-32.0	(1.19-3.49)
Mystic River, Boston	90	24 ins. Akron pipe	12.22	1.60	4.85-9.40	(1.54-2.99)
Mystic River, Charlestown	...	5 ft.	[illegible]	..	.....	....
Abbajona River, Medford	[illegible]	20 ins. Akron pipe	5.93	0.45	3.94-6.14	(1.80-2.82)

Note.—Careful observations of loss of head at all the siphons are made at frequent intervals; these show that after 5 years' service no appreciable deposits exist within the siphon.

INVERTED SIPHONS ON THE METROPOLITAN SEWERS IN THE VICINITY OF BOSTON, MASS.

Data concerning the length, diameter, depth at low point, loss of head, available quantity and velocity for some of the metropolitan sewers near Boston are given in Table I. It will be noted that the greatest range as well as the maximum velocity occur in the Malden River siphon, the third on the list, where the range in velocity is from 1.19 to 3.49 ft. per second. Careful observations made at frequent intervals of the loss of head on all these siphons show no appreciable deposits within any of the siphons after five years' service.

DATA ON INVERTED SEWERS IN BOSTON.

In Table II data are given relative to the inverted sewer siphons in Boston. The engineers of the engineering department of the city of Boston state that but little trouble has been experienced and that the mainte-

TABLE II.—DATA ON INVERTED SEWER SIPHONS IN BOSTON.

No.	Location and description.	Length.	Diameter.	Depth, low point.	Outlet lower than inlet.
1.	Tennean or Smelt Brook crossing of intercepting sewer, Dorchester, Mass.[1]	52 ft. 3½ ins.	4 ft. 6 ins.	7 ft. 8 ins. ±	
2.	Storm overflow siphon under Met. Sewer Conn., Saratoga and Bennington Sts.[2]	55 ft.	4 ft. 6 ins.	8.30 ft.	0.05 ft.
3.	Parsons Brook Brighton, N. Beacon St. double Siphon[3]	11 ft.	12 ins. } twin 36 ins. } twin	2.55 ft.	Practically none.
4.	Davenport Brook, Adams St. and Minot St., Dorchester, double siphon.	20 ft. 6 ins.	20 ins. } twin 36 ins. } twin	3:57 ft.	0.04 ft.
5.	Hanover St, under subway[4]	......	6 ft. 6 ins. x 3 ft. 10 ins.	1.03 ft.	
6.	Oakland Brook over Dorchester high level intercepting sewer on River St.	38 ft.	4 ft. 9 ins.	1.50 ft.	Practically none.
7.	Muddy River, town of Brookline[5]	28 ft. 4 ins.	24 ins. 36 ins.	13 ft. ±	6 ins. ±
8.	Huntington Ave. and Parker St., Stony Brook improvement[6]	64 ft. 52 ft.	48 ins. Two 48 ins. C. I. pipes	8.6 ft. No drops.	0.02 ft. 0.02 ft.

[1]Has sump manhole at low point, also means of flushing with storm water. 45° incline both ends.

[2]Had considerable trouble. Tidal culvert overflow.

[3]Overflow into brook.

[4]Not used now; has been stop-planked off. Sewer is 5 ft. 0¼ ins. x 4 ft. 9 ins. "egg."

[5]11 ft. x 7 ft. divided sump manhole. Cost more for maintenance than any other in city of Boston. Much trouble with manhole from lack of proper attention, coupled with the fact that the vertical manholes are perhaps unnecessarily large.

[6]Common sewer overflow provided with means for flushing from Stony Brook Brighton intercepter.

TABLE III.—INVERTED SEWER SIPHON UNDER CAMBRIDGE SUBWAY.

		Length, ft.	Diam.	Depth, low point.	Outlet lower than inlet.
a.	Portland St. Metropolitan sewer	55.0±	36-in. C. I. pipe 20-in. C. I. pipe	12.	0.00
b.	Douglass St., City of Cambridge	....	....	....	....
c.	Plympton St., City of Cambridge	59.0±	4 ft. 6 ins. x 4 ft. 6 ins. sq. Also 24-in. pipe for dry weather flow.	20.1	0.06
d.	Portland St. (City of Cambridge) sewer.	44.0±	42 ins.	22.0	0.10

Note.—The "a," "b" and "c" siphons were built in old sewers. The cost of maintenance of "a" was $700 to $1,000 the first year, due mainly to a settlement of 2 ft. in the old sewer at the crossing and also perhaps partly to the design of the siphon itself. The conditions there were very difficult and any design must have uncertainties until tried out.

The "b," "c" and "d" siphons cost less than $100 yearly for maintenance for all three. No records of cost kept as yet. Siphons have been in operation about a year, but little or no obstructions have been found, merely a few sticks of wood in one or two instances.

nance costs have but little exceeded those for the same length of sewer except in the case of the Hanover Street siphon where grease from the hotels caused considerable trouble, and of the Muddy River siphon from wrong design and lack of proper attention. The terminal manholes were too large in the latter siphon. All of the siphons mentioned except Nos. 2 and 7 were installed in old sewers.

INVERTED SEWER SIPHONS UNDER CAMBRIDGE SUBWAY.

Table III gives data on the inverted sewer siphons constructed under the Cambridge Subway by the Boston Elevated Railway Co. in 1910. In designing the inverted siphons for the Boston Elevated Railway Co., the approval of Mr. Lewis M. Hastings, the city engineer, was required under the act of legislature authorizing the construction of the subway. The writer is largely indebted to Mr. Hastings, under whom he previously served for seven years as assistant engineer, and from whose experience he obtained much of his material for the design of these and other inverted sewer siphons as well as to the late George A. Kimball, Mem. Am. Soc. C. E., Chief Engineer Boston Elevated Railway, under whose guidance and direction all the design and construction of the work was executed. Mr. Hastings is authority for the maintenance data given at the bottom of Table III. The principal sections of the Plympton St. inverted siphon are shown in the accompanying figure.

No trouble has arisen from the construction of three of these siphons, but the one which had to pass the approval of the Metropolitan Sewerage Commission of Massachusetts, having been designed to operate under the peculiar conditions existing there is said to have cost from $700 to $1,000 the first year for maintenance, perhaps largely due to the settlement in the main sewer in which the inverted siphon was introduced and which amounted to 2 ft. at this point. This meant a damming of the sewage with the consequent formation of a pool above the siphon and the probable deposit of sludge therein.

Notes on American and European Practice in the Design of Intercepting Sewers.

Interesting comment on American and European practice in the design of intercepting sewers is made in the final report of the Metropolitan Sewerage Commission of New York City. In discussing the proposed sewerage improvements for the Lower East River, Hudson and Bay Division the Commission considered that the interceptors should be designed according to European rather than American practice. According to the American method, the interceptor is usually constructed entirely below the invert of the lateral sewer, with openings connecting the two, leaving the outer end of the lateral open for a storm overflow. The flow into the interceptor is controlled by a regulator, which is operated by a float. When the flow in the lateral reaches a certain amount, say twice the mean dry-weather flow, the regulator closes, cutting off the entrance to the interceptor and permitting the storm flow to discharge into the river by way of the original outlet.

According to European practice, the interceptor is built across the lateral sewer, its crown above the invert of the lateral. The interceptor therefore acts as a dam, preventing both the flow of sewage to the river, and the back flow of the tide from the river. All the sewage is diverted until the water surface in the lateral rises above and overflows the crown of the interceptor. The excess storm water then flows directly into the river. Where the crown of the interceptor is too high to permit such discharge, a special section may be adopted where it crosses the lateral.

The chief advantages of this arrangement are: The elimination of mechanical regulators which are not always reliable; a raising of the hydraulic gradient in the interceptor and consequent reduction of lift at the pumping station; and less depth and consequently less excavation for the interceptors. The system has the disadvantage of requiring larger cross-sections for the sewers. The European system involves placing, at intervals, overflow dams with storm water conduits to the river so that when the flow is collected from a number of sewers, to which has been added storm water from three to six times the dry-weather flow, the diluted sewage overflows the dams and reaches the harbor. The only regulators which are required under this system are the dams themselves, automatic tide-gates being unnecessary, except as in the case of Hamburg, where, by reason of large tidal range, the sewers are called upon to act as a reservoir for a part of the time.

A system which is a modification of the principles in use abroad, which would admit of considerable saving over the usually applied American system, has been developed in the case of the collectors at Philadelphia. This system is here described from information contained in the report of George E. Datesman, principal assistant engineer of the Philadelphia Bureau of Surveys, to the Metropolitan Commission. (Mr. Datesman was employed by the Commission to make an expert report upon the proposed drainage scheme for the Lower East River, Hudson and Bay Division.)

It has been found economical to build both high and low-level sewers, carrying every cubic foot of sewage practicable by gravity to the treatment works. In the low-level system, by substituting a dam to exclude tide water in the combined sewer at such a point removed from the outlet that the crest will not rise beyond two-thirds of its vertical diameter, large tide-gates at the outlets may be eliminated. Thus to the advantage due to grade in the sewer, saved in the depth of the main collector, is added the advantage of carrying the interceptor through the dam, resulting, in some cases, in raising the whole length of the interceptor from 6 to 10 ft. over its position in the usual American plan. This, of course, involves the construction of a system of domestic sewage sewers in streets tributary to the main combined sewer below the dam and the making of new house connections.

The effect is to leave open to tidal fluctuation that portion of the combined sewer between the dam and outlet. If the sewage is carried away from a river, even the tide-locking of this portion of the sewer should not be a detriment to the intercepting sewers, which are not affected thereby in a pumping system.

The benefit in the item of cost of a long interceptor, by being able to raise it from 6 to 10 ft., is apparent; also the saving due to decreased lift at pumping stations.

Where grades of combined sewers admit, additional savings may be made in depth of interceptors by placing them at some distance from the marginal avenue. This should, however, always permit of a house sewage sewer, when running against grade, reaching the interceptor.

Another accessory of this plan is the introduction of overflow chambers of enlarged section to admit of carrying off the storm flow into the conduit below, wherever the dams are introduced.

In Philadelphia it has been planned to introduce such dams, even in low-lying areas where tidal influence extends between 4,000 and 8,000 ft. from the outlets. The dams will be placed at about 2,500 ft. inland from the river, involving the building of a separate system of sewers in the territory below the dams and carrying of the drainage back into the main collector. Light grades may be used on the small sewers, which may be flushed at high tide from the river either automatically or by hand-operated gates.

An Efficient Portable Mechanical Device for Cleaning Catch Basins, Successfully Employed at Pawtucket, R. I.

Contributed by Joseph Wood, Assistant Commissioner, Department of Public Works, Pawtucket, R. I.

Owing to the steadily increasing cost in cleaning our 1,300 catch-basins due to increasing wages and scarcity of dumping places which necessitated long hauls, the Board of Public Works of Pawtucket, R. I., a city of 55,000 people, decided that this work could be done more efficiently and economically by some kind of motor apparatus. The writer evolved the idea embodied in the machine here illustrated and described. The machine was made and assembled in our own shop by Mr. James Nisbet, mechanician in the public works department.

The device consists of a 3-ton "Standard" chassis upon which was mounted a dumping body of the following dimensions: Length inside, 9 ft.; width in front, 4 ft.; width in rear, 4 ft. 9 ins., and height, 28 ins.

This body is dumped by a hydraulic lift operated by an oil pump driven by a chain from a jack shaft engaged with motor by sliding gear. In the rear of the driver's seat, on I-beams extended across the chassis frame, is mounted a 2-HP. "Fairbanks-Morse" gasoline engine which draws its gasoline from the same tank that supplies the motor; its water circulation is through the radiator of the motor, thus preventing freezing in cold weather when motor is stopped for loading the truck, and the exhaust of this small engine goes into the muffler, thus reducing noise to a minimum.

This 2-HP. engine furnishes the power for raising and lowering the orange-peel bucket, drives the oil pump for opening and closing this bucket and for operating the 70-gal. centrifugal pump which empties the basins of surplus water. The orange-peel bucket has a capacity of 1½ cu. ft. and makes two trips per minute from the basin to the truck.

This apparatus was first put into service on July 17, 1913, with a plain bucket which was filled by a shoveler in the basin. This reduced the cost of cleaning from $2 per cubic yard by the old method to $1.05. On November 17 the orange-peel bucket was installed with the result that the cost dropped to 86 cts. per cubic yard.

This device takes the place of six men, six one-horse carts with drivers and an inspector whose work is now done by the driver of the truck.

The centrifugal pump in five minutes empties a basin of water that under the old method of bailing by hand would take from 30 to 40 minutes. The discharge end of the hose is thrust into the trap and the water goes into the main sewer. The suction and discharge hoses are shown in Fig. 1, coiled up for transportation, at the side of and beneath the driver's seat.

The same view shows the orange-peel bucket rising from a basin containing sand and leaves. The operator, standing on the drop platform, raises and lowers the bucket by moving the lever upon which his right hand rests. Near his right foot is seen the plunger of a 4-way valve which opens the bucket when pushed down by his foot. The foot being removed from the plunger closes the bucket by shifting the oil pressure from the bottom to the top of the piston of the bucket. In this illustration the writer and Mr. Nisbet are seen at the front of the machine. In Fig. 2 the truck is shown in dumping position with swing door open and bucket and crane swung aside to allow body to rise.

This outfit cost the city $4,200 and was, excepting chassis and body, all built and assembled in our own shop. Up to the present time this device has not lost two hours of working time and the city takes a great deal of pride in this municipal invention.

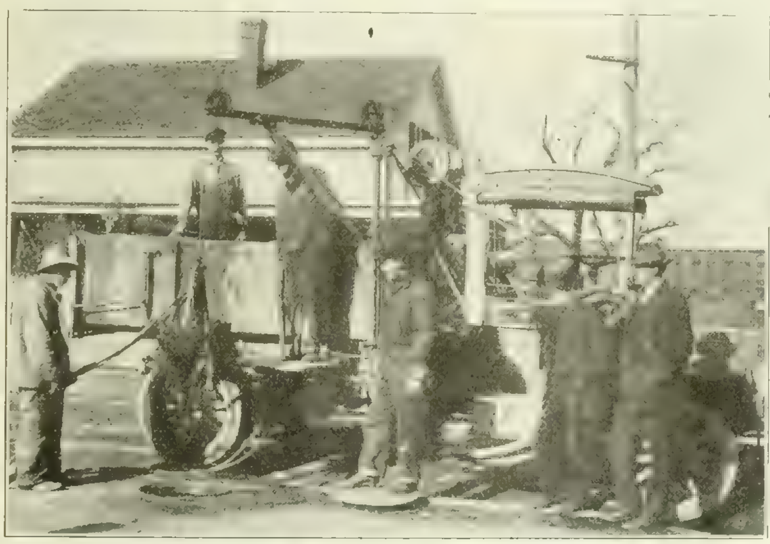


Fig. 1. View of Portable Machanical Device for Cleaning Sewer Catch Basins at Pawtucket, R. I.

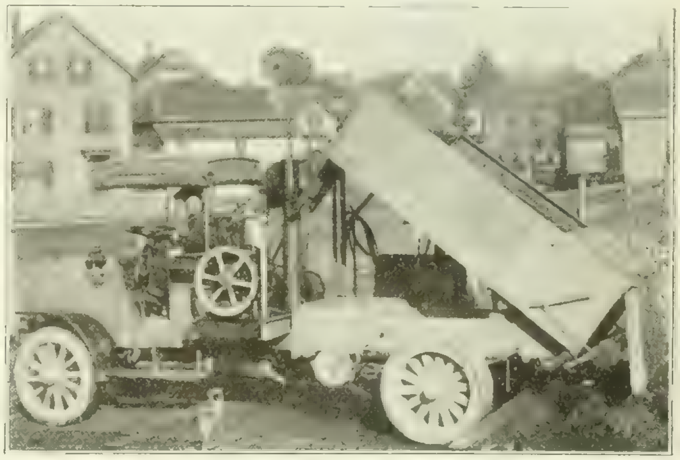


Fig. 2. View of Pawtucket Catch Basin Cleaning Machine in Dumping Position.

Credit for Engineering on West Grove, N. J., Sea-Outfall Sewer.

An article was published in this journal of April 1, 1914, entitled: Method Employed in Laying a 1,000 ft. Sea-Outlet for a 12-in. Sewer at West Grove, N. J. Attention has been called to an error in giving credit for the engineering work on this job. The erroneous statements previously made are here repeated and the facts in the case are recorded as follows:

It was stated: "Mr. William M. Aitchison, who was the supervising engineer in charge"; also "Matthews Brothers of Red Bank, New Jersey, were the contractors on this work. Clyde Potts, Civil and Sanitary Engineer, 30 Church St., New York City, looked after the interests of West Grove, the writer (Mr. Aitchison) being his representative."

The submerged sewer outfall, described in the article, was run out into the ocean to discharge the effluent from the clarification tanks of Sewer District No. 1, Neptune Township. The township comprises what is commonly known as West Grove and Ocean Grove. The sewer district is in West Grove and the outfall sewer extends through property of the Ocean Grove Camp Meeting Association. The commissioners of the sewer district and the Camp Meeting Association failed to reach an amicable arrangement as to the right of way through certain private property of the Camp Meeting Association. Mr. Clyde Potts was the engineer of the Camp Meeting Association and testified for it in injunction suits which were brought by the association in an endeavor to stop the construction of the submerged ocean outfall by the West Grove Commissioners. Subsequently the Camp Meeting Association receded from its position and made an agreement by which the right of way through its property was granted.

The entire design and supervision of the construction of the sewage clarification works, outfall sewer and ocean outlet were in charge of Pugh & Hubbard, of Philadelphia, engineers for the commission. The Monmouth Contracting Co., of Red Bank, N. J., was the contractor for the disposal works and outfall sewer. The sub-contractors for the ocean outlet were Matthews Brothers, also of Red Bank.

Inspection of Sewerage Plumbing at New Orleans.

The procedure of the plumbing department of the Sewerage and Water Board of New Orleans is here described from information taken from the latest semi-annual report of Mr. G. A. Middlemist, civil engineer in charge of the inspection of plumbing.

Before any plumbing work at all is done it is necessary that a licensed master plumber file a plan showing the character of plumbing that is to be done in the premises in question (signed by the property owner, authorizing the work to be done), the plan showing the size of pipe, kind of traps to be used, and the various methods of venting. Upon examination, should this plan be found to be in conformity with the rules, it is approved by the supervisor of plumbing, or his assistant, and permits are granted to the plumber, one to do the work, and the other to permit the premises to be connected to the sewer after the work has been inspected and satisfactorily completed in accordance with the rules.

Four inspections are generally required for every job before it is declared satisfactory in its entirety. These tests consist of a ball test, a house connection or tie-in test, a water test and a peppermint test.

The ball test is an inspection and test of the underground terra cotta pipe which runs from the property-line back, receiving the discharge from the plumbing fixtures in the building and conveying same to the public sewer in the street.

The inspector sees that the pipe is laid at a grade of not less than ¼ in. to 1 ft., and that the joints are properly made, caulked and cemented, and that the proper covering is maintained in accordance with the plumbing rules. The line is then tested by sending a wooden ball, ½ in. less in diameter than the diameter of the pipe, through it, from end to end.

The house connection inspection is made where the public and house sewer are joined together at or near the property-line. This inspection consists of seeing that the proper fittings are used, clean-outs brought up, so as to enable the proper cleaning out of the line in the event of stoppage, and the proper concreting of the whole.

The water test is made after the plumbing work inside of the building has been "roughed-in." The stacks are filled with water and allowed to remain in that condition for some hours. The inspector then examines the joints closely, and if they are improperly caulked, leaks will appear at these points. At the time of this inspection it is the duty of the inspector to examine the kind of pipe used; i. e., whether standard or extra heavy (for in some instances only the latter can be used), the depth of hubs, the size of pipe, the proper use of vents, and, in fact, to see that in every particular the work conforms to the rules of the board and is sanitary in every respect. This is the severest test put on the work, for, having passed this test, the only thing necessary to be done is the setting of the fixtures.

The fixtures having been set, the work is then subjected to what is known as the peppermint test. This test consists of pouring into the clean-out a mixture of hot water and oil of peppermint. The fumes that arise from this mixture are so pungent that if the work is not absolutely tight the odor of peppermint will be immediately detected in the premises. The inspector then tries out all the fixtures for the purpose of ascertaining whether or not they are in good working order. At the time of this inspection all vaults and cesspools must be excavated, cleaned and filled, and the work as a whole must be in a completed condition.

It is only after these tests and inspections have been made that what is known as a final certificate is issued. This certificate is to the effect that the plumbing has been inspected and tested, and found in substantial accord with the plumbing rules and regulations.

CONSTRUCTION PLANT

MACHINES DEVICES MATERIALS

The Meridiograph.

(Contributed.)

The meridiograph is a new instrument for determining true north accurately and quickly. With its aid, anyone who can read an angle may determine in a few minutes a true meridian, at nearly any hour of the day, without computation, books, tables or attachments. It is only necessary to measure the sun's altitude, as shown in Fig. 1; its declination is taken from the ephemeris, and latitude from a map. Two settings of these data on the meridiograph and a simple addition give the true bearing of the sun. One additional setting gives, if desired, accurate astronomic time.

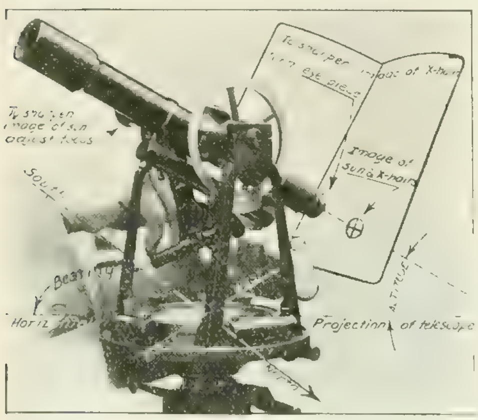


Fig. 1.

The meridiograph, as shown in Fig. 2, consists of two circular rotating discs, 7 ins. in maximum diameter, and a reading arm. On these discs scales of latitude, altitude, declination and bearing are graduated to 5′ or 10′ and on these scales the data are set, to an accuracy of about 1′, thus:

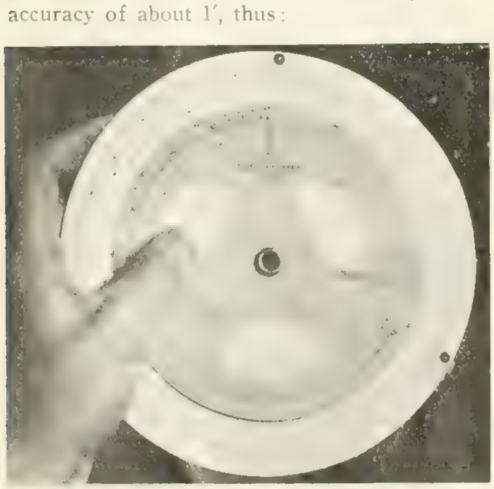


Fig. 2.

Set altitude against latitude on scales a; opposite index read number A; set altitude against latitude on scales b; opposite given declination read number B; opposite number (A + B) read the true bearing of the sun.

Figure 3 shows the connection between the sun's true bearing and the true north line to be located on the ground.

The theory is: The angle scales are plotted on the number scale as a base, all of them being logarithmic. They are arranged to solve:

$A = \tan \text{altitude} \times \tan \text{latitude}$.

$B = \sec \text{altitude} \times \sec \text{latitude} \times \sin \text{declination}$.

$A + B = \cos \text{bearing}$.

For determining astronomic time the scales solve directly, with one setting, $\sin \text{time} = \sin \text{bearing} + \cos. \text{altitude} \times \sec. \text{declination}$.

The connection of these equations with the elemental laws of spherical trigonometry and with the ordinary principles of the slide-rule will be obvious to the critical reader.

Its accuracy is within 1′ in the early forenoon or late afternoon, and within 2′ closer to noon. The field procedure takes 2 or 3 minutes, while the reduction with the meridiograph is made right at the transit (or checked in the office) in about a minute.

The meridiograph is made of bronzed metal, with finely etched graduations; a secondary model is made of ivory-celluloid—water and sun proof. Clear and explicit directions with auxiliary diagrams for obviating all computation and errors—with an abbreviated ephemeris—accompany the instrument and make it, in connection with a transit, a complete north-finder, independent of all books, tables, or attachments, and suitable for all latitudes. The meridiograph has been invented and is manufactured by Louis Ross, Civil Engineer, 268 Market St., San Francisco.

A Portable Sand and Gravel Washing Plant For Concrete Construction and Road Work.

(Contributed.)

A portable washing outfit, including disintegrating screen complete with tank, bucket elevator, hopper bin with gate, and countershaft with pulley ready for attaching power, is illustrated here. The washing element consists primarily of a cylindrical grizzly of 1 in. round bars spaced 1 in. apart, and outside of this a close fitting perforated metal cylindrical screen. Heavy cast heads and interior spiders hold the grizzly rods and the screen in position and support them on the longitudinal revolving shaft. This cylindrical body sets in a tank kept filled with running water and is revolved in the tank; the fine material passes through the perforated screen into the tank and is lifted out by the elevator cups shown attached to the screen; the coarse material retained by

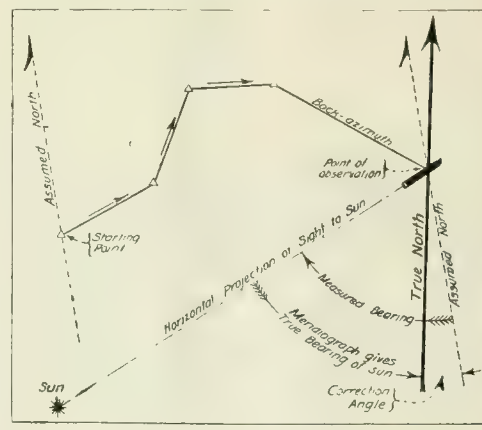
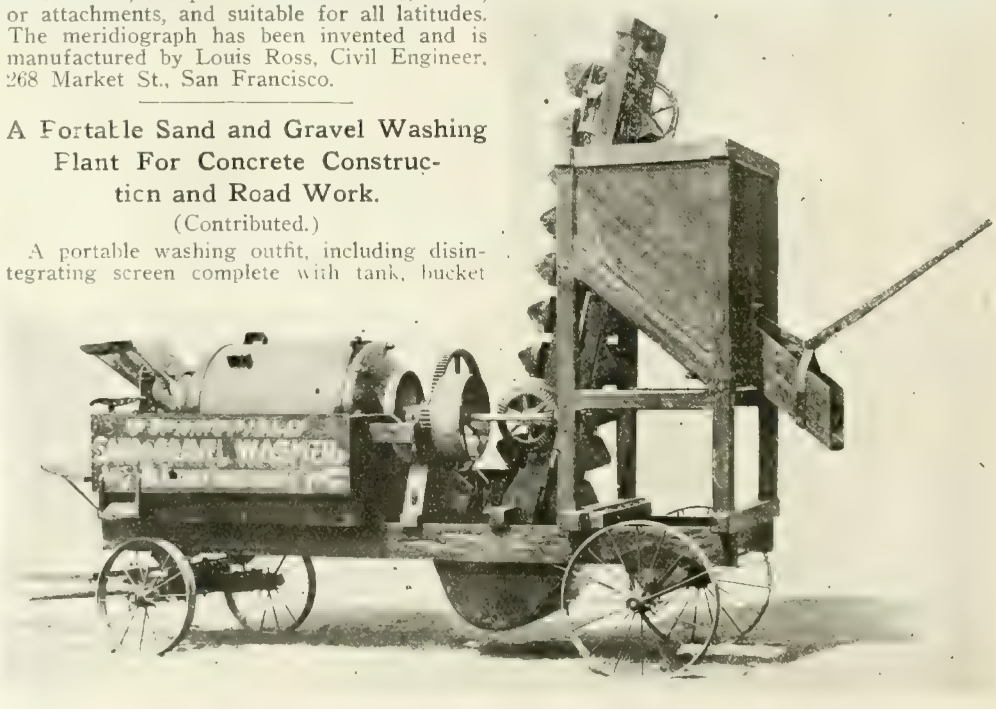


Fig. 3.



Portable Sand and Gravel Washer.

the screen is picked up by a plow shaped casting secured to the discharge head and is deposited into a chute and thence into the boot of the bucket elevator. The other operations are evident from the illustration.

The disintegrating screen used in the outfit shown can be fitted with any size perforation to suit the requirements of the contractor. The wear and load of the material inside of the screen does not come directly upon this perforated metal, for the reason that the screen is lined on the inside with 1 in. round steel bars placed 1 in. apart as previously described.

The main elevator has 12 in. wide cups mounted on No. 114 detachable chain. This elevator is provided with adjustable take-up bearings at the top. The speed of the screen is 20 r.p.m.; the speed of the elevator is 80 ft. per minute; the speed of the driving pulley on the machine is 120 r.p.m. The disintegrating screen is 42 ins. in diameter and 4 ft. long, that is, the length of the perforated plate.

The machine as shown is intended to be driven by gasoline engine or motor located at right angles to the truck. The makers are also contemplating building these outfits with a single or multiple cylinder gas or gasoline engine mounted upon the truck and located directly under the washed sand and gravel bin, in which event the drive on the disintegrating screen will be through bevel gears, and the drive from the engine to the screen countershaft will be through a silent or roller chain. The gate supplied on the bin is of the quadrant type and easy to manipulate The water discharged from the screen carrying the clay in suspension can be settled in a pond or basin close to the operation and pumped back into the machine, thereby using the water over again. Fresh water is fed into the disintegrating screen at the discharging end.

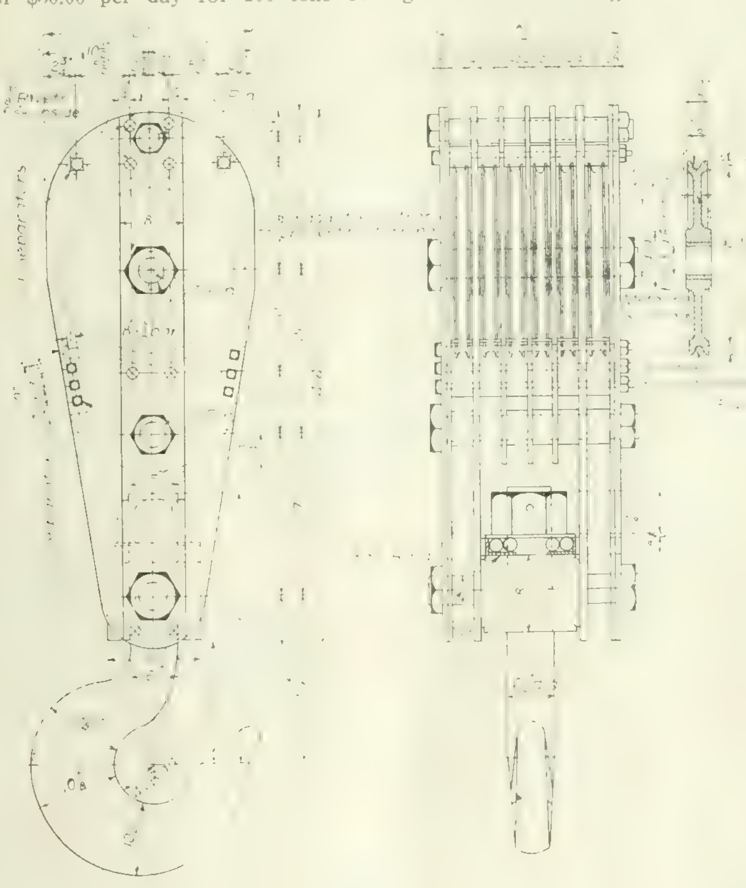
This sand and gravel washer will handle 100 tons in 10 hours; assuming a cost of 25 cts. per ton for handling the sand and gravel from the bank and delivering it into the machine, and an additional 5 cts. per ton for power, oil, etc., would make a total cost of 30 cts. per ton, or $30.00 per day for 100 tons of washed sand and gravel delivered into wagons. A 10 H. P. engine will operate the machine. The amount of water required will depend upon the amount of clay in the sand and gravel, and in most all cases this water can be settled in the pond and used over again.

This washer is made by the American Con centrator Co., Springfield, O., H. V. Croll, Sales Manager.

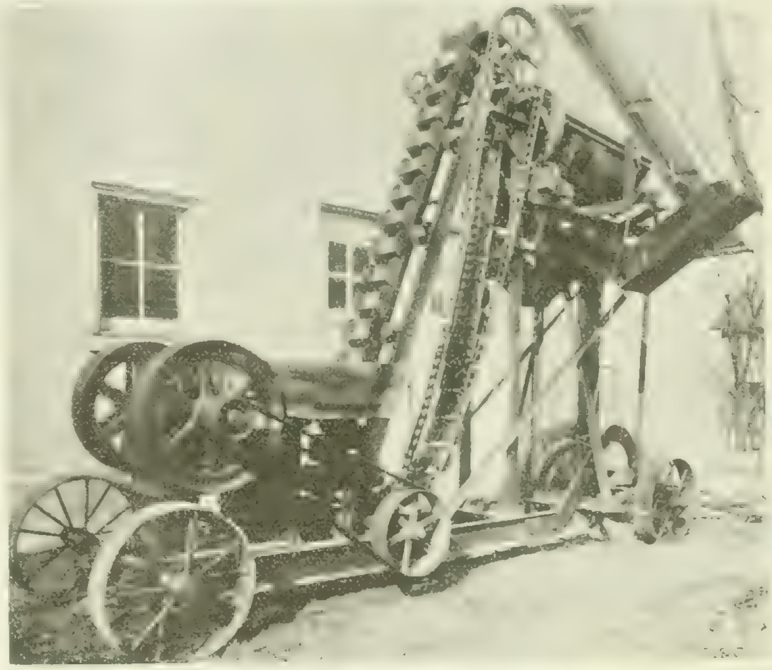
A Fifty-Ton Block and Hook.

There is shown on this page a detailed drawing of one of the largest hook blocks ever manufactured. The total weight of the block is 2,500 lbs., and it was designed for a working load of 50 tons, with a stated factor of safety of 3½. The dimensions given in the drawing show the magnitude of this hook and block. The tail bolt is 2 ins in diameter, the sheave pin 3 ins. in diameter, the head bolt 3 ins. in diameter, and the lugs of the beam which support the hook are 3½ ins. in diameter. The sheaves are 20 ins. in diameter, ground smooth in the groove for a ⅞-in. wire cable, and bushed with "Metaline" bushings. The hook and the beam which supports it in the block weigh 1,040 lbs. As can be seen by referring to the figure, the hook is ball bearing. It was forged from an 8x8-in. special forging billet, and after being flattened measures 10⅝ ins. deep by 6½ ins. wide at the back.

The block and hook were designed by the W. W. Patterson Co., 56 Water street, Pittsburgh, Pa., for the Illinois Central R. R., and will be used on a 50-ton capacity "Mitchell" derrick car.



Fifty-Ton Block and Hook.



Fort Wayne Portable Crushing Outfit Mounted on a Truck.

A Compact Portable Crushing Outfit.

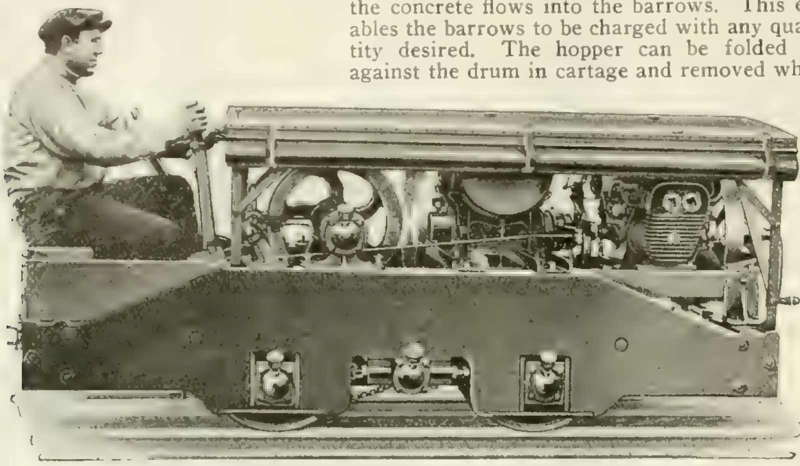
A compact, low-priced, portable crushing plant has been developed by the Good Roads Machinery Co., Ft. Wayne, Ind., which has interesting features. It is especially designed to meet the requirements of contractor, small villages and owners of private estates who desire to use comparatively small quantities of crushed stone. The outfit is also adapted to the use of bridge contractors on isolated jobs. When provided with special dies stone may be reduced to a fineness suitable for agricultural purposes.

The entire crushing outfit, shown in the illustration, is mounted on a steel truck. The weight of the crusher alone mounted on skids is 3,500 lbs.; the entire outfit shown in the illustration, four tons. The crusher has a receiving opening 8x12 ins. in size and, it is claimed, will produce from four to six tons per hour of crushed stone of a size suitable for road construction work. The crusher is built of steel. An 8-HP. engine is required to drive the crusher and screen.

A Gasoline Locomotive for Mines, Tunnels, Industrial Plants and Contractors' Haulage.

The gasoline locomotive offers distinct advantages wherever a self-contained motor independent of a central station or any other outside source of power is a quality of value. In mine haulage and for hauling about industrial plants gasoline locomotives have had extended use and have proved their utility. They have been but little used for haulage in engineering construction and contractors would, we believe, do well to consider their more extended use. For tunnel haulage in particular they present indubitable merits. In the accompanying illustration is shown a gasoline locomotive that is offered for contractors' use. use.

This locomotive has a two-piece, cast iron frame, giving weight for tractive power. The engine, a two-cylinder approved air-cooled engine, operating a longitudinal shaft carrying a friction disk. This disk drives by rolling



Gasoline Locomotive for Contractor's Use.

contact a wheel on a transverse shaft. A chain from the driven wheel shaft to a jack shaft and another from the jack shaft to the driving wheel shaft completes the driving mechanism. High efficiency is claimed for this friction drive and it is simple, noiseless and flexible. All operating parts are particularly well built and there are complete appurtenances for lubrication, gasoline storage, etc.

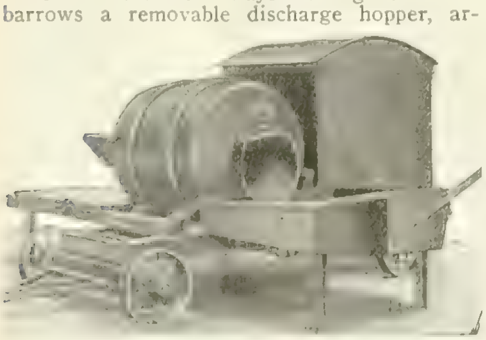
This locomotive is built for 24, 30 and 36-in. gages, and weighs according to gage 2½, 2¾ and 3 tons; its main features are tabulated as follows:

Wheel base	38 ins.
Gage	24 ins. to 36 ins.
Wheels, cast steel	18 ins. by 3¼ ins.
Gasoline capacity	20 gals.
Brakes and sand	Four wheels
Motor	Monarch, 2 cyl.
Cylinders	5 ins. by 4¾ ins.
Ignition	Bosch magneto, start on spark
Speed	0 to 10 miles per hr.
Bearings	Hyatt roller and ball thrust
Draw bar pull	800 at 6 miles per hr. for 2½ tons
Transmission	Friction and chain
Machinery	Covered
Size of axle	2⅞ ins.
Size of disc	22 ins.
Size of fiber wheel	22 ins. by 1¾ ins.

Gasoline consumption is estimated at 1 gal. per hour. Hauling capacity depends on speed, grade, curves and kind of cars, but based on 40 lbs. per ton resistance is, on level track, 22½ tons at 3 miles, 15 tons at 6 miles and 10 tons at 10 miles. Complete tabulations of these and of the mechanical details can be secured from Catalog L of the manufacturers, The J. D. Tate Co., Plymouth, Ohio.

A Removable Discharge Hopper For a Concrete Mixer.

To reduce mixer delays waiting for wheel barrows a removable discharge hopper, ar-



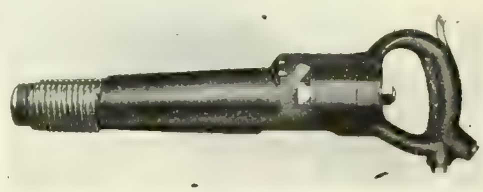
Removable Discharge Hopper for Standard Concrete Mixers.

ranged as indicated by the illustration here is being provided by the Standard Scale & Supply Co., Chicago, Ill., for all sizes of mixers made by it. As described by the makers the removable discharge hopper is placed so that the entire batch can be discharged from the drum into the hopper. Then the wheelbarrows are loaded from it, while another batch is being mixed. The hopper holds more than one batch of concrete, so that delays in placing concrete in forms by the wheelers will not effect a delay in the mixing process. A door is provided in the hopper end and by operating it the concrete flows into the barrows. This enables the barrows to be charged with any quantity desired. The hopper can be folded up against the drum in cartage and removed when discharging into wheelbarrows, or when discharging into forms or into hoisting elevators.

A Rivet Set Retainer for Pneumatic Riveters.

(Contributed.)

Several states are drafting safety appliance laws, among the provisions of which are requirements that riveting hammers embody in



Pneumatic Riveter With Spiral Rivet Set Retainer.

their construction devices to prevent the accidental ejection of the rivet set from the nozzle of the hammer. One of the most novel rivet set retainers is that now put out with the "Little David" pneumatic riveter, of the Ingersoll-Rand Co., New York.

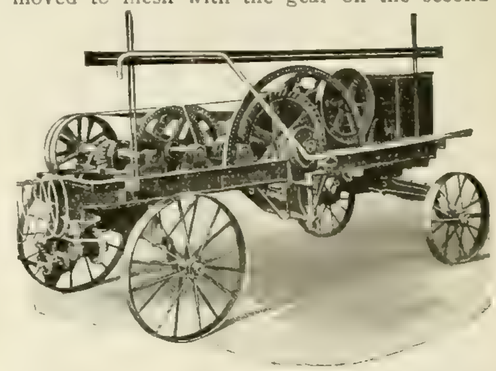
The retainer consists of a single piece of heavy spring steel, closely wound into a spiral form. One end of this spring fits over the outside of the hammer nozzle and hooks over a projection integral with the nozzle. The other end is wound to a smaller diameter. Sets for rivets over ⅞ in. diameter are formed with a coarse thread and are simply screwed into place. Sets for rivets ⅞ in. in diameter and smaller are formed with a shoulder and are slipped into the retainer while it is detached from the hammer, the shoulder holding it in place. The device prevents the rivet set or piston from being driven out, even when the hammer is run free.

A Truck-Mounted, Power-Operated Bender for Reinforcing Rods.

(Contributed.)

A bar bending machine equipped either with gasoline engine or electric motor and mounted on wheels for transportation by team is illustrated here. This machine is designed to bend any size or shape of reinforcing rod that is likely to be used in building operations. It will bend rods up to 1½ in. diameter, round, square, or deformed, and is also provided with an attachment at the rear by means of which spirals or rings of any diameter from 10 ins. up may be formed. It is, for example, used frequently for turning rings for column head reinforcement used in the Turner Mushroom system of reinforcing.

The operation is extremely simple, the action of the bending member being controlled by a lever at the rear of the machine, which operates a specially designed friction clutch inside and between two loose pulleys set on the rear shaft, these pulleys being driven by belts from the engine or motor. A one-piece pinion and sprocket is keyed to the rear shaft but left free to move laterally on this shaft. When moved to mesh with the gear on the second



Truck-Mounted Power Operated Reinforcing Bar Bender.

shaft it drives the bending member, and, when moved to the left from this position, is connected by a loose link chain to a sprocket on the spiral former, and so drives this mechanism.

The machine is fitted with gages, scaled in feet and inches, so that no marking of the rods is necessary, and so that height of truss and angle of bend can be set. Wagon trucks and steel wheels are provided so that the machine may be readily moved from place to place. Each rear wheel is provided with a square steel shaft which fits a hollow square axle so that it can be readily removed, should it be desired to have greater freedom of movement about the machine by having the wheels out of the way. The front truck can be turned under the machine.

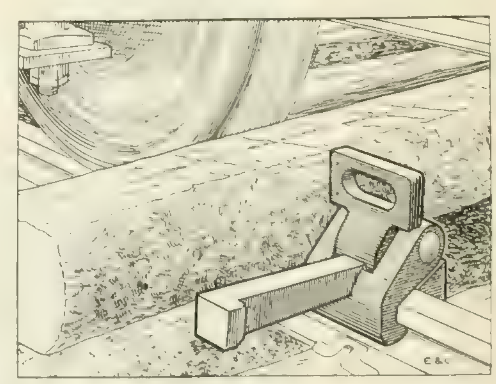
The total cost of operating this machine per ten hour day is given as follows:

Labor (3 men, one at 30, two at 25 cts. per hr.)	$8 00
Gasoline (3 gals.) and oil	.40
Interest and depreciation	.35
Total	$8.75

One man is required to direct the work and operate the machine, and two men to handle the steel. The manufacturer's statement as to performance is one ton per hour, average output, at a cost of $1.00 per ton, but this output has frequently been exceeded at a correspondingly lower cost. The best achievement has been 39 tons in 9 hours at a cost of 22 cts. per ton. The machine is manufactured by Kardong Bros., and is for sale by Robert B. Taplin, 514 Free Press Bldg., Detroit, Mich.

A Rail Clamp for Steam Shovels.

The steam shovel rail clamp shown by the accompanying sketch works like a pair of ice tongs. It grips the ball of the rail and the



Bates Rail Clamp for Steam Shovels.

wedge is driven across the top of the rail; it can therefore be placed anywhere on the rail, even directly over a cross-tie as illustrated There are only two parts, the wedge of hardened steel and the tongs which are steel castings hinged by a heavy pin. These clamps are made in three sizes for 50-60 lbs., 70-80 lb., and 90-100 lb. rails by the Bucyrus Co., Post Office Box Z, South Milwaukee, Wis.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design and to Methods and Cost of Construction

Volume XLII. CHICAGO, ILL., JULY 22, 1914. Number 4.

An Important Series of Articles on Water Tunnel Construction.

The first in a series of four articles on the construction of water supply tunnels in the Metropolitan Water District of Massachusetts is published in the Water Works section of this issue. These are all methods and cost construction articles, except the fourth which relates to work now under construction and for which cost data are not yet available for publication, but in each case the essential features of the design are illustrated and described. The scope of the several articles is here indicated as follows:

The first article pertains to the construction, under free air and by contract, of a land tunnel through rock, and the laying of 60-in. cast iron and 80-in. steel pipes and appurtenances. This article gives the methods and unit cost of top soil excavation and surfacing, earth and rock excavation in open trench, refilling open trenches and building embankments, tunnel excavation, crushing stone, placing the concrete lining in the tunnel, placing the concrete cover over the large steel pipe in open trench, brick masonry, grouting fissures in the tunnel hole, placing the cement mortar lining in the 80-in. steel pipe, and laying 60-in. cast iron and 80-in. steel pipe. The wages paid are stated and the general costs of materials and miscellaneous expenses are recorded, and the construction plant is illustrated and described.

The second article gives the methods and cost of constructing a brick-lined tunnel under Chelsea Creek for a 36-in. water main. This work was carried on by the pneumatic process and by day labor. The treatment of the subject is analogous to that indicated in the foregoing digest of the first article. The third article gives the methods and cost of extending in 1912 a 24-in. water pipe tunnel, which was constructed in 1900, under the north channel of the Mystic River at Chelsea Bridge, Boston. The fourth article, as stated, relates to the design features of a subaqueous water pipe tunnel, under Chelsea Creek, which is under construction at the present time.

All of these articles, as may be inferred from the foregoing, contain much specific information of great interest and permanent value to water works engineers, to water department officials and to contractors, and to such readers the articles are especially commended. The articles are contributed by Mr. William E. Foss, who, throughout the construction of the works considered, has been Assistant to the Chief Engineer of the Metropolitan Water and Sewerage Board, and who, therefore, writes with full knowledge of the matters discussed.

Concerning Strength Specifications for Lime Mortar.

The continued use of lime mortar in walls and in various classes of work has resulted in attempts to specify its compressive strength and other properties. Until recently the compressive strength of lime mortar was of little importance, as the conditions under which it was used did not require any considerable degree of strength. However, the construction of high storage buildings, warehouses and similar structures, in which the walls are sometimes designed to carry a considerable part of the load in addition to the weight of the wall itself, has made the factor of strength an important one. Moreover, the practice of breaking joints taken in connection with wind pressure and settlement of foundations may cause large shearing or even tensile stresses in the mortar. As the hardening of lime mortar is due to the action of the carbon dioxide of the air it is evident that the rate of hardening and consequently its strength at any age will depend largely upon the conditions under which it is used. The fact that lime mortar has been forced to compete with Portland cement mortar, coupled with the realization that the factors influencing the strength of these two mortars differ widely, have made it imperative to determine more definitely the properties of lime mortar and the conditions under which it may be safely used. Some important data on this subject are given in a paper by W. E. Emley and S. E. Young, presented at the recent annual meeting of the American Society for Testing Materials, some of the conclusions of which are herewith given:

It was concluded from numerous tests (a) that the compressive strength of lime mortar is of value for comparison only when the conditions under which it was made and stored are known; (b) that it is useless to specify any value for the compressive strength unless the method of measuring it is also given in detail, as well as the consistency and the condition of the atmosphere in which the specimens are stored; and (c) that the real strength of the mortar as used is probably greater than the measured strength, so that this latter value can only be comparative at best and cannot be used in any engineering calculations. The tests demonstrated that the extreme difficulty of maintaining uniform atmospheric conditions over long periods of time, and the necessity of specifying every minute detail of manipulation, made a strength specification of little value, and did not warrant the establishment of any definite relation between the strength as measured in the laboratory and that which may be expected in practice.

Some tests were also made to determine the effect of adding lime to Portland cement mortar. It is well known that the working qualities of cement mortar are greatly improved by the addition of small proportions of hydrated lime, and the practice of combining these materials has become quite common. Although the tests are not conclusive they point to the following conclusions: (a) The addition of small quantities of hydrated lime does produce a weaker mortar, but this decrease of strength is quite small if the volume of lime is not more than one-third of that of the cement—this statement being true whether the mortar is stored in air or under water; and (b) the results obtained with dolomitic hydrate were at least equally as satisfactory as those using calcium hydrate. These conclusions are based upon the use of mortars whose consistency was much thinner than that ordinarily used for Portland cement, although other investigators using different consistencies have reached similar conclusions.

The County Engineer of Yesterday and Today.

Within the past decade the county engineer has come into prominence in this country. A few years ago the position of county surveyor, or engineer, in the average county of the United States went begging. The honor was an empty one. His chief duties consisted of running an occasional land line and selling county maps. Today in many sections the county engineer is the highest salaried officer of the county and his duties are varied and interesting. The increased importance of the duties of this official is probably due in a large measure to the activity in road construction of recent years.

In England and in some sections of this country this post has always been one of honor and importance—often decending from father to son—and justly so, for as a rule a man to successfully fill this position must possess a combination of tact and engineering and executive ability which is native in few men.

The successful modern county engineer is first of all an executive. His work consists largely of superintending foremen or inspectors, or both. His tact and uniformity of spirit must be exceptional since his dealings with men in many different walks of life are frequent and of importance. Affairs pertaining to the acquisition of right-of-way, the purchase of supplies, the letting of construction contracts, the hiring of labor, the organization and regulation of work, accounting and many similar matters calling for the exercise of judgment pass through his hands.

As an engineer he must be especially familiar with the details of road and bridge building, since a large portion of his work is in connection with these classes of construction. The engineering features connected with road maintenance are becoming important and require his constant personal inspection and judgment as to proper methods and tools to use. In addition to this knowledge he is required to know something of the operation and repair of tools and machinery using horse, gasoline and steam power, the composition and durability of paints and bitumens, methods of accounting, the properties of road materials, the purchase and care of horses and mules and other matters small and large, too numerous to mention.

The writer has in mind an engineer for a typical rural county who receives a salary of about $1,800 per year. He has under his direction three foremen and two inspectors, and has supervision of about 200 miles of road in all stages of improvement. Mules, machinery and equipment of all classes are in his charge, the value of which approximates $20,000. During the course of a year he directs construction work amounting to $150,000. In looking after their work he travels by horse and automobile several thousand miles a year in all kinds of weather. It is a man size job.

Perhaps the most attractive feature of this work to many engineers is its independence. While his term of office is, unfortunately, uncertain, and many undesirable factors of local politics enter, the county engineer is, nevertheless, his own boss. Quite frequently, also, an opportunity is afforded for outside work, which often adds materially to his income. Moreover, if he has leanings toward a farmer's life—as many county engineers have—an opportunity is afforded of carrying on this class of work in addition to his regular duties.

It is desirable that the standard set by county engineers throughout the country be high. Instead of belittling this class of work it behooves engineers in general to realize its advantages and lend their efforts toward the eradication of some of the attendant evils which make it objectionable at the present time.

GENERAL

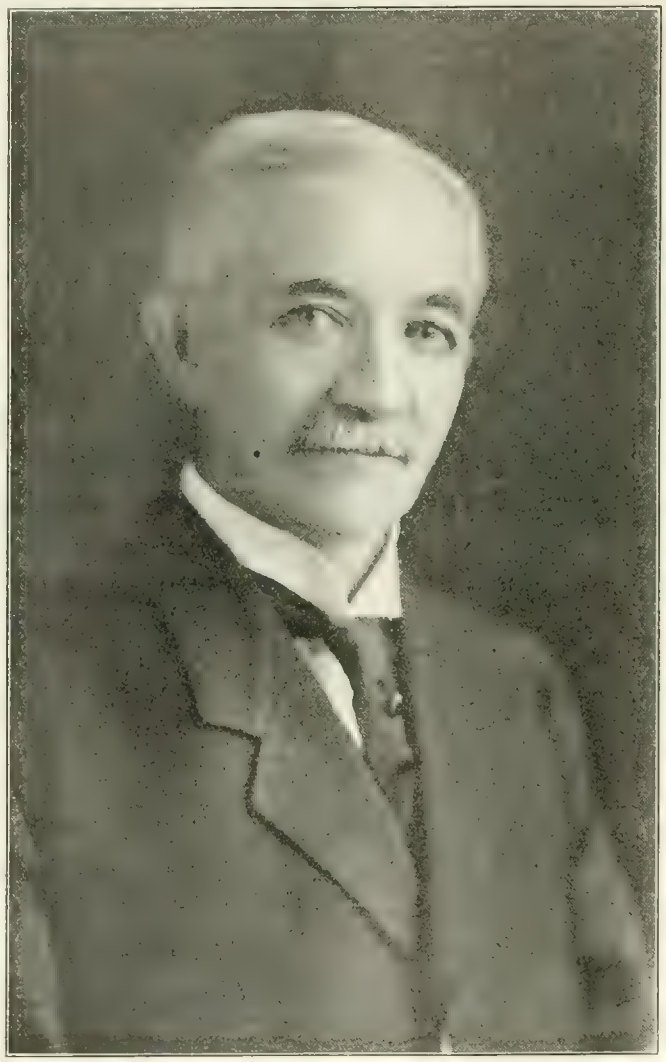
The Relation Between Research and the Activities of the American Society for Testing Materials.

Annual Address by the President, Arthur N. Talbot.

The American Society for Testing Materials, in its 16 years of activity, has had to do with the methods of testing engineering materials—standardizing methods to produce uniformity in testing and to secure tests which will properly differentiate qualities and distinguish between acceptable and rejectable materials. The society has therefore discussed the desirable properties of materials, and naturally it has dealt largely with the qualities and defects and with the nature of the uses of materials, and with the requirements accompanynig those uses. As one function of the society is to bring the producer and the consumer on common ground, the society has dealt with specifications and requirements for the material; definite, explicit and sane requirements, acceptable alike to producer and consumer, and these have come to form the basis for commercial contracts over the entire country. To secure the formulation and adoption of such specifications has involved an intimate contact between producer, consumer, and unattached expert in ways that have in themselves resulted beneficially to producer, consumer, and expert, as well as to the society itself.

In considering these activities of the society, it is apparent that the constructive work of the society must be based upon information—definite, trustworthy, complete information, on the properties of materials, their action under stress, load, and time, and in combination with other parts in a machine or structure. Obviously, to get hold of this information and to pass upon its value must be within the province of the society. The element of investigation, then, is vitally connected with the activities of the society. It is to the question of research in engineering materials and the relation between research and the activities of the society that I invite your attention for a brief time.

It is unnecessary in this audience to establish the need for definite, trustworthy and complete knowledge of the properties and actions of engineering materials. There was a time when general information, common belief, or even say-so opinion was all that was available for outlining specifications upon which material was to be purchased. Since the formation of the society, great advance has been made in our real knowledge of materials, very much of it being due to the activities of the society itself, and we are now where the value of the fruit of research is appreciated. As soon as the surface is scratched over, the need of systematic investigation becomes apparent. Ultimate progress involves a searching, critical, and thorough inquiry and investigation into the facts and principles relating to the subject. Real research lies at the basis of that complete and definite knowledge of the properties and actions of engineering materials that is so essential in the formulation of specifications for the selection of materials, and in the understanding of the nature of the resistance of the machines and engineering structures in which they are used; and more and more as time goes on will the society need to avail itself of the fruits of research.



ANTalbot

Fortunately for us, the agencies for gathering knowledge and for conducting research are many and varied, and their number and opportunities are increasing. First in time, and in the past perhaps first in opportunity, may be named the producer or manufacturer. Next to the producer may be mentioned the consumer, whose interests in research should be as large or larger than the producer's, though neither of these agencies has always realized its investigational opportunities and responsibilities. The private laboratory of the consulting engineer, testing engineer, or chemist and the independent research constitute another agency. The various government laboratories, the engineering experiment stations, and the laboratories of engineering colleges and technological schools may be grouped together as still another agency. The list would not be complete without including an agency which has been especially productive in effective investigational work in the past few years, and which may be expected to render still more valuable service in the future—I refer to co-operative work by scientific and engineering societies and their committees in connection with producer, consumer, scientific laboratories and individuals.

It is not easy to discuss this topic along general lines, since the problems to be considered vary so greatly in nature and complexity. Many of the matters are very simple—involve only a description of common wants or of the product commonly used, or comprise only a statement of the defects commonly found or of conditions to be avoided in manufacture or fabrication. Again, certain problems involve the possibility of manufacturing material having the desired qualities within a limit of cost of practical production. Information of the highest value is obtainable by systematic observation on the action of materials in use, their action under stress, their development of defects, their resistance to the effects of time, provided the exact nature of the materials is also known. Another class of problems requires the determination of internal action under stress or of resistance to external agencies, and these are generally complex problems which need special investigation, time-consuming and expensive research, diligent examinations and analysis, sometimes running over many years. I have in mind a research on a topic that is generally considered one of the simple matters connected with reinforced concrete, in which the results of the first two years' work were thrown away or used only as preliminary information or as a reconnaissance survey, and on which six years more were spent in a series of tests, the program of the tests of each year depending directly upon earlier tests and the interpretation and analysis of the results of the work of all the different years being interrelated and interdependent. Many of the problems still before the society are even more complicated and their solution will require long and careful investigations and critical and skilful analysis of data. In some lines of investigation the outcome may be only negative, and negative results are to be expected. Such tests are not wasted work, for if the conditions, methods, and data are placed on record, the results will be helpful in clearing the field for another method of attack.

There was a time when the knowledge of the properties of certain engineering materials emanated principally from the producer, or was even held by him as trade secrets. The manufacturer has special opportunities for learning the qualities of his output—in certain directions at least. He needs to know something of its properties, for during the initial stage of business he must develop his product and generally he must work to develop its field of usefulness and applicability in order that his business may grow to proper proportions. Competition compels him to know about his product and perhaps about the product of his competitor. There was a tendency then to withhold information; the brand of the established house was held sufficient as a specification. Time has thrown purchase by brand into disuse except for small purchases. The brand of a cement or of boiler plate still has selling value, but the producer must contract to deliver an article which will comply with definite specifications. Buying by specification is considered to be of great advantage to the consumer, for it permits of intelligent competition and gives an agreed statement of what properties are wanted, and is it not therefore advantageous to the producer also? Granting purchase by specification, knowledge of the properties of a material is of great importance to the producer. And the producer is in a position to learn about his product. He is on the job continually. His chemist and his testing engineer keep tab on the work, and these trained and experienced men are able and willing to conduct researches to learn more of the material and of its action under varied conditions of service. A large amount of valuable investigation has been carried on by the producer. May we not expect with proper recognition of their work and with proper suggestion and co-operation that the fruits of the producer's research laboratories may multiply greatly in amount and in value to the public?

The consumer is in a different position; he has to live with the material; he knows how it works, how it wears, whether it breaks and what the disastrous consequences are. He comes to think he knows what qualities he wants in the material, and he may ask for these qualities. The producer may have to tell him that it is not commercially practicable to make such a material, or he may insist that the cost would be prohibitive. The consumer may find that it will be to his advantage or to the advantage of his client, the public, to use the better material even at an increased cost, as in the case of the rail which reduces the chance for accident and loss of life. The consumer has many opportunities to learn the peculiarities of a material or a product; and the intelligent systematic record and analysis of failures or defects, of service and durability, add greatly to our knowledge of the properties of materials and may be made to form an even more important source of information. The laboratories of the consumer have been of great assistance in determining properties and in formulating adequate requirements, as is instanced in the work of the laboratory of the Pennsylvania Railroad Co., developed so highly under the leadership of our lamented former president, Doctor Dudley. But here again, with an increasing number of laboratories and with increased interest on the part of the consumer in the work of their laboratories, may we not expect, as the years go on, that the contributions to knowledge from these laboratories will increase many fold?

The next group, the private research laboratory, may at times serve the producer and at other times the consumer, in which cases it may be classified with the producer's and the consumer's laboratories; or it may serve as an independent research laboratory to investigate and report independently upon a research problem which is of common interest. This agency is also increasing in usefulness and we may expect its influence to grow with the years.

Government and college laboratories occupy an independent position. They enjoy the very full confidence of the public who feel that these laboratories are not connected with special interests and that research problems will be handled with judicial fairness and impartiality. These laboratories enjoy the further privilege of being able to devote time and money in researches into the principles of action and fundamental nature of materials, which are seemingly of remote applicability and do not appeal directly to the producer or consumer, but which finally, after a number of pieces of work have been completed and fitted together, may prove of intense practical value. Among these laboratories, the testing laboratories of the national government are rapidly assuming a place of inestimable value in the bearing of their work on the industrial and engineering interests of the country. A recent bulletin of the society named a number of these government laboratories which have expressed a willingness to co-operate with the committees of the society. The expansion of the Bureau of Standards in recent years is an instance of the growth of government testing, and presages something of future opportunities in investigations in engineering materials. In the field of college laboratories, there has been a development that may surprise those who have not kept informed. At least six engineering experiment stations have been organized in connection with the engineering departments of state universities. Research laboratories of similar purpose are connected with other engineering schools. Still other schools are doing investigational work in an informal way. From all these sources are coming contributions to engineering knowledge of great value in the form of bulletins and other papers. As these laboratories become better organized and their staffs acquire training and skill, is it not to be expected that this agency for research will become a powerful factor in the widening of our knowledge of engineering materials?

And now I come to the last of the research agencies to be named, one which I feel is most important in the work of the society, and one on which the future progress of the society greatly depends—co-operative work between the committees of engineering and scientific societies and the laboratories, facilities and store of information of the producer, consumer, and independent laboratory. This form of activity takes advantage of the knowledge and experience of the man who is familiar with the processes of manufacturing and their peculiarities and limitations; of the man who knows the needs and the shortcomings of the articles in use; and of the unattached expert who may be able to help to correlate the views and interests of the other two. It gives breadth and depth to the scope of the inquiry and purpose to the method of testing. It assists in the analysis of data and the formulation of conclusions, and may aid in making the action of the society more readily acceptable to all interests concerned. Doubtless, however, the element of patience may need to be developed in such an organization. You are all familiar with this form of activity in this society and in other societies, and I need only refer to a few instances to bring to your mind the results which this agency has already accomplished. A case at hand is the report of the Committee C-6 to be presented at this meeting, a very valuable piece of co-operative work on the methods of testing and on the available strength of both clay tile and concrete tile, made possible through the facilities of the Engineering Experiment Station of Iowa State College. Committee C-3 has now available valuable data on the strength of the building brick that are now in use in various parts of the country, secured through co-operation with college laboratories, and the same committee has recommended a standard method of testing paving brick based on a somewhat similar form of co-operation. The work of the committees on iron and steel and on preservative coatings furnish many similar examples. The report of the committee on concrete and reinforced concrete, which brought order out of chaos, was based on co-operative research work. Other societies are using the co-operative agencies. Several recent reports of committees of the American Concrete Institute include valuable research material obtained in this way. The American Railway Engineering Association has utilized this agency extensively, though its most important research, that on rail, which is destined to put rail purchase and rail use on a new basis, has been carried on almost independently. In the work of the committees of the American Society of Civil Engineers, the extensive tests of steel columns now being carried out form an interesting example of co-operative work. Perhaps the recently constituted joint committee of the two societies last named formed to investigate the stresses in railroad track—rail, fastenings, ties, ballast and roadbed—will prove to be the most extensive piece of co-operative work yet undertaken.

I have named these examples of co-operative work to freshen your memory and to suggest the possibilities of this form of research. I want now to advocate the extension of its use as an activity of the society and to urge upon our committees the value of co-operative research work. It seems to me, too, that members may well encourage the making of larger appropriations for national, state and semi-public laboratories, and may properly ask manufacturers and consumers themselves to support liberally research in all lines touching on engineering materials. The initiative for this co-operative work and its direction may well be left to the committees of the society, whose members are so intimately interested in their work. Possibly some day there may need to be a general research commitete of the society to co-ordinate work and to stimulate opportunities. What is wanted now is a fuller understanding by the committees of their needs and opportunities and especially of a formulation and direction of the problems before them. Make co-operative research work a feature of the society, and the fruits will be visible everywhere.

Research requires patience, diligence, and skill as well as knowledge and opportunity, from outlining the problem and separating the essentials from the non-essentials to analyzing the data and drawing conclusions. See that men of ability and training are selected for such work. Express appreciation of the results obtained and encourage the extension of the work. For one function of the society is to widen our knowledge of materials.

Costs of Surfacing Track and Features Which Influence That Cost.

(Contributed.)

ORGANIZATION.

In doing any kind of track work with common laborers, the organization or distribution of the men is a matter requiring careful consideration and close attention, for it is the proper disposal of men which more than any other one thing effects the amount and quality of work done. It has been said with truth that a gang of poor workers well organized is a better gang and will accomplish more than will a gang of good workers poorly organized. Time and confusion are saved by assigning each man his task, and requiring him to remain as placed until otherwise ordered.

The value of organization is not fully appreciated by all track men, and is badly misunderstood as respects track work by those unfamiliar with the subject. The word not only designates placing men so that they will work harmoniously and advantageously, but it implies also that each man shall be placed at the work for which he is best fitted. One can readily see that this result cannot be immediately attained, but it is the idea aimed at; and not only must the physical makeup of the man be measured, but a man's disposition, age, willingness, natural skill, and familiarity with the work must also be considered. A man who shows a disposition to lag will many times hold his own against a more willing workman if the work is arranged on a competitive basis. For instance, if a laggard is placed where he must rehandle a certain amount of material which is being passed to him by another, he will, in many cases, do enough work to keep from being "snowed under."

The amount of work accomplished by the individual is very closely related to gang organization. The best men should be put in the lead, and wherever possible the work should be arranged so that each man will have to do an equal share with the head man, or else fall behind and be easily detected. It is characteristic of the laborer experienced in track work, that he is loth to admit the superiority of another man. When such a man is placed so that the amount of work he accomplishes is directly measured by the amount some other man does, he will generally do his share.

The greatest amount of work will be obtained from most gangs by treating them considerately, and this policy makes the work more pleasant for both laborer and foreman. "Driving" men is being done away with to a great extent, as it is becoming difficult to keep laborers when such a policy is pursued.

The idea that men should be treated considerately does not mean that discipline should be sacrificed. The foreman's discipline should be strictly maintained under all circumstances. He should personally see to it that each of his orders is obeyed. In case an assistant foreman is employed and the gang is not separated, all orders should in general be given through the assistant foreman. This will to a great extent prevent conflicting orders being given to one man. The assistant should be backed up by the foreman in all cases in order to maintain his authority.

In order to provide maximum and easy supervision, the men should be kept as close together as is possible, without interfering with each other. The scattered condition of a gang may be due to poor organization or to irregularities occurring in the work, and a foreman will show his ingenuity in arranging the work and the laborers so that the gang is kept compact.

PREPARATION OF SUB-GRADE.

The American Railway Engineering Association has specified that before ballasting a new track "all dirt above the bottom of the ties shall be removed." This is a very necessary procedure, for after the track has been ballasted, the part of the sub-grade directly under the track will be compressed and will settle, and this action will continue possibly for a number of months. So that even if the top of the grade is made level before initial ballasting, the top of the dirt sub-grade will gradually assume a longitudinal trough or ditch-like surface. The depth of this trough has actually been found to be from 8 to 10 ins. on a grade in Illinois, from which the ballast was removed.

Ballast of nearly every kind—gravel, cinders, crushed stone—is much more pervious to water than the dirt which is in the embankment. Rain water falling on these ballasts will soon penetrate to the dirt sub-grade. If there is a basin beneath the ballast, water will be retained on top of the sub-grade. If the original depressions made by the ties are left in the sub-grade, the greatest accumulation of water (which will cause softening of the track foundation) is immediately beneath the tie where the greatest pressure comes and where the greatest resistance should be. The softening of the grade will tend to allow the ballast and the track to settle further and thus make the condition still worse. All water which collects in this manner must soak through the dirt of the sub-grade till it finally finds an outlet. If, however, the surface of the sub-grade were higher in the middle than on the shoulders or even if it were level, the water which penetrated the ballast would find a natural outlet along the top of the more impervious dirt of the sub-grade.

The usual width of roadbed for single track main line is from 18 to 20 ft. and for double track from 30 to 33 ft. For double track, therefore, it is even more imperative that the surface of the sub-grade be level, or better yet, slope toward the outside. The American Railway Engineering Association specifies three widths of roadbed, 14, 16 and 20 ft. for single track roadbeds, but common practice seldom allows anything less than 16 ft. The distance from the end of the tie to the edge of a 16-ft. roadbed would be 4 ft.; to the edge of a 20-ft. roadbed would be 6 ft. A specification for a sub-grade should, I believe, require that the middle 10 ft. of the grade be 6 or 8 ins. higher than the shoulder. If the width were made less than 10 ft., the difficulty of laying track would be unduly increased.

HEIGHT OF RAISE, TAMPING, ETC.

Surfacing new track differs essentially from raising track on an old bed. Track must be raised high, and because of great inequalities in the solidity of the roadbed, which will cause unequal settling, it is not quite so important that the "top" or surface of the track should be the best obtainable.

Giving the new track a high lift is especially essential if it is down and the mud shows occasionally between the ties. Even the second lift on new truck will not be of much account if it is only 2 or 3 ins., unless the sub-grade is of unusually good quality. Of course care must always be taken not to raise the track so high that sun kinks will occur. With heavy, coarse gravel, there is not as much danger of sun kinks as with fine gravel. Good judgment must be used in putting up the track only so high that the ballast dressed in will be sufficient to hold it in surface and line.

The rule has been generally accepted that tie centers should not be tamped beyond about 15 ins. from the rail, in gravel ballast, the idea being to prevent what is called center bound track. This rule is undoubtedly a good one in regular maintenance work. There is danger that the centers will be tamped up too stiff, causing poor riding track and broken ties. On construction work, however, full center tamping is usually advisable. When surfacing is done by contract there is practically no danger of getting the centers too well tamped, and where the lift is high, and the grade new, the settling of the roadbed will be the most serious feature and will entirely overshadow the effect caused by too stiff center tamping.

While most track men are entirely familiar with surfacing track on an old bed, there are some differences in surfacing track on a new bed which should be brought out, together with a suggested organization.

DISTRIBUTION OF MEN AND WORK PERFORMED BY EACH.

The gang should be organized somewhat as follows, varying the number of men filling in and tamping track as conditions require:

1 spot board man	1 level board man
2 jack hole diggers	1 hammer man
6 jack men	8 men filling ends
4 jack tampers	16 men tamping ends
4 men filling in for jack tampers	8 men filling centers
	12 men tamping centers

The spot board man sets up the board from sights made by the foreman or competent assistant, unless sufficient ballast is unloaded to make a complete raise to grade stakes. Two spot boards should be used, and the man sets one spot board in advance and has it ready so there is no delay of the jack men.

Of the three jack men on each jack, two carry the jack ahead and the third carries the jack board ahead (preferably a base plate with a wire handle), and places it in the next jack hole. Extra jack boards and a man to do nothing but place these in advance of the jack men, are sometimes advantageous and increase the amount of work done.

The level man levels up the track and also sets the spot board block back for the foreman to sight across, unless one side of the level is trimmed down to the correct height for a spot board block and is used for that purpose. After the jacks are let off, the hammer man knocks down track by hammering the ties if necessary. He should also carry a spot board block so that the jack men will not have to wait for blocks while a joint is being knocked down.

It is perhaps needless to say that the men around the jacks should be the best men, as the amount of work accomplished depends so greatly upon them.

The foreman should sight from about the same distance back of his jacks each time, and should raise both centers and joints with the spot. It is a waste of time and energy for the jack men to raise the joints first and then double back and raise the centers.

The tampers should be organized to tamp the track in the following manner; if there are four pairs of tampers on the ends each side, have the head pair tamp every fourth tie, the next pair tamp the tie behind that, etc. The center tampers should follow a similar system. Besides keeping the men at work close together without interference, this method accomplishes two other very desirable conditions, viz.: it makes it possible to get more work out of men who would soldier under the old methods; and more important still, the track as a whole is tamped in a more uniform manner. Under the old method where each pair of tampers was assigned a rail or half rail length, the sections tamped by the better tampers stood up better than the rest. Where the good tamping is mixed with the poor the whole track is more likely to settle uniformly.

A gang of 60 or more men should have a first class general or head foreman, and at least two assistants, one of whom should "follow the jacks," that is, sight the blocks for the actual raising of track; the other should have charge of finishing and lining after track is raised. The general foreman should personally see to all matters pertaining to organization, and to the general conduct of the work. And it will fall to him to see that the tamping is properly done, as his assistants will be kept busy with their duties and unable to look after this very important item. The placing of men is likely to need changing several times every day, for conditions change as the work moves along. For instance, the uneven distribution of ballast may make it necessary to make a change in the lineup. If gravel is unloaded too heavily, which is often the case, the number of jack hole diggers will have to be increased, while the "swampers," or men filling in can be reduced; if the unloading is too light, the swampers are not likely to be able to keep ahead of the tampers. All these things and many others are matters for the foreman to adjust. Highway crossings also force a readjustment of the force, as they have to be planked and graded as soon as they are crossed. The foreman who is "onto" his job can generally spot some man in the gang, usually some old man who has followed track work for years as a means of getting his living and whiskey, who, while unable to keep pace with the younger men at tamping, is worth his wages to look after such jobs as fixing up crossings, and other items like that. The foreman will find that there is enough to keep him busy without having to stand over a crossing that is being put in shape, and will appreciate an old man like this.

The foregoing applies alike to either contract work, or work done by the company forces. Contractors nearly always see the economy of having ample supervision, while on the contrary railway companies are usually averse to having enough assistant foremen to properly look after the work. In these days, the laborers being nearly all foreigners who do not understand the English language, it is absolutely necessary to have more supervision than is usually accorded by the railways if the work done is to be up to the standard it should be.

COSTS.

The costs given below are in two parts. In the first portion, the rate paid laborers was $2 per day, the work being done in the cool, early spring months, from April 17, to May 28, 1913. The costs in the second part represent totals where the ordinary rate for laborers was $2.25 per day, work done in the summer months from June 7 to Oct. 3, 1913.

The total cost chargeable to track raising during the early period was $5,064 for raising 60,100 ft. of track, average raise 6 ins., with laborers at $2 per day. This total figures out at $445 per mile, including lining and dressing up the track to standard.

The total costs chargeable to track raising during the period with laborers at $2.25 per day, was $7,850 for 74,400 ft., average raise 6 ins., which figures out $621 per mile. This figure includes dressing track to standard.

The average raise was 6 ins. Ballast was good clean gravel, but was too fine to make the best ballast. It was easily and quickly tamped, however. The track was shovel-tamped only, and was tamped clear across ties, as the softness of the grade precluded the possibility of center bound track.

Especial emphasis should be given these costs as they give comparison between:

(1) Costs of doing work with $2 labor and $2.25 labor.

(2) Unit costs in the early spring and unit costs in the hot summer months.

(3) The effect on costs of breaking up and reorganizing a gang.

The first gang struck for a raise of wages which was refused. Then new men could not be obtained at $2 per day, so it was necessary to offer $2.25 per day. A very poor class of labor was obtained for the second gang. This was due to the great demand for laborers in the summer months, and the resulting scarcity of good laborers. The gang hired in the early spring was American labor, while many foreigners were included in the second gang.

The difference in quality of laborers obtained on this work bears out very strongly the contention of track men that a much better class of labor can be obtained in the spring than in the summer. The cool weather also has a very marked effect on the costs, as shown in this work.

The effect of breaking in or organizing a new gang is not generally realized by many railway men and contractors. The costs were so much greater that the contractor in this case would have been justified in raising the wages of the old gang to $2.50 or $2.75. Such occurrences are all too frequent. An experienced and reliable man is refused a raise, and then it is found necessary to offer a considerable higher salary to obtain even a green man to do the work, frequently one who cannot even approach the efficiency of the former man.

It is safe to assume that the difference in efficiency between the better men obtained in the spring and those obtained in the summer will be at least 25 per cent. The work on the above was 42 per cent cheaper in the spring, but the cool weather was undoubtedly responsible for part of this. It is fair to assume, however, that the old gang would have continued with at least as high effieciency as they had attained, and on this basis the last 12 miles put up at $621 per mile represented a loss of $2,032.

The costs, especially during the early spring, are very low, much below the average of surfacing by railway forces. There are two reasons for this low cost—ample supervision and quality labor. Few railways would allow three assistants and one general foreman at a total of $15.50 per day to supervise a gang averaging about 70 men. The writer's experience would indicate that the usual supervision would consist of one foreman and one assistant, totaling about $6.50 per day, for this size railway company gang. And with this inadequate supervision they pay 25 cts. to $1 less per day for laborers, and get a class of men which is really in greater need of supervision.

There are two other features that attention should be drawn to: (1) The same foreman handled both gangs: (2) Costs in each case include the extra cost of organizing, each gang being made up from new shipments of men.

COMPARISON OF COSTS.

There are no other authentic costs available with which to compare the above. Some costs were found of work done in the early 90's, but these costs were so low as to give rise to a doubt of their reliability. The writer's experience would indicate that $445 per mile for giving a 6-in. raise and dressing up track, is one that it is impossible for many railway company gangs to even approach, at the prevailing wages of $1.50 to $1.75 per day.

CONCLUSIONS.

The conclusions to be deduced from the above is that it is highly profitable to:

(1) Allow ample supervision—say one assistant foreman to each 25 men.

(2) Hire men as early as possible in the spring.

(3) Pay quality prices and obtain quality labor.

(4) Push the work vigorously in the cool months.

(5) Use every means to retain an experienced and organized gang—it will pay well.

(6) Raise the wages of old men rather than hire inexperienced, incompetent men.

Cost of Repairs of Drills Employed in Tunneling.

From data collected by personal visits to and special reports from a large number of tunnels, Messrs. D. W. Brunton and J. A. Davis, in Bulletin 57, Bureau of Mines, present the following statement:

From September, 1905, to March, 1906, hammer drills were employed at the Gunnison tunnel with a drill-repair cost per machine of 13 cts. per foot of hole drilled; but when piston drills were substituted the repairs were reduced to 3 cts. per foot. In addition to the cost of materials these figures include also a charge for the labor of the machinist making the repairs, which is not embraced in any of the values which follow. This fact must be considered in making comparisons. Two years later (September, 1907, to August, 1908), in driving the last 3,000 ft. of the Yak Tunnel, the cost of materials only for repairs to the hammer drills employed was only 1¾ cts., approximately, per foot of hole. At the Marshall-Russell tunnel, where hammer drills were employed, the average cost of drill repairs from June, 1908, to June, 1911, was 1½ cts. per foot drilled. Piston machines were used at the Strawberry tunnel from January, 1909, to September, 1911, the cost for repairs being nearly 2½ cts. per foot drilled. On the Little Lake division of the Los Angeles aqueduct, where hammer drills were employed, the average cost of drill-repair materials from July, 1909, to May, 1911, as shown by Table I, was only 24 cts. per foot of tunnel excavated. As each of the two machines in the heading drills approximately 8 ft. of hole for every foot of tunnel excavated, the cost per machine per foot of hole is 1½ cts.

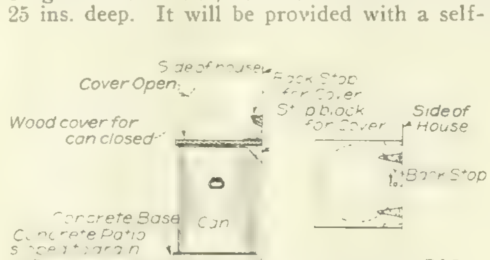
TABLE I.—COST OF REPAIRS FOR HAMMER AIR DRILLS, LITTLE LAKE DIVISION, LOS ANGELES AQUEDUCT, JULY, 1909, TO MAY, 1911.

Name of tunnel.	Distance excavated. Lin. ft.	Total cost of drill repairs.	Cost of drill repairs per foot of tunnel.
1B, south	1,030	$160.39	$0.156
2, north	926	180.72	.195
2, south	419	64.75	.154
2A, north	460	46.28	.100
2A, south	375	55.50	.148
3, north	864	113.60	.131
3, south	2,149	505.01	.235
4, north	448	67.03	.149
4, south	725	215.48	.297
7, north	1,911	399.70	.209
7, south	1,024	493.46	.482
8, north	225	146.56	.651
8, south	1,334	530.52	.398
9, north	777	230.51	.297
9, south	2,479	404.94	.163
10, north	2,626	585.78	.223
10, south	1,776	577.24	.325
10A, north	1,373	303.06	.221
10A, south	1,756	359.27	.204
Average			$0.24

For 1910 and the first half of 1911 the repair cost of hammer drills at the Carter tunnel was 2 cts. per foot drilled. At the Lucania tunnel the repairs cost ½ ct. per foot drilled, but the hammer drills had been in use only one month at the time the tunnel was visited. The hammer drills at the Rawley tunnel were new also, the repairs for June and July, 1911, averaging 1 ct. per foot of hole.

Standard Installation for Garbage Can at Panama City.

The health officer of Panama City has approved, for future use, the type of garbage can installation here illustrated and described from information published in a recent issue of The Canal Record. The can will be strongly made of galvanized metal, 19 ins. in diameter and 25 ins. deep. It will be provided with a self-



Sketch of Self-Closing Garbage Can and Installation Now in Use in Panama City.

closing cover fitting over the top of the can. These cans will be placed in an approved location, and where the floor is sloping, as in the patios of most buildings, on a concrete foundation; these bases will be at least 2 ins. high. The covers to the cans are of wood, home made, fitted to two hinges fastened to the wall of the building, and to prevent them from staying back, when opened, a wooden check is fastened to the wall at a point where it will strike the cover about midway. The accompanying sketch shows how the can appears when in proper position.

Under the new arrangement, it is mandatory on the property owners to provide their premises with these cans at their expense, whether occupied by themselves, or by tenants, and a reasonable length of time from the posting of the notice will be allowed in which to comply. Garbage cans are sold at $5.25, Panama silver, each, and the self-closing covers at $1, Panama silver, each. The concrete foundations will be installed by the health authorities at actual cost.

The adoption of a standard type of garbage can, and making its use mandatory, will do away with the heterogeneous collection of containers which, in the past, has offended the eyes and nose of the passerby on Panama City streets at about 10 o'clock in the evening, and will tend to greatly improve the sanitary conditions, as under the tolerated custom above referred to, a large proportion of the contents of the cans fell out, or the cans were kicked over into the street before the garbage wagon made its rounds.

In order adequately to care for the garbage, the health officer has arranged that, in future, three collections in every 24 hours will be made in certain districts; two collections in others, and one in the residence section. The practice indulged in by the garbage collectors ever since the collection of garbage was first begun, namely, of throwing the cans to the sidewalk or street with all the force at their command, after emptying them, has been discontinued, and this historic nuisance is now subject to a fine.

Estimating Small Tracts of Standing Timber.—The following method of estimating small tracts of timber is recommended by the New York State College of Forestry:

(1) Count all the trees in a circle 118 ft. across; ¼ acre. (2) Select a sample tree as nearly average as you can. (3) Determine how much of the tree you can saw (or use for any purpose) in 16-ft. logs (8 ft. logs count as halves). (4) Add the top and bottom diameters inside the bark, and divide by two. (Only solid wood considered, bark excluded.) This will give you the average diameter of the used length. (5) Square average diameter thus obtained, subtract 60, multiply by .8 and you will have the contents of an average 16-ft. log. (6) Multiply by the number of logs in the tree and then by 4 times the number of trees on your plot (since ¼ acre plot was used) and you will have the contents of that acre in board feet. *Example.*—Basswood, 85 ft. total height, can saw 40 ft. of it (2½ logs). Top diameter inside the bark, 10 ins.; diameter of lower cut, inside the bark, 20 ins. (average diameter 15 ins.). Fifteen squared = 225; (225—60) × .8 = 132 B. F. contents of average log. 132 × 2½ logs = 330 B. F. contents of tree. 10 trees on plot 330 × 40 = 13,200 B. F. per acre. By selecting 8 to 10 sample plots in different parts of the tract the average stand per acre may be found.

Additional Waterworks for Hongkong.—The problem of providing a sufficient supply of water for the increasing population of Hongkong has been a serious one, and several times there has been almost a water famine, due to the shortness of the rainy season. The Government has been engaged for a few years in devising methods whereby the supply would be ever sufficient to meet all reasonable demands and two years ago a contract was let to construct the Tytam Tuk Reservoir and the necessary pumping equipment.

The main features of the scheme will be the construction of an enormous masonry dam, 160 ft. high, to impound 1,500 million gals. of water, the extension of the existing pumping station near the shore of Tytam Bay, and the provision of two sets of pumping machinery each capable of delivering 2,000,000 gals. of water per day of 24 hours. The work will be completed in 1917. The estimated cost of this undertaking is about $1,200,000 gold. When it is completed the water storage and catchment area of the island will embrace an area of over 2,000 acres.

BUILDINGS

A Simplified Formula for Determining the Deflection of Reinforced Concrete Beams.

Some difficulty has been experienced in the past in attempting to check the deflection formulas commonly used for reinforced concrete beams with the actual measured deflections. The following article, which is based on a paper by G. A. Maney, presented before the annual meeting of the American

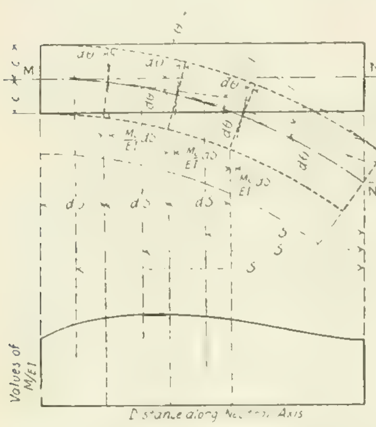


Fig. 1. Section Along Beam Before and After Bending and Curve Showing Values of M ÷ EI.

Society for Testing Materials, develops a comparatively simple formula for the deflection of reinforced concrete beams which gives results showing close agreement with those found by experiments. An examination of the deflection formulas now in use indicates that considerable uncertainty exists with regard to those particular features of the action of reinforced concrete beams which are the determining factors in their deflections. The use of existing formulas becomes somewhat difficult both because of the number of constants involved and because of the indefinite nature of some of these constants. The attempt is made in this article to show that the unit deformations in the steel and in the extreme fiber of the concrete are the only

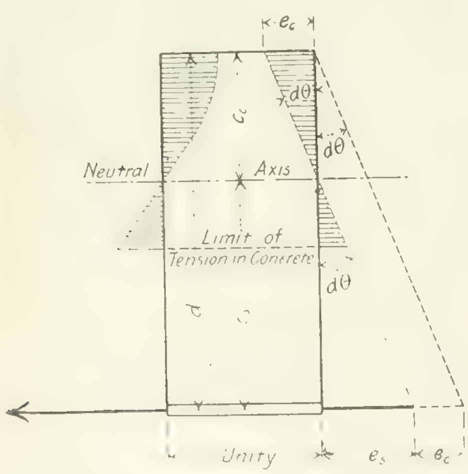


Fig. 2. Variation of Stress and Deformation Over the Beam Section and $d\theta$.

determining factors in the deflection, except the depth, span and method of load distribution.

DISCUSSION.

The formula which will be developed shows that the deflection is equal to the product of a constant into the sum of the deformations at the two extreme fibers. In any case this constant depends for its value only upon the dimensions of the beam and the method of loading. The formula is based upon a certain property of the $\frac{M}{EI}$ curve for any beam, namely, that the deflection of the beam is proportional to the statical moment of the area under this curve. The curve is obtained by dividing each ordinate of the bending moment curve by the product of the corresponding values of E and I (the modulus of elasticity of the material and the moment of inertia of the sectional area of the beam, respectively).

As a practical method of finding deflections, this property applies to homogeneous beams only. A precise statement of the property as here used, with certain modifications for reinforced concrete, is: The statical moment of the area under the $\frac{M}{EI}$ curve between any two points on the elastic curve, taken about one of these points, is equal to the vertical deflection of this point from a tangent to the elastic curve at the other point. (The beam is assumed to be horizontal.) This may be proved as follows:

Proof.—The unit stress on the extreme fiber is equal to Mc/I. The unit deformation, e, of that fiber is therefore equal to Mc/EI, and the total deformation in the distance dS along the elastic curve equals $Mc\,dS/EI$. Referring to Fig. 1, it is seen that this total deformation of the extreme fiber divided by the distance, c, from that fiber to the neutral axis is equal to the change of slope, $d\theta$, in the distance dS. Or,

$$d\theta = \frac{Mc\,dS}{EI} \div c = \frac{M\,dS}{EI} \quad \ldots\ldots(1)$$

It will be seen that the term $M\,dS/EI$ is equal to the area under the M/EI curve for the distance dS along it; that is, $d\theta$ is equal to a differential element of the area under that curve.

Now let S equal the distance from any differential element dS to the point at which the deflection is desired. Then, referring to Fig. 1, it is clear that the deflection of N' from the tangent MN at M is equal to the sum of the products of each $S\,d\theta$ along the elastic curve from M to N, S being measured from N'. Or,

$$f = \Sigma\, S\,d\theta \quad \ldots\ldots\ldots\ldots(2)$$

But $S\,d\theta$ is equal to the statical moment of the differential element of the area under the M/EI curve about N', by the definition of statical moment. Therefore, the deflection of any point from the tangent to the elastic curve at any other point is equal to the statical moment of the area under the M/EI curve about the first point.

(In this proof, as in all treatments of the elastic theory, it is assumed that the projection of the neutral axis after bending, upon its original position, is the same as its length before bending.)

This property may be conveniently expressed in the following form for homogeneous beams:

$$f = kl^2(M/EI) \quad \ldots\ldots\ldots\ldots(3)$$

The value for M/EI in equation (3) will be taken for the point in the beam at which the bending moment is maximum. Then the value of k is such that the statical moment of the M/EI area between any two points, which is equal to the deflection of one point with respect to the tangent to the elastic curve at the other, is expressed in terms of this maximum value of M/EI and the span of the beam, l.

It is evident that the value of k for any case is equal to the coefficient of deflection for homogeneous beams in the well-known equation $f = k_1 W l^3/EI$ divided by the coefficient for the maximum bending moment, which in general may be expressed as $M = k_2 Wl$, from which $W = M/k_2 l$. Substituting this in the last equation for deflection, we have:

$$f = k_1 \cdot \frac{M}{k_2 l} \cdot \frac{l^3}{EI} = \frac{k_1}{k_2} l^2 \left[\frac{M}{EI}\right] \quad \ldots(4)$$

Comparing equation (4) with equation (3) it is seen that $k = k_1/k_2$. To illustrate, consider the deflection at the center of a freely supported beam with a single concentrated load at the center; that is, the deflection of

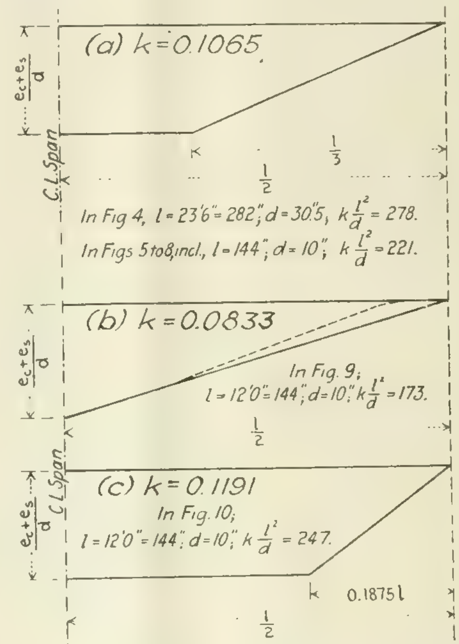


Fig. 3. Variation of $\frac{e_c + e_s}{d}$ Values Between Supports and Center Line of Span; (a) Third-Point Loading; (b) Center Loading; (c) Loads at 0.1875*l* from Ends of Span.

the support with respect to the tangent to the elastic curve at the center. The coefficient k_1 is 1/48, and k_2 is 1/4. Therefore, the coefficient k in equation (3) will become 1/12 (= 0.0833; see Fig. 3 (b)).

The maximum deflection of a simple beam is equal to the deflection of the support from the *horizontal* tangent to the elastic curve. Usually the maximum deflection occurs at the

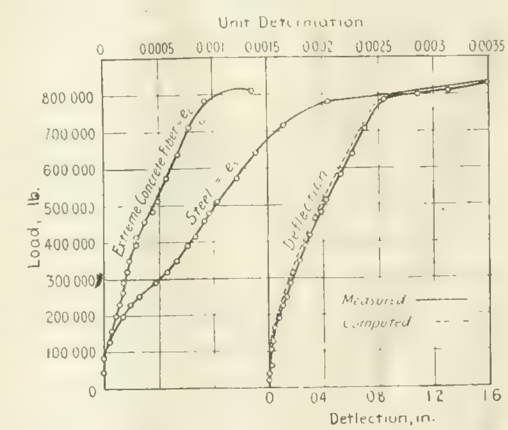


Fig. 4. Deformation and Deflection Curves of Reinforced Concrete Beam; 1:2:5 Concrete; 1.26 Per Cent Steel; Square Corrugated Bars; and Third-Point Loading.

center of the beam, as in all cases of symmetrical loading. With irregular loading it usually occurs near the center of the beam. Then, by the property of the M/EI curve above referred to, we may express this deflection as statical moment about the support of the area under the M/EI curve between the point at or near the center of the span, where the slope is zero, and the support. If this statical moment is expressed in terms of the maximum ordinate of the M/EI curve and the whole span an expression is obtained of the form of equation (3), in which the coefficient k will equal k_1/k_2.

It may be of interest here to point out that M/EI equals the rate of change of the slope of the elastic curve, or the change of slope per unit distance. This may be seen by dividing equation (1) by dS, which gives $d\Theta/dS = M/EI$.

DEFLECTION OF REINFORCED CONCRETE BEAMS.

The application of this principle to the deflection of reinforced concrete beams consists in finding a convenient expression for the value of M/EI.

Let e_c and c_c be the unit deformation in the extreme fiber and the distance from the extreme fiber to the neutral axis, respectively, for the concrete, and let e_s and c_s be the same quantities for the steel. In a homogeneous beam, as above indicated, $e/c = M/EI = d\Theta$, and is the same for both extreme fibers. In this case, since the values of EI are constant throughout the length of the beam, e/c varies directly as M throughout the span.

In a reinforced-concrete beam (see Fig. 2),

$$\frac{M}{EI} = d\Theta = \frac{e_c}{c_c} = \frac{e_s}{c_s} = \frac{e_c + e_s}{d} \quad (5)$$

Substituting this value of M/EI in equation (3), the following expression for the deflection of a reinforced concrete beam is obtained:

$$f = k\frac{l^2}{d}(e_c + e_s) \quad (6)$$

This is based on the usual assumption that a plane before bending remains a plane after bending, which seems justifiable from the evidence of reliable tests herein referred to.

From the preceding analysis it is evident that the deformations of the extreme fibers are the only determining factors in the deflection, except the span, depth of beam, and load distribution—these latter all being known for any given conditions. It is also evident that the distribution of the stresses in the steel and concrete over the section have no influence on the deflection, except in so far as they influence the deformations of the extreme fibers.

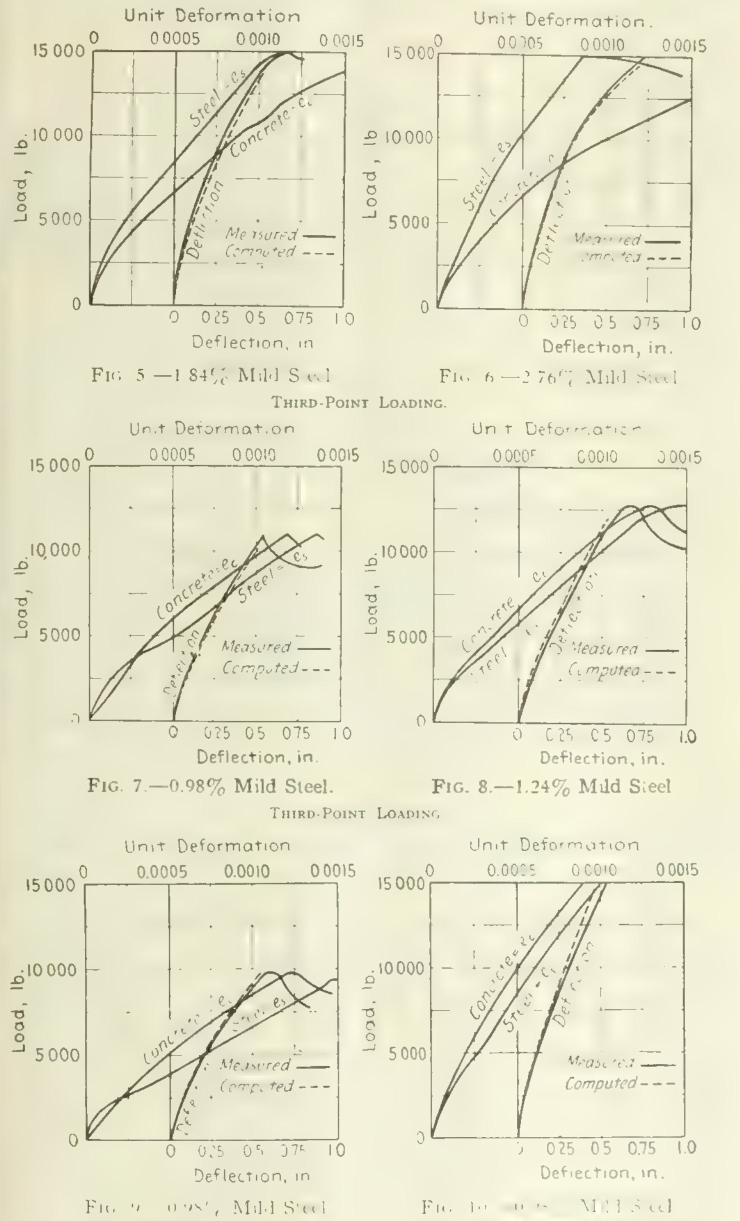
The influence of tension in the concrete might well be discussed here. We know from the principles of equilibrium (see Fig. 2) that the effect of the tension in the concrete at low stresses is to reduce to some extent the compressive stresses in the concrete and the tensile stresses in the steel. This means that we might expect a stiffer beam in the earlier stages of the loading, as the steeper slope of deflection and deformation curves show.

For the reason, therefore, that tension exists in the concrete and that usually near the end of the beam the rods are bent up, we know that the value of the $(e_c + e_s)/d$ curve, with which we deal to obtain the deflection, will not have the same variation as the values of M.

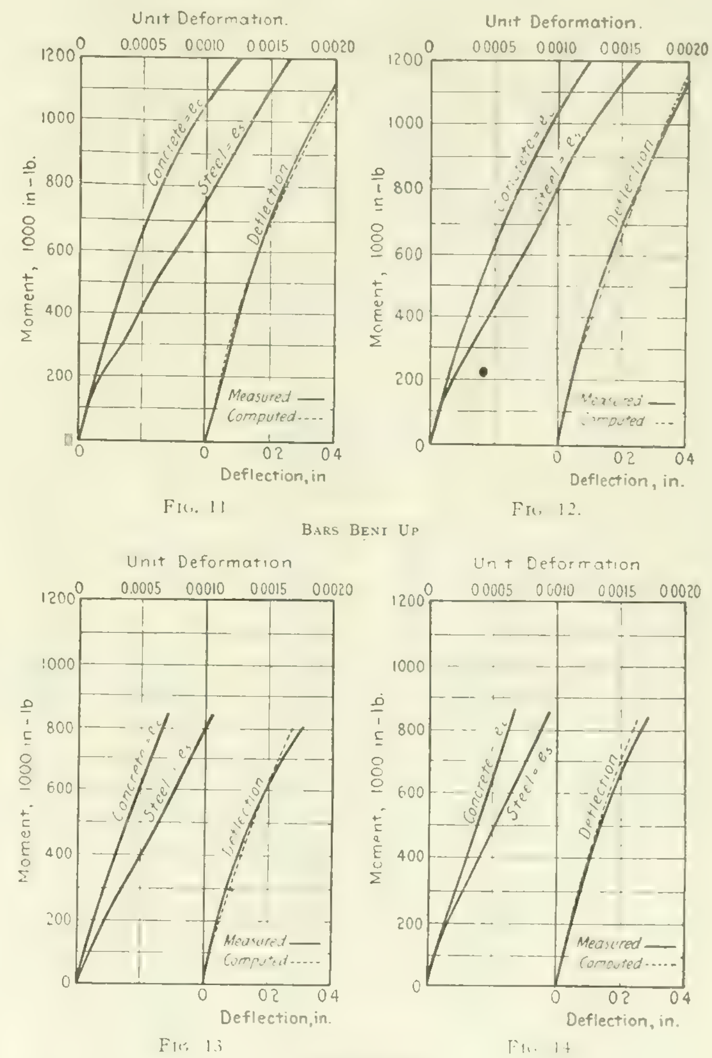
Near the supports of a simple beam and at all points where the bending moment is small we would expect considerably smaller values of $(e_c + e_s)/d$ than the value of M at such a point, relative to the value of M at the point of maximum moment would indicate. The values of $(e_c + e_s)/d$ would probably follow the dotted line indicated in Fig. 3 (b) because of the tension in the concrete. A glance at Fig. 3 (b) will show, and computations will prove, that a small decrease in values of $(e_c + e_s)/d$ ($= d\Theta$) near the point about which moments are taken (the support, in this case), changes the values of this moment only slightly.

EXAMPLES SHOWING AGREEMENT BETWEEN COMPUTED DEFLECTIONS AND THOSE FOUND FROM TESTS.

In all the curves shown in Figs. 4 to 14, inclusive, which represent tests on beams made by the Engineering Experiment Stations of the Universities of Illinois and Wisconsin, deformations in the concrete and the steel were measured at the point of maximum bending moment only, which is at the middle for center loading and between loads where they



Figs. 5-10. Deformation and Deflection Curves of Reinforced Concrete Beams; 1:3:6 Concrete; Percentage of Steel and Loading as Noted.



Figs. 11-14. Deformation and Deflection Curves of Reinforced Concrete T-Beams; 1:2:4 Concrete; 1.04 Per Cent Steel; and Third-Point Loading.

symmetrically applied. When the sum of the extreme fiber deformations in the concrete and the steel at this point of maximum deformation were multiplied by the constant indicated in Fig. 3 for their respective conditions of span, depth and load-distribution, a deflection curve which is indicated by a dotted line in each of Figs. 4 to 14 is obtained, agreeing very closely in every case with the actual measured deflection curve.

This fact indicates that it is proper to make the deflection a function of the deformation in the extreme fibers at the most-stressed section, and that the effect of tension in the concrete and of the change in the moment of inertia due to bent-up bars is negligible. Where such bars are used they are always bent up at points where the bending moment is comparatively small, and the decrease in the moment of inertia of a section due to the bent-up bars would be counterbalanced to some degree by the increase of the moment of inertia due to the larger percentage of tensile stress in the concrete at this section.

Figs. 4 to 10, inclusive, give curves of e_c, e_s, and deflection for some beams tested by A. N. Talbot (Bulletin No. 4, University of Illinois Engineering Experiment Station). They cover widely varying percentages of steel and three different methods of application of load. The areas between the center of the span and the support, of which moments about the support are taken for these three different cases of loading, are indicated in Fig. 3.

When moment-area is expressed in terms of $(e_c + e_s)/d$ and l, we get the constant k, indicated. The uniformly close agreement of the dotted line, which is plotted according to the formula, and of the deflections actually measured is interesting.

Figs. 11 to 14, inclusive, show the results of tests made by M. O. Withey (Bulletin No. 197, University of Wisconsin Engineering Experiment Station). These are all T-beams and have either stirrups of bent-up rods.

Tests of three large beams are described by A. N. Talbot (Bulletin No. 28, University of Illinois Engineering Experiment Station). These beams are 23 ft. 6 ins. long, 6 ft. 3 ins. wide, 30½ ins. in effective depth, and have 1.26 per cent of reinforcement. Many of the rods are bent up, and stirrups are used outside the third points. These beams are larger than most beams used in practice and the details are typical. Deformations in the concrete and the steel were taken on 50-in. gage lengths at the middle, and the beams were loaded at the third points.

The results for one of these beams are shown in Fig. 4. For this beam as for the other two, when the deformations measured in the concrete and in the steel are used to obtain the deflections by substituting them in the formula proposed in this paper, a deflection curve is obtained as shown by the dotted line, which agrees very closely at all loads with the actual measured deflection.

Values of k for maximum deflections under several conditions of loading are here given:

Beam with uniform load:
Ends freely supported, $k = 5/48$ or 0.1041;
Ends fixed, $k = 1/32$ or 0.0313.

Beam loaded at the third points:
Ends freely supported, $k = 23/216$ or 0.1065;
Ends fixed, $k = 5/144$ or 0.0347.

Beam loaded at the middle:
Ends freely supported, $k = 1/12$ or 0.0833;
Ends fixed, $k = 1/24$ or 0.0416.

Effect of Bonus System on Quantity and Unit Cost of Basement Excavation.

(Staff Article.)

It is unquestionably true that most workmen will put forth extra effort when they are certain of receiving monetary returns for extra performance. If adequate supervision is furnished a direct gain may often be made in construction work, and this is especially true where the time element is of greatest importance. The following data show the results of a bonus system as applied by the Aberthaw Construction Co., of Boston, to excavation work. The work consisted of excavating the site of a reinforced concrete building in New Haven, Conn. The building was 62 ft. wide by 400 ft. long, the basement floor being about 10 ft. below the natural grade. The work was done in mid-winter. As the excavated material was to be used in bringing up to grade the depressions in other parts of the lot the contractors decided to use wheel scrapers. In addition to the excavated earth, quantities of sand, of good quality, were taken out and placed in storage piles for use later in concrete. The loam and top soil were first removed by means of plows and frost wedges.

A study of the length of haul and of the number of wheel scraper loads per day showed that on the average 120 loads constituted a full day's work, although for the longer hauls only about 110 loads per day were made. The teamsters, when going about their work in the usual leisurely way and with no incentive for high performance, at best were hauling only 120 to 130 loads per day. The application of the bonus system changed the entire tone of the work from half-hearted endeavor to enthusiastic effort.

In starting payment for extra work each driver who had made 120 loads or more during the day was given a bonus or 50 cts. This bonus was later increased to $1 for each man who made 150 loads during the day—a mark

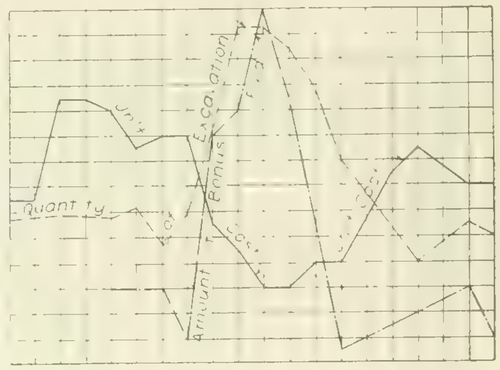


Fig. 1. Curves Showing Variation in Quantity of Excavation, Bonus Paid, and Unit Cost for Basement Excavation in New Haven, Conn.

which several reached. It was expressly stipulated that the horses should not be mistreated and that loads which were not full would not be credited on the tally board. While these instructions were well followed, it is probable that the horses were worked to their limit.

The curves of Fig. 1 show the variation in quantity of excavation, amount of bonus paid, and cost per cubic yard for excavation. No attempt has been made to give actual values as comparative values only were desired. By referring to these curves it is seen that the cost per cubic yard of excavation was lowest when the quantity of excavation and consequently the amount of bonus were highest. It will be noted that increase or decrease in cost per cubic yard of excavation is in inverse ratio to the increase or decrease of quantity of excavation and bonus paid, throughout the length of the curves. The decrease in cost from about 35 cts. per cubic yard at the time the bonus was first applied to 25 cts. per cubic yard and under for the peak of the excavation curve involved a saving of more than $60 per day during the period of maximum excavation.

Standardizing Concrete Construction.—Three cement companies of the northwest states have joined in a movement for the standardizing of concrete construction. These companies have employed a firm of engineers to inspect and report on the quality of workmanship and material furnished cement users, in order that uniformly good results may be obtained.

Structural Features of the Michigan Central Station and Office Building at Detroit, Mich.

(Staff Article.)

II.

In our issue of June 24, 1914, we described and illustrated the principal architectural features and gave a general description of the combined station and office building recently completed at Detroit, Mich., for the Michigan Central R. R. In this issue we shall treat of the structural features of this terminal, giving particular attention to the foundation of the main building.

SUBSTRUCTURE.

Design Features.—The type of foundation adopted for the combined station and office building is unusual for a structure of this magnitude, consisting of a reinorced concrete mat over the entire area. The original design provided for caissons, but the experience of the Detroit River Tunnel Co. in the construction of its tunnels was that the blue clay had a tendency to flow under heavy pressure, while the borings taken at the site showed that the average distance to rock was 110 ft. These conditions, coupled with the fact that there was a possibility of striking a sulphur vein, such as was found in constructing the foundation for the Ford building, led to a very exhaustive series of tests to determine the bearing power of the clay and of several types of concrete piles. After carrying on the tests for some time it was concluded that piles were inadequate and the use of a reinforced concrete mat was decided upon. In some cases the clay would withstand a pressure of 7,000 lbs. per square foot with no appreciable settlement, but its sustaining power was extremely variable. The allowable bearing pressure finally decided upon for the main building was 4,000 lbs. per square foot, and this pressure was also used for the spread footings of the train shed.

The reinforced concrete mat under the high office portion is 42 ins. thick, while under the waiting room and concourse the thickness is 30 ins. The entire mat is composed of 1:2:4 concrete, reinforced in both directions for the bending caused by the pressure due to the flow of the clay. Figure 1 shows a plan of the foundation under the east half of the office building and under a part of the main station. The construction of the west half of the foundation is similar to that shown in Fig. 1. The station and its reinforced concrete mat foundation extends a considerable distance each side of the portion shown in Fig. 1. By referring to the elevations of the top of the mat and of the top of the concrete piers which carry the columns it will be noted that the piers extend a considerable distance above the mat. This was due to the fact that the type of foundation was changed after the material for the columns had been ordered and partially fabricated. It will be noted that combined piers were used for a number of the columns. The lines along which the change in depth of footing occurred are indicated in Fig. 1.

Figure 2 shows cross-sections of the mat and piers taken at three different locations. Figure 2 (a) shows a north-and-south cross-section of the mat taken near the center line of the office building. The change in the thickness of the mat and the arrangement and size of the reinforcement outside of the office building portion are shown in this drawing. Figure 2 (b) shows an east-and-west and a north-and-south section of the portion of the mat at piers Nos. 2 and 3 (for location see Fig. 1). To provide for the column loads and for the flow of the underlying clay the mat is reinforced at both top and bottom, the center of the reinforcement being 6 ins. from the bottom and 4 ins. from the top of the mat. The arrangement and size of the bars are given in the drawing. Figure 2 (c) shows a cross-section of the mat taken through piers Nos. 108, 143 and 83. By referring to this drawing it will be noted that the reinforcement in the bottom of the slab consists of two tiers of bars running in one direction and one tier in the opposite direction, while the reinforcement in the top of the mat consists of one tier in each direction. These drawings show the height to which some of the piers were built above the top of the mat to accommodate the steel work.

In general the piers rest on spread footings which in some cases carry the combined loads of several columns. The heaviest load on any column is about 600 tons. In all cases where the economic depth of footing is less than or is equal to the depth of mat the piers rest directly on the mat, and the reinforcement is so arranged as to spread the load over the required area. Under the high portion of the building, however, the economic depth of footing is considerably greater than the depth of mat, and in these cases the footing projects up through the mat.

Construction Features.—The original intention was to construct the mat as a monolith, but this was found to be impracticable, as the concrete would begin to set in places before it could again be freshened. It was finally decided to pour the mat in sections about 20 ft. wide extending the full distance east and

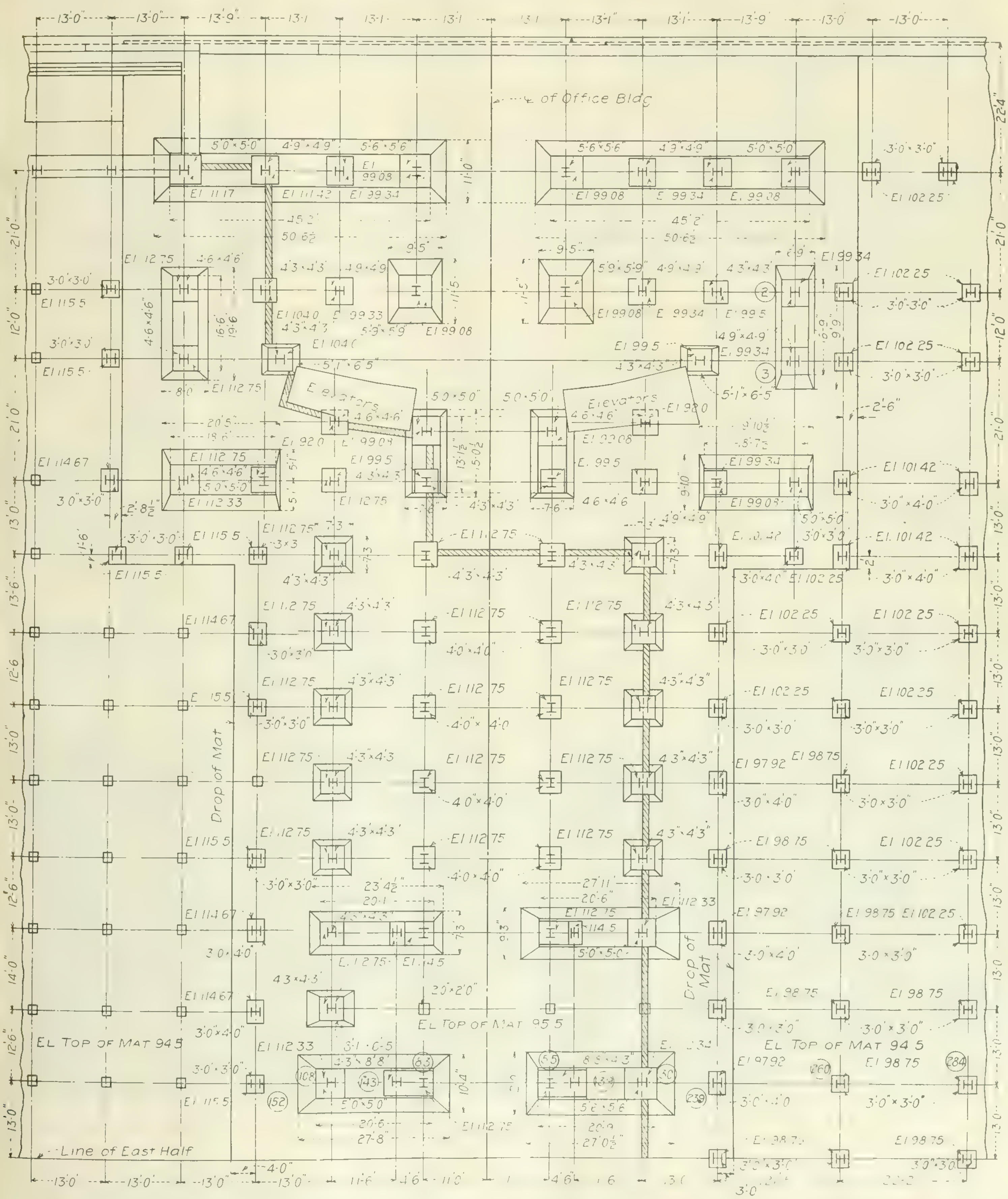


Fig. 1. Foundation Plan of East Half of Michigan Central Station and Office Building, Detroit, Mich.—Plan Shows Portion Under Office Building and Central Part of Station.—Change in Thickness of Concrete Mat Indicated by Heavy Lines.

west. Dowels, of sufficient cross-sectional area to resist the horizontal shear, were provided for each pier or footing.

The entire lot was excavated to the required depth with two steam shovels, this work being

The contractors poured all of the concrete from trestle-work, which rested on 4x6-in. timber posts. Figure 3 shows a view from the north of practically one-half of the site, the arrangement of trestle-work and concrete plant

the reinforcement, but proved to be a very efficient scheme for the handling of the concrete buggies. Two, and sometimes three, "Smith" mixers of ½-cu. yd. capacity were used day and night until the foundation was

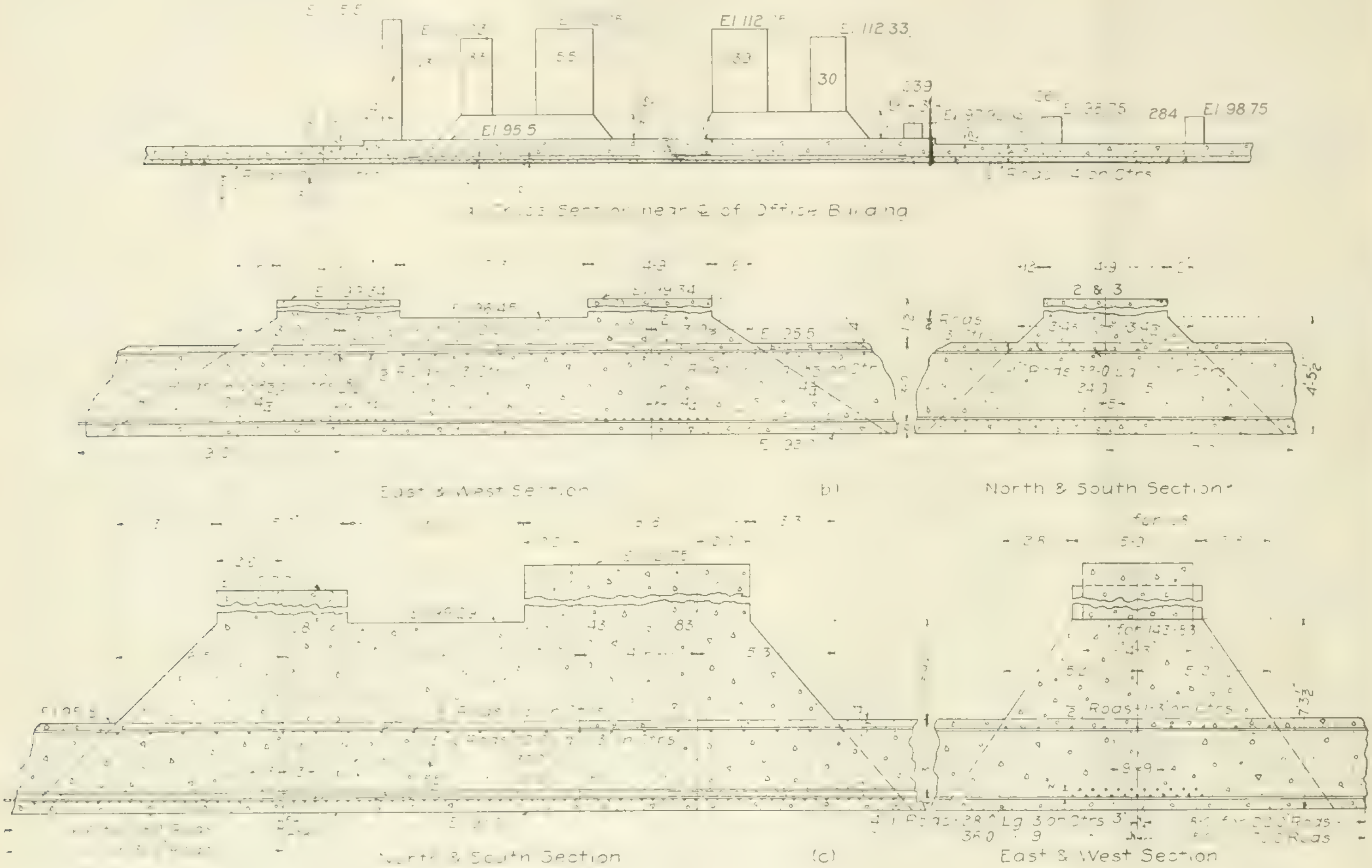


Fig. 2. (a) North-and-South Cross Section of Reinforced Concrete Mat Near Center Line of Office Building; (b) Cross Sections of Mat Mat Through Piers Nos. 2 and 3; (c) Cross-Sections of Mat Through Piers Nos. 108, 143 and 83. (See Fig. 1.)

done by the forces of the Michigan Central. The original design (which provided for caisson foundations) did not contemplate the excavation of the entire lot, but the railroad company did this work in order that the bot-

being duplicated on the other half (not shown in the view). This view clearly shows the arrangement of the trestle-work, the material piles, the concrete plant and concrete buggies used. The steel viaduct, upon which is the

finished. As will be noted by referring to Fig. 3 (which shows one of the mixing plants) the mixers were placed along the south side of the building, at an elevation such that the loaded concrete cars would run gently, by gravity,

Fig. 3. View From North Showing East Half of Site of Detroit Station. Note Constructi on Plant and Concrete Mat—Plant and Type of Construction for West Half Similar to That Shown in This View.

tom of the entire mat would be on the same level. The bottom of the mat is at elevation 92, which is 24 ft. 9 ins. below the waiting room floor. Its bottom surface is unbroken except for the elevator pits.

derrick car, used for unloading materials, is shown in the background. A close inspection of Fig. 3 will disclose details of the reinforced concrete mat and piers. The trestle-work not only gave a substantial means of supporting

over the trestle-work to any part of the work. Two laborers were easily able to return the cars to the mixers. The stone, sand and cement for the entire work were delivered on the tracks of the viaducts at such places as the contractor desired and were stored underneath the viaduct. As the stone and sand were unloaded near the hopper of the mixer, the hauling of these materials in wheelbarrows was reduced to a minimum.

Precise levels have been taken on bench marks established on the concrete piers, but no appreciable settlement has been noted. Some cracks have appeared throughout the building, but a careful investigation has shown them to be temperature cracks.

SUPERSTRUCTURE.

The combined station and office building consists of a lower story having a frontage of 345 ft. and a depth of 266 ft., above which rises an I-shaped office building. The office building, which has a height from the curb to the top of the pent-houses of about 240 ft., is 345 ft. long, with a central stem 54 ft. wide and two end wings about 108 ft. wide by 56 ft. long. (A description of the architectural features of this building is given in our June 24, 1913, issue).

The office building consists of a 11.5-ft. storage story, the floor of which is 34 ft. 3 ins. above the section floor; above this is a 10-ft. pipe loft story, followed by ten 12.5-ft. office stories, two 14-ft. office stories, and one 12-ft. office story. The structure is of steel skeleton construction, with reinforced concrete floors, plastered terra cotta partitions, and brick walls above the storage story, which is the first story above the station proper. The walls below this story are stone.

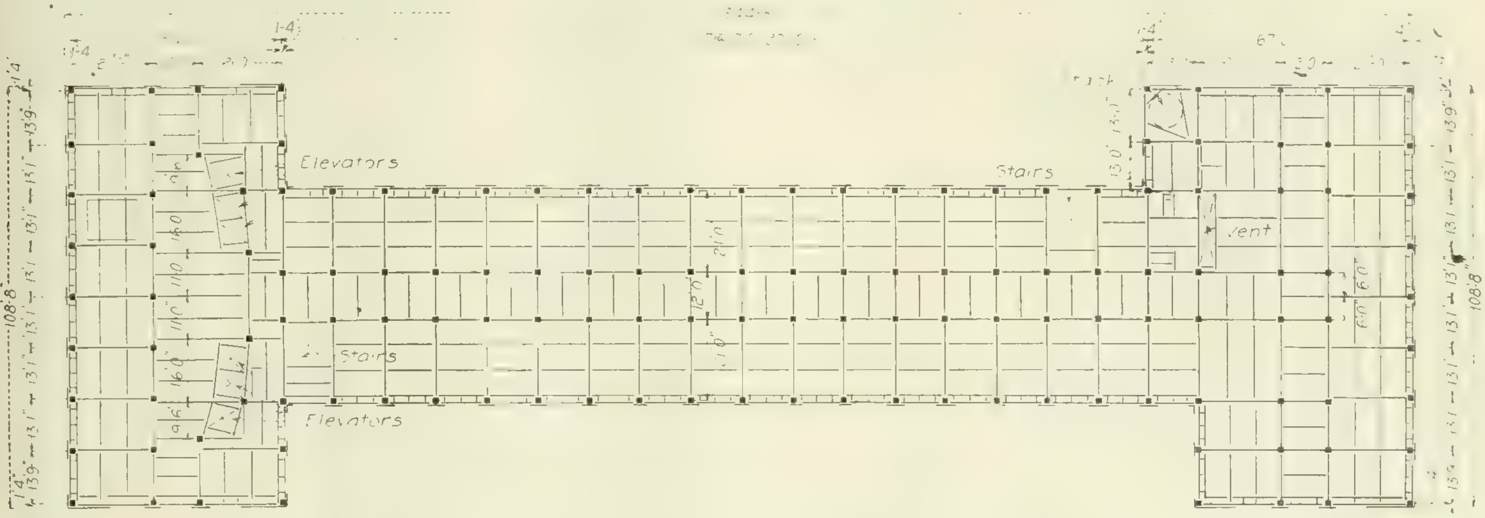


Fig. 4. Framing Plan of Typical Office Floor of Michigan Central Office Building at Detroit, Mich.

Figure 4 shows a typical office floor framing plan. The column spacing and the type of framing used are shown on this drawing. It will be noted that the six elevators for the office building are placed in one end of the building, the stack for the mechanical plant being located near the other end.

Figure 5 shows an erection view, looking east. The heavy steel construction required for the waiting room is shown at the left of the view. This view also shows a part of the steelwork for the office building and indicates the erection procedure. Several of the derricks are shown in place for erecting parts of the structure.

Figure 6 shows a rear elevation of the main building and indicates a part of the erection procedure. The steel framework for the concourse is shown in the foreground. The view also shows the manner of supporting the derricks on the erected steelwork, and indicates the type of cornice framing used.

This terminal contains about 7,000 tons of structural steel, 125,000 cu. ft. of stone, 7,000,000 common bricks, and 1,500,000 face bricks. The foundations and side walls contain about 20,000 cu. yds. of concrete and about 500 tons of reinforcing bars, while the concrete fireproofing and floors contain about 5,000 cu. yds. of concrete.

PERSONNEL.

The terminal building was designed by Reed & Stem and Warren & Wetmore, architects, of New York, under the direction of Mr. George H. Webb, chief engineer, Michigan Central R. R. and The Detroit River Tunnel Co. The steelwork was designed by Balcom & Darrow, engineers, of New York. Mr. W. B. Goddard was engineer in charge of the inspection, assisted by Mr. F. A. Pruitt, to whom we are indebted for some of the data contained in this article. The George A. Fuller Co., of New York, was the general contractors, the steelwork being fabricated by the McClintic Marshall Construction Co., of Pittsburgh.

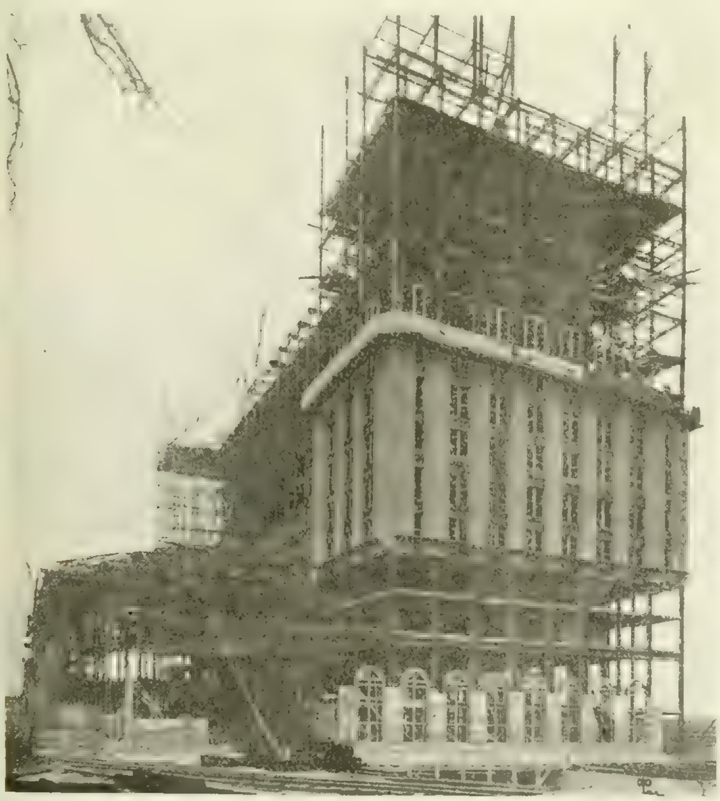


Fig. 5. View Looking East of Detroit Station and Office Building Showing Erection Procedure—Steelwork for Waiting Room Shown at Left.

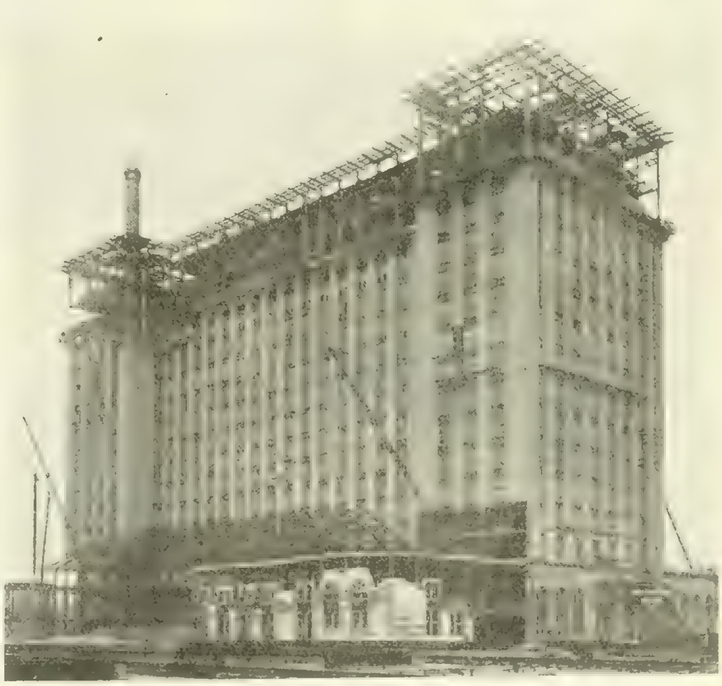


Fig. 6. View of Rear of Detroit Station and Office Building Showing Erection Procedure—Steelwork for Concourse in Foreground.

WATER WORKS

Construction of Water Works Tunnels in the Metropolitan Water District of Massachusetts.

I.

Method and Cost of Constructing Pressure Tunnel in Rock Through Waban Hill in Newton, Mass., and of Laying Large Water Pipes on Section 7 of the Weston Aqueduct Supply Mains.

Contributed by William E. Foss, Assistant to the Chief Engineer, Metropolitan Water and Sewerage Board, Boston, Mass.

(The present article is the first of a series of four articles relating to the construction of water works tunnels in the Metropolitan Water District of Massachusetts. The first three articles contain itemized cost data taken from the inspectors' force account records. These data pertain to the construction of three short water works tunnels as follows:

(1) A pressure tunnel in rock constructed through Waban Hill in Newton, under free air, by contract in 1910 and 1911; (2) A subaqueous 36-in. water pipe tunnel constructed under Chelsea Creek by the pneumatic process by day labor in 1910; (3) An extension, by the pneumatic process and day labor in 1912, of an existing subaqueous 24-in. waterpipe tunnel built under the Mystic River in 1900.

These tunnels were built by the Metropolitan Water and Sewerage Board of Massachusetts in connection with the supplying of water to the Metropolitan Water District which includes the city of Boston and 18 other municipalities located within a circle of ten miles radius with its center at the State House in Boston.

The fourth article will relate to the construction by contract of a subaqueous water pipe tunnel under Chelsea Creek, now under construction by the pneumatic process. This tunnel is similar to the one built under the same stream by day labor in 1910, described in the second article of the series. The location of the four tunnels is shown in Fig. 1. —Editors.)

An expenditure was authorized in 1909 for the construction of a second Weston Aqueduct supply main for the purpose of increasing the amount of water supplied from that aqueduct to the Metropolitan District. The present article relates to the cost of constructing a pressure tunnel and laying large water pipes on Section 7 of this Weston Aqueduct supply main. A profile and sections of the pipe lines and pressure tunnel are shown in Fig 2. This work was carried on under contract No. 314, in 1910 and 1911, and included the construction of 2,042 ft. of 76-in. concrete lined pressure tunnel in rock, the laying of 363 ft. of 80-in. steel pipes in deep cut and lining them with cement mortar, and the laying of 935 ft. of 60-in. cast-iron water pipe. In this article the item numbers used in the specifications are retained.

The work was begun May 24, 1910, and was suspended for the winter on December 31 of that year. It was resumed on April 1 of the following year and completed on Nov. 25, 1911.

Prior to July 1, 1911, the working day included 9 hours, from 7:30 a. m. to 5 p. m. with ½ hour for lunch, with the exception that during the driving of the tunnel the night shift worked from 7:00 p. m. to such time as the drilling was completed, with one hour for lunch. As blasting was not allowed between 6:30 p. m. and 6:30 a. m., the night shift remained on the work until the blasting was completed after 6:30 a. m. The steam plant was operated continuously through the 24 hours, the engineers working 8-hour shifts.

After July 1, 1911, all work was conducted on an 8-hour per day basis. The work of lining the tunnel with concrete was carried on continuously for six days per week. All wages were substantially the same for the 8-hour day as for the 9-hour day.

Steam plants for driving air compressors, for operating the drills, dynamos, for lighting the tunnel, and the engines for operating the stone crushers were installed at both ends of the tunnel, and the work was carried on from both portals. The arrange-

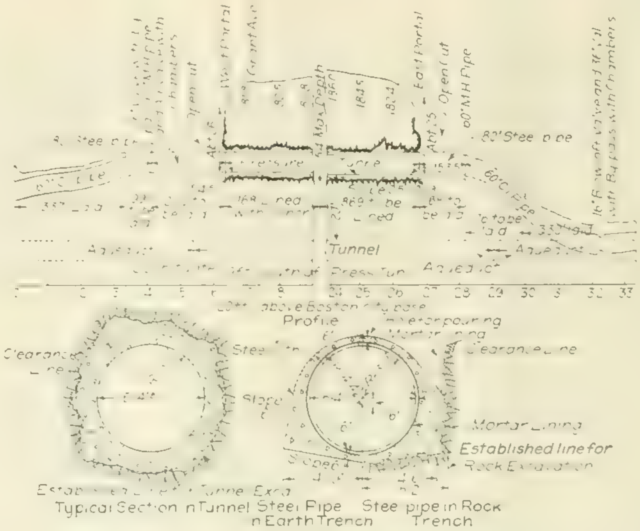


Fig. 2. Profile and Sections of Pipe Lines and Pressure Tunnel of Section 7 of the Weston Aqueduct Supply Mains. (Progress Indicated on Profile is of June 21, 1911.)

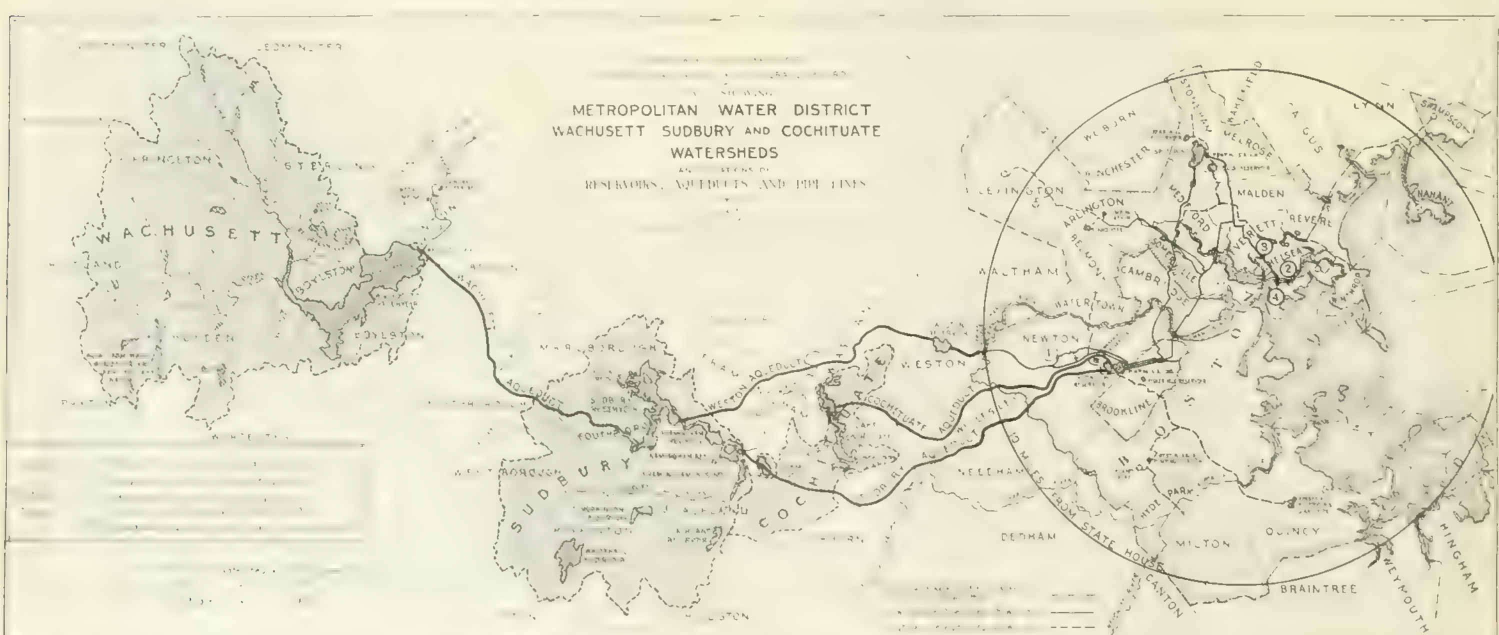


Fig. 1. Map of Metropolitan Water District of Massachusetts Showing Location of Four Water Works Tunnels.

ment of the machinery at both ends of the tunnel is shown on Figs. 3 and 4.

CONSTRUCTION PLANT.

An approximate estimate of the value of the plant when new is as follows:

Plant at East End of Tunnel:	
2 Erie City Iron Works 75 HP. horizontal locomotive type boilers, 54 ins. diameter, 18 ft. long	$1,800.00
1 Buffalo Forge Co. dynamo engine	500.00
1 30 KW. Eddy 120-volt D.C. dynamo, with appurtenances	500.00
1 Rand Drill Co. 110 HP. air compressor	1,650.00
1 Air receiver	45.00
1 No. 3 Austin gyratory stone crusher, conveyor and screens, complete	1,975.00
1 Nagle 55 HP. crusher engine	535.00
1 Blacksmith's outfit, complete	65.00
2 2-in. Canton duplex pumps	120.00
1,200 ft. 2-in. iron pipe	120.00
1,200 ft. ¾-in. iron pipe	40.00
1 wooden water tank, 4 ft. by 5 ft.	25.00
Total cost of plant at East End	$7,375.00
Plant at West End of Tunnel:	
2 Erie City Iron Works 75 HP. horizontal locomotive type boilers, 54 ins. diameter, 18 ft. long	$1,800.00
1 Nagle 32 HP. dynamo engine	500.00
1 110-volt DC. dynamo, with appurtenances	550.00
1 1890 Model Rand Drill Co. 110-HP. air compressor	1,650.00
1 Air receiver	45.00
1 No. 4 Austin gyratory stone crusher, conveyor and screens, complete	2,365.00
1 Buckeye Engine Co. 50 HP. crusher engine	500.00
1 Friction hoist	400.00
250 ft. ¾-in. cable	45.00
1 Blacksmith's outfit, complete	65.00
1 2-in. Canton duplex pump	60.00
2,800 ft. 2-in., 1-in. and ¾-in. iron pipe	270.00
Total cost of plant at West End	$8,250.00
General Plant:	
15 tons steel rails	$ 600.00
1 Smith concrete mixer, ½ cu. yd. capacity	700.00
13 3¼-in. Rand rock drills	2,795.00
6 ¾-in. Chicago Tool Co. jap drills	300.00
9 columns, arms, etc., for rock drills	450.00
3 tripods for rock drills	135.00
1 Tidewater Iron Works Cunniff type grout machine	200.00
42 steel dump cars, ⅔ cu. yd. capacity	2,100.00
10 slip scrapers	70.00
2 single dump carts	150.00
1 2-horse wagon	150.00
1 4-ton differential hoist	76.00
1 large vise	5.10
1 pipe vise	4.00
2 2-in. pipe stocks and dies	7.20
1 lead heating outfit	36.50
1 breaking up plough	24.50
6 wheelbarrows	22.50
1 stiff leg pipe derrick	56.00
Total cost of General Plant	$7,881.80
Total estimated value of entire plant when new	$23,506.80

Including an allowance at the rate of 25 per cent per year on this valuation, for interest and depreciation on the plant during the time that it was in use, the total cost of the work exclusive of the expense of the contractor's Chicago office, his personal traveling and other expenses, and his expenses in connection with the litigation and settlement of the claims made by several property owners in the vicinity of the work for alleged damages from the blasting, is as follows:

Interest and depreciation on plant	$ 7,386.81
Labor and teaming, on payrolls	47,821.56
Materials and expenses, on bills	43,744.74
Expenditures for extra work not included above	504.88
Total expenditures	$99,457.99
Amount of final estimate	114,472.13
Profit	$15,014.14
Profit per cent of expenditures	15.1

WAGES.

The prices paid for labor were as follows:

Superintendent, per month	$200.00
Clerk, per month	100.00
Brick mason, per day	$4.00 and 5.60
Blacksmith, per day	$3.50 and 4.00
Calker, per day	$3.00 and 3.60
Carpenter, per day	4.50
Drill runner, per day	3.50
Drill runner's helper, per day	2.50
Engineer, per day	$3.00 to 5.00
Foreman, per day	$4.00 and 5.00
Sub-foreman, per day	$2.50 and 3.00
Laborers (per day):	
Crusher man, lead man, powder man, stableman and teamster	2.50
Tunnel man	2.25
Ordinary	2.00
Teams (per day):	
4-horse hitch, with driver and helper	12.00
2-horse cart and driver	6.00
1-horse cart with one driver for two carts	3.00
Mules, cost of maintenance at contractor's stable, per day	1.03

MATERIALS AND MISCELLANEOUS EXPENSES.

The prices paid for materials and miscellaneous expenses on bills were as follows:

Accidents, damages, etc.		$ 48.64
Blacksmith and jobbing		579.99
Bond premium ($5 per annum per $1,000 on amount of contract, as shown by canvass of bids)		818.14
Bricks, at $9.50 per M		171.50
Brick mason		49.70
Cement, delivered on work $1.72 per bbl., including cost of bags—actual net cost on work, including teaming, storage, loss of bags and cement damaged, 7,308 bbls. at $1.386		10,129.21
Coal—		
1,226 tons bituminous at $3.83 f. o. b. cars per gross ton	$4,694.02	
152 tons at $4.60 delivered on work per gross ton	698.63	
Blacksmith, 8.69 tons at $4.85	42.12	
		5,434.77
Drills and incidentals		2,394.73
Dynamite—		
40%, 951 lbs. at $0.1225	$ 116.50	
60%, 25,294 lbs. at $0.1625	4,110.29	
Exploders, 2,000 at $3.43 per 100	68.60	
		4,295.39
Express		294.95
Forms for concrete and mortar lining, rental		1,617.00
Grouting machine, rental		18.00
Hay and grain		1,379.33
Insurance		1,486.03
Jute, 400 lbs. at $0.06		24.00
Lead, 1.050 tons at $95.00, 4.772 tons at $97.50		565.10
Livery		65.93
Lumber—Georgia pine at $35.00; hemlock boards at $25.00; spruce, 8-in. and under, at $28.00; spruce, over 8-in., at $30.00		2,516.95
Oil—Castor at $0.35 per gal, f. o. b. Boston; compressor at $0.20 per gal, f. o. b. Boston; cylinder at $0.215 per gal., gasoline at $0.15 per gal; kerosene at $0.13 per gal, machine at $0.185 per gal		502.94
Repairs to plant		1,079.25
Sand—		
coarse, 1,830.4 cu. yds at $0.90 delivered on work	$1,647.36	
Fine, 238.3 cu. yds. at $1.25 delivered on work	298.20	
		1,945.56
Sand blasting steel pipe		701.80
Teaming		1,292.32
Telephone		231.19
Tools, hardware and miscellaneous		4,490.28
Transporting plant		1,183.97
Water		428.07
Total for materials and miscellaneous		$43,744.74

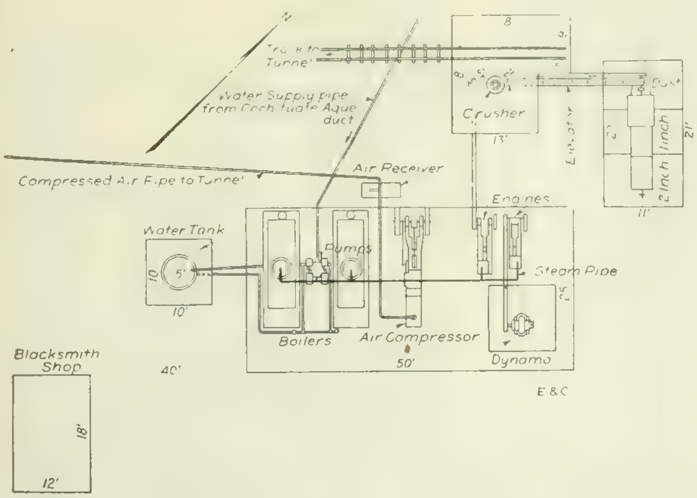


Fig. 3. Sketch of Contractor's Plant Layout at East Portal of Water Tunnel Through Waban Hill, Newton, Mass.

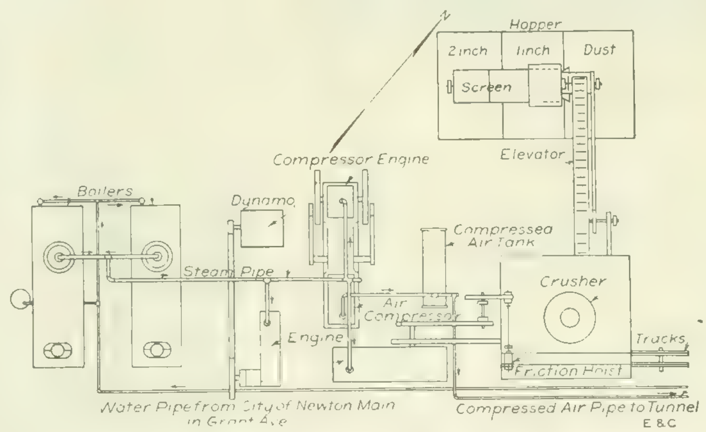


Fig. 4. Sketch of Contractor's Plant at West Portal of Waban Hill Water Tunnel, Newton, Mass.

ITEMIZED AND UNIT CONSTRUCTION COSTS.

The cost of the various items of work under this contract, in detail, was as follows:

Item 1.—Top Soil Excavation. (1412 *cu. yds.*)—Under this item the top soil was excavated from an area of about 0.95 of an acre for an average depth of 11 ins., where other excavations were to be made or embankments were to be built. The material was loosened with plows and transported about 180 ft. to spoil banks with slip scrapers. The cost of the work was as follows:

		Cost per cu. yd.	Per cent of total cost.
Superintendence and general labor	$ 51.88	$0.04	9.2
Labor	292.00	0.21	51.6
Teaming	154.37	0.11	27.3
Small tools, etc.	33.10	0.02	5.9
Incidental expenses and insurance	21.21	0.014	3.7
Plant, interest and depreciation	12.96	0.01	2.3
Total cost	$565.52	$0.404	100.0
Value of work	847.20	0.60	.....
Profit	$281.68	$0.196	49.8

Item 2.—Top Soil Surfacing. (1390 *cu. yds.*)—Under this item an area of about 1 acre was covered with loam from spoil banks to an average depth of 6 ins. at the west end of the tunnel, and at the east end an area of about 0.27 acre was covered to an average depth of 1 ft., and another area of about 0.27 acre was covered to an average depth of 3 ins. The entire work was done with teams. The haul averaged about 210 ft. The cost of the work was as follows:

		Cost per cu. yd.	Per cent of total cost.
Superintendence and general labor	$ 62.36	$0.05	8.4
Labor	351.00	0.25	47.3
Teaming	263.19	0.19	35.4
Small tools, etc	39.76	0.03	5.4
Incidental expenses and insurance	25.50	0.015	3.4
Plant, interest and depreciation	0.83	0.001	0.1
Total cost	$742.64	$0.536	100.0
Value of work	756.80	0.546	.....
Profit	$14.16	$0.01	1.9

Item 3.—Earth Excavation in Open Trench (4184 *cu. yds.*)—Under this item about 480 lin. ft. of trench for the 60-in. pipe line and 350 lin. ft. of trench for 80-in. pipe line was excavated at both ends of the tunnel. The trench for the 60-in. line averaged about 8 ft. in depth, and that for the 80-in. line was made in open cut and varied from 10 to 25 ft. in total depth in

earth and rock, the depth of the earth ranging from 5 to 14 feet. The width of the trench was from 20 to 35 ft. at the top and 10 ft. at the bottom, and no attempt was made to brace the sides, which were allowed to take a natural slope. The earth was loosened with picks and shoveled into cars, which were hauled by mules to the spoil banks. The haul averaged about 350 ft. The earth excavated in the 80-in. pipe trench at the east end of the tunnel was a compact binding gravel; so hard that dynamite was used in loosening it. The large percentage of loss on this item was due to the extremely hard material in the 80-in. pipe trench at the east end of the tunnel, and to the nature of the material in the 60-in. pipe trench at this end which was largely a mixture of stone chips and clay and was hard to excavate and brace. The cost of the work was as follows:

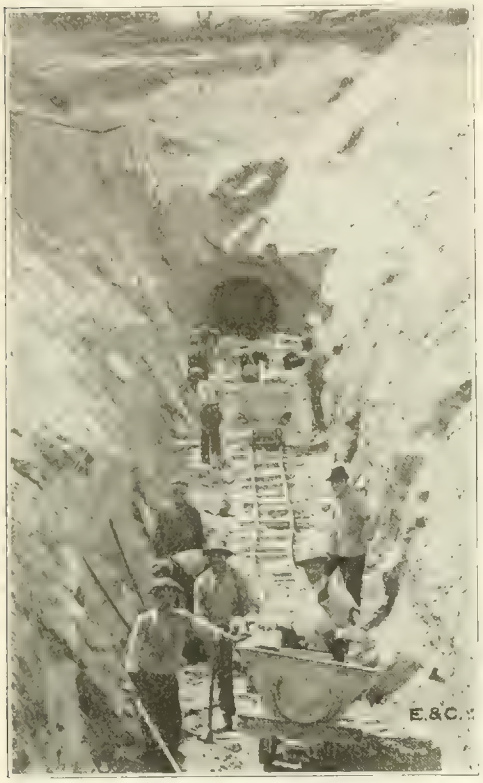


Fig. 5. View of Excavation of Trench for Steel Pipe at East Portal of Waban Hill Tunnel.

Item.		Cost per cu. yd.	Per cent of total cost.
Superintendence and general labor	$ 473.25	$0.11	12.0
Labor	2,663.66	0.64	67.4
Teaming	206.32	0.05	5.2
Lumber for bracing	30.00	0.01	0.8
Small tools, etc.	302.60	0.07	7.7
Incidental expenses and insurance	192.62	0.04	4.8
Plant, interest and depreciation	83.05	0.02	2.1
Total cost	$3,951.50	$0.94	100.0
Value of work	2,180.00	0.52	...
Loss	$1,771.50	$0.42	44.8

Item 4.—Rock Excavation in Open Trench (788 *cu. yds.*).—About two-thirds of the rock excavation under this item was in the deep open cut at the east end of the tunnel, where the rock was extremely hard. A view of this excavation is shown in Fig. 5. On account of the liberal dimensions of the trench and the isolated location, conditions were favorable for excavating the rock cheaply. The rock was loaded on cars and transported by mules to the crusher. The haul averaged about 380 ft. The cost of the work was as follows:

Item.		Cost per cu. yd.	Per cent of total cost.
Superintendence and general labor	$ 154.97	$0.20	5.9
Labor	872.25	1.11	33.4
Teaming	82.53	0.10	3.2
Explosives	150.00	0.19	5.7
Drill incidentals	194.73	0.25	7.5
Small tools, etc.	131.80	0.16	5.0
Incidental expenses and insurance	78.01	0.10	2.9
Plant—			
Transportation, erection, repairs, operation and dismantling	740.45	0.94	28.2
Interest and depreciation	213.7_	0.27	8.2
Total cost	$2,618.46	$3.32	100.0
Value of work	2,574.41	3.26	.....
Loss	$ 44.05	$0.06	1.7

Item 5.—Refilling Open Trenches and Building Embankments (8,569 *cu. yds.*).—In refilling the pipe trenches and building embankments selected fine material thoroughly consolidated with rammers and tamping irons was used for bedding the pipe. The remainder of the material was delivered from the spoil bank in cars and was spread in 6-in. layers. About one-fourth of the material paid for under this item was a sharp sandy gravel from the spoil bank of the surplus material from the 60-in. pipe trench on the adjoining section to the west. The remainder of the material was a clayey gravel excavated from the trenches. The average haul for this work was about 300 ft. The cost of the work was as follows:

Item.		Cost per cu. yd.	Per cent of total cost.
Superintendence and general labor	$ 482.84	$0.05	11.9
Labor	2,717.62	0.33	67.0
Teaming	273.51	0.03	6.7
Small tools, etc.	308.60	0.03	7.6
Incidental expenses and insurance	196.67	0.025	4.8
Plant, interest and depreciation	79.30	0.009	2.0
Total cost	$4,058.54	$0.474	100.0
Value of work	4,757.72	0.554	
Profit	$ 699.18	$0.08	17.2

ft. in diameter, and as the established line for tunnel excavation provided for an excavation 9 ft. in diameter, with a cross-sectional area of 63.62 sq. ft. the actual cross-section exceeded the established section by 17.38 sq. ft., or about 27 per cent. It was provided that the excavation should be trimmed so that the minimum distance from the axis of the tunnel to the rock should be 3 ft. 11 ins., which would leave 9 ins. as a minimum thickness for the concrete lining. Very little trimming was necessary.

Fig. 6. View of Timbering in Waban Hill Water Tunnel.

For a distance of 600 ft. from the easterly portal the tunnel was excavated in hard trap rock. For the remainder of the distance the excavation was in conglomerate with quartzite pebbles varying from 3 or 4 ins. to ¼ in. in diameter. The felsite cement was very hard in some places and extremely soft at other points, where it had changed to kaolin. About 75 ft. from the west portal a seam of clayey gravel was encountered in the roof

Fig. 7. View of Crusher Plant at Westerly End of Waban Hill Water Tunnel.

Item 6.—Tunnel Excavation (2,042.5 *lin. ft.*; 6,125 *cu. yds.*)—The tunnel was excavated in rock for the entire length of 2042.5 lin. ft. The volume of material excavated was 6,125 cu. yds., which is equivalent to an average excavation of 3 cu. yds. per lineal foot. The average cross-sectional area of 81 sq. ft. is equivalent to the area of a circle 10.15

of the tunnel, and it was necessary to support the roof on timbers for a distance of about 26 ft. The scheme of timbering is shown in Fig. 6. Timbering was also necessary at five other points to support the side of the tunnel where the excavation broke through into the loosely backfilled shafts of the old Cochituate Aqueduct tunnel, which is located about 9 ft. south of and 20 ft. below the new tunnel. This old tunnel was constructed by the city of Boston in 1848. At one of the old shafts the entire filling caved into the tunnel and had to be removed. Less drilling and explosives were required for the excavation of the trap rock than for the conglomerate, but it frequently broke wide of the desired line and formed an unnecessarily large section, which delayed the progress of the work because of the increased quantity of material to be moved and the caution required to prevent accidents from falling rocks. The work was carried on at both headings with day and night shifts. From 15 to 21 holes from 5 to 6 ft. in depth were drilled and blasted per shift at each heading. The force usually employed included about 12 men and 1 mule. The progress averaged about 5 ft. per shift at each heading. The drilling was done with Ingersoll-Rand drills mounted on vertical columns and operated by compressed air under a pressure of about 100 lbs. per square inch. The following cost of the tunnel excavation includes the cost of loading the excavated material on cars and transporting it to the crusher or dumps by mules. The haul averaged about 750 ft.

Item.		Cost per lin. ft.	Cost per cu. yd.	Per cent of total cost.
Superintendence and general labor	$2,318.87	$1.13	$0.38	5.6
Labor	13,051.60	6.40	2.13	31.4
Teaming	1,266.03	0.62	0.21	3.0
Explosives	4,144.71	2.03	0.68	10.0
Lumber	108.25	0.05	0.02	0.3
Drill repairs	2,200.00	1.08	0.36	5.3
Small tools, etc.	1,972.00	0.96	0.32	4.8
Incidental expenses and insurance	1,604.39	0.78	0.26	3.8
Plant—				
Transportation, erection, repairs, operation and dismantling	11,079.46	5.43	1.80	26.7
Interest and depreciation	3,780.22	1.85	0.62	9.1
Total cost	$41,525.53	$20.33	$6.78	100.0
Value of work	51,074.40	25.01	8.33	
Profit	$9,548.87	$4.68	$1.55	23.0

Item 7.—Crushing Stone (9,779 *cu. yds.*)—About 80 per cent of all the rock excavated was crushed. At the east portal the rock was delivered directly on the crusher platform and at the west portal the rock was hauled from the tunnel in cars which were left in the open cut at the foot of an incline, up which they were hauled to the crusher platform by a friction hoist operated by the crushing machinery. A view of the crusher plant at the west portal is shown in Fig. 7. At both crushers the product was screened and separated into three sizes, one including stones 2 ins. to ¾ in. in diameter, another stones ¾ to ¼ in. in diameter, and the remaining portion included all materials less than ¼ in. in diameter. At the east portal it was necessary to haul about 70 per cent of the product about 100 ft. to storage piles, and at the west portal the entire product was hauled about 125 ft. to storage piles. Most of the rock was delivered to the crusher in convenient size for crushing and very little hand breaking of material was necessary. The large crusher at the west portal was operated during the day only and crushed the 24-hour output from the tunnel easily, as there were ample storage facilities for the muck at this place. At the east portal, on account of the limited storage facilities and smaller size of the crusher, it was necessary to operate the crusher during both shifts. The cost of the work was as follows:

Item.		Cost per cu. yd.	Per cent of total cost.
Superintendence and general labor	$302.72	$0.03	5.1
Labor	1,703.86	0.17	28.9
Teaming	161.09	0.02	2.7
Small tools, etc.	255.50	0.03	4.3
Incidental expenses and insurance	154.37	0.013	2.7
Plant—			
Transportation, erection, repairs, operation and dismantling	1,446.38	0.15	24.5
Interest and depreciation	1,873.63	0.19	31.8
Total cost	$5,897.55	$0.603	100.0
Value of work	7,334.25	0.75	
Profit	$1,436.70	$0.147	21.4

Item 8.—Portland Cement Concrete Masonry in Tunnel. (2,330 cu. yds. were placed within the line of the established excavation; 144 cu. yds. were placed in old shafts; 1,268 cu. yds. were placed beyond line of established excavation but only 50 per cent of this last amount was estimated for payment, according to terms of the contract; total, 3,742 cu. yds.)

The 3,742 cu. yds. of concrete placed in lining the tunnel is equivalent to an average of 1.83 cu. yds. per linear foot. The concrete was mixed in the proportion of 380 lbs. of Portland cement, 8 cu. ft. of loosely compacted sand, and 15 cu. ft. of loosely compacted mixture of 2-in. and ¾-in. size crushed stone, giving a 1:2.22:4.17 mixture. The concrete was mixed in a steam-driven Smith mixer of ½-cu. yd. capacity, set on the platform at the tunnel so that the concrete was discharged directly into cars which were run through to the point where the lining was being placed. The concrete was dumped upon

Fig. 8. View of Concrete Forms for Lining Waban Hill Water Tunnel.

a temporary floor of steel plates inside of the circular forms which consisted of channel iron ribs spaced 5 ft. on centers, to which the curved side plates were bolted. The concrete was shoveled from the floor into the space between the forms and the rock walls and was thoroughly spaded and churned. Successive side plates were bolted to the ribs as the work progressed, and this portion of the work was completed by filling the key space at the top, the keying plates being 2.5 ft. in length, so that the concrete could be firmly packed. The form for lining with concrete is shown in Fig. 8. One hundred and fifty linear feet of forms were used, and as no inside braces were required cars could be run through them. The forms made the mould for the entire cross-section of the tunnel, except the invert strip which was 2.5 ft. wide. In placing the concrete the bottom layer was put in to within 1 ft. of the invert. The side walls and key were then filled and the 2.5 ft. wide invert was placed later.

In the westerly portion of the tunnel the bottom layer was placed on both sides of the track, which was left supported on a central strip of muck which was later removed, just before placing the invert.

In the easterly portion of the tunnel the track was thrown to one side while the bottom layer of concrete was placed on the opposite side. The track was then shifted on to the concrete already placed and concrete was then placed on the other side. The work was carried on in three 8-hour shifts. Forms were removed and set up in one shift and concrete was placed during the remaining 16 hours. The average progress per 24 hours was about 35 lin. ft. of completed section, except for the 2.5 ft. invert strip which was placed and finished to line with a screed after the track and forms were removed.

The concrete was transported an average distance of 620 ft. and the crushed stone an average distance of 240 ft. from the storage pile to the mixer. The sand and cement were usually delivered to within a short distance of the mixing platform.

The cost of this work has been sub-divided to show the cost of forms separate from the cost of mixing and placing the concrete, as follows:

Item.		Cost per cu. yd.	Per cent of total cost.
Forms:			
Superintendence and general labor	$300.97	$0.08	5.0
Labor	1,694.00	0.45	28.4
Teaming	237.85	0.06	4.0
Lumber	108.00	0.03	1.8
Small tools, etc.	255.70	0.07	4.3
Incidental expenses and insurance	207.35	0.06	3.5
Rental and transportation of forms	1,489.48	0.40	25.0
Plant—			
Transportation, erection, repairs, operation and dismantling	1,438.02	0.38	24.1
Interest and depreciation	229.38	0.06	3.9
Total cost	$5,960.75	$1.59	100.0
Mixing and Placing Concrete:			
Superintendence and general labor	$749.77	$0.20	3.7
Labor	4,220.02	1.13	20.7
Teaming	401.89	0.11	1.9
Sand	1,428.40	0.38	7.0
Cement	7,869.40	2.11	38.6
Small tools, etc.	643.00	0.17	3.1
Incidental expenses and insurance	496.61	0.13	2.4
Plant—			
Transportation, erection, repairs, operation and dismantling	3,601.32	0.96	17.7
Interest and depreciation	1,009.90	0.27	4.9
Total cost	$20,420.31	$5.46	100.0
Total:			
Superintendence and general labor	$1,050.74	$0.28	4.0
Labor	5,914.02	1.58	22.4
Teaming	639.74	0.17	2.4
Lumber	108.00	0.03	0.4
Sand	1,428.40	0.38	5.4
Cement	7,869.40	2.10	29.9
Small tools, etc.	898.70	0.24	3.4
Incidental expenses and insurance	703.96	0.19	2.7
Rental and transportation of forms	1,489.48	0.40	5.6
Plant—			
Transportation, erection, repairs, operation and dismantling	5,039.34	1.35	19.1
Interest and depreciation	1,239.28	0.33	4.7
Total cost	$26,381.06	$7.05	100.0
Value of work	31,568.81	8.44	
Profit	$5,187.75	$1.39	19.7

Item 9.—Portland Cement Concrete Masonry in Open Trench (451 *cu. yds.*).—With the exception of a little concrete used for anchorages and backing on the 60-in. cast-iron pipe line, the concrete masonry placed in open trenches was used for covering the 80-in. steel pipe line. The quantity used for this purpose averaged about 0.84 cu. yd. per linear foot of pipe line. This concrete was mixed in the proportion of 380 lbs. of Portland cement to 10 cu. ft. of loosely compacted sand and 18 cu. ft. of a loosely compacted mixture of 2-in. and ¾-in. size crushed stone, making a 1:2.78:5 mixture.

At the west portal the concrete was hand mixed and as the trench was almost entirely in earth, wooden forms were used for the entire section. At the east portal, where the trench was almost entirely in rock, the concrete was placed up to the springing line of the arch, without forms, wooden forms being used for the remainder of the section. At this place the concrete was mixed in a steam-driven Smith mixer and hauled about 100 ft. Before constructing the forms and placing concrete around the steel pipe, it was braced inside to true circular form, and the outside was cleaned to bright iron with a sand blast and then [illegible] coat of cement paint made by mixing 10 lbs. of cement with 5 lbs. of water. Holes were left in the concrete at the top of the pipe, through which the cement mortar lining was placed later.

The cost of the work has been sub-divided to show the cost of forms separate from the

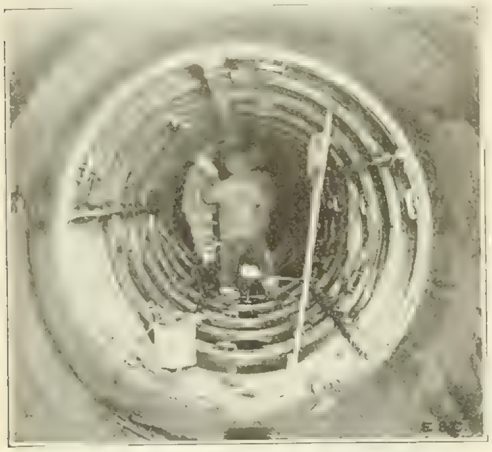


Fig. 9. View of Forms for Placing Mortar Lining of 80 in. Steel Pipe of Weston Aqueduct Supply Main.

cost of mixing and placing the concrete, as follows:

Item.		Cost per cu. yd.	Per cent of total cost.
Forms:			
Superintendence and general labor	$ 51.90	$0.12	10.1
Labor	292.12	0.65	57.0
Teaming	22.63	0.05	4.3
Lumber	91.75	0.20	17.9
Small tools, etc.	33.13	0.07	6.5
Incidental expenses and insurance	21.18	0.04	4.1
Plant, interest and depreciation	0.69	0.001	0.1
Total cost	$ 513.40	$1.131	100.0
Mixing and Placing Concrete:			
Superintendence and general labor	$ 120.06	$0.27	4.6
Labor	675.75	1.50	25.7
Teaming	58.58	0.13	2.2
Sand	239.31	0.53	9.1
Cement	734.70	1.63	28.0
Sand blasting	350.90*	0.78	13.4
Small tools, etc.	90.31	0.20	3.4
Incidental expenses and insurance	55.27	0.12	2.1
Plant—			
Transportation, erection, repairs, operation and dismantling	269.39	0.60	10.2
Interest and depreciation	33.61	0.07	1.3
Total cost	$2,627.88	$5.83	100.0
Total:			
Superintendence and general labor	$ 171.96	$0.39	5.5
Labor	967.87	2.15	30.8
Teaming	81.21	0.18	2.6
Lumber	91.75	0.20	2.9
Sand	239.31	0.53	7.6
Cement	734.70	1.63	23.4
Sand blasting outside of 80-in. pipe	350.90	0.78	11.2
Small tools, etc.	123.44	0.27	3.9
Incidental expenses and insurance	76.45	0.16	2.4
Plant—			
Transportation, erection, repairs, operation and dismantling	269.39	0.60	8.6
Interest and depreciation	34.30	0.07	1.1
Total cost	$3,141.28	$6.96	100.0
Value of work	2,895.82	6.43	
Loss	$ 245.46	$0.53	7.8

*Cost per sq. ft. of surface cleaned = 4.6 cts.

Item 10.—Brick Masonry (36 cu. yds.)—This item included brick masonry used in constructing valve chambers and raising manholes on the Cochituate Aqueduct. The cost of the work was as follows:

Item		Cost per cu. yd.	Per cent of total cost.
Superintendence and general labor	$ 30.25	$0.84	5.7
Labor	220.01	6.12	41.3
Sand, obtained on work	5.16	0.14	1.0
Cement	78.64	2.19	13.8
Bricks	166.54	4.62	31.2
Small tools, etc.	19.17	0.53	3.6
Incidental expenses and insurance	12.27	0.34	2.3
Plant, interest and depreciation	0.40	0.01	0.1
Total cost	$ 532.41	$14.79	100.0
Value of work	561.41	15.59	
Profit	$ 29.00	$0.80	5.4

TABLE I.—COST OF PLACING 2-IN. MORTAR LINING OF 80-IN. STEEL PIPE.

Item.		Cost per lin. ft. of pipe.	Cost per sq. ft. surface covered.	Per cent of total cost.
Forms:				
Superintendence and general labor	$ 47.55	$0.13		7.3
Labor	267.62	0.74		40.8
Teaming	13.30	0.04		2.0
Small tools, etc.	29.85	0.08		4.5
Incidental expenses and insurance	18.96	0.05		2.9
Rental, transportation and repairs of forms	278.10	0.77		42.4
Plant, interest and depreciation	0.63	0.002		0.1
Total cost	$ 656.01	$1.812	$0.086	100.0
Mixing and Pouring Lining:				
Superintendence and general labor	$ 43.24	$0.12		4.2
Labor	243.38	0.67		23.7
Sand blasting	350.90	0.97		34.1
Sand	76.60	0.21		7.5
Cement	267.33	0.74		26.0
Small tools, etc.	27.46	0.07		2.7
Incidental expenses and insurance	17.63	0.05		1.74
Plant, interest and depreciation	0.58	0.002		0.06
Total cost	$1,027.12	$2.832	$0.135	100.00

Total:		Cost per cu. yd.	Cost per lin. ft.	Cost per sq. ft. of surface covered.	Per cent of total cost.
Superintendence and general labor	$ 90.79	$1.97	$0.25		5.4
Labor	511.00	11.10	1.41		30.4
Teaming	13.30	0.29	0.04		0.8
Sand blasting	350.90	7.62	0.97		20.8
Sand	76.60	1.66	0.21		4.5
Cement	267.33	5.80	0.74		15.9
Small tools, etc.	57.31	1.24	0.15		3.4
Incidental expenses and insurance	36.59	0.79	0.10		2.2
Rental, transportation and repair of forms	278.10	6.03	0.77		16.53
Plant, interest and depreciation	1.21	0.03	0.004		0.07
Total cost	$1,683.13	$36.53	$4.644	$0.222	100.00
Value of work	2,098.90	45.50	5.78	0.276	
Profit	$ 415.77	$8.97	$1.136	$0.054	24.7

Item 11.—Cement Grout in Tunnel (292 cu. yds.)—When the concrete tunnel lining was placed, 1½-in. steel pipes with couplings on the outer ends, which were temporarily plugged with wood, were run into all seams and cavities which could not be properly filled with concrete. The spacing of the grout pipes was governed to a large extent by the character of the walls of the tunnel at various points. Extra pipes were placed at each of the old shafts on the Cochituate Aqueduct, and in the section where the roof was supported by timbers in the gravel about 75 ft. from the west portal. Under ordinary conditions the average distance between grout pipes was about 20 ft. The pipes and couplings were furnished by the Commonwealth. The specifications provided that all voids could be filled without forcing the grout more than 10 ft. in any direction, but it was found that the grout actually traveled much greater distances.

The grouting was not begun until after the lining was entirely completed and the concrete had attained considerable strength and most of the shrinkage cracks had developed. It was required that the sand used for grouting should all pass through a sieve having 64 meshes per square inch, and that at least 40 per cent should pass through a sieve having 1,600 meshes per square inch. In making the grout 4 cu. ft. of sand was mixed dry with 380 lbs. of Portland cement. The mixed material was then divided into nine equal parts and put up in bags in which it was transported to the point where grouting was in progress. Three bags of this material and 1.2 cu. ft. of water were used in charging the grout machine. The machine was a tight steel cylinder about 4 ft. high and 18 ins. in diameter, with a 1-in. connection for admitting the compressed air, and a 2½-in. outlet. It was charged through an opening at the top provided with a heavy cover fitted with a rubber gasket. The compressed air for operating the machine was obtained from the plant at the east portal, which at this time it was necessary to keep in operation solely for this purpose. The pressure averaged about 80 lbs. per square inch. The grout was mixed by turning in compressed air at the bottom, which kept the mixture "boiling" and prevented the sand and cement from settling and choking the outlet pipe. About 0.14 cu. yd. of grout was used per linear foot of tunnel. Three hundred and eighty pounds of cement made about 0.9 cu. yd. of grout. The amount of grout required was probably increased about 25 per cent on account of the amount required at the old Cochituate Aqueduct shafts and at the point where timbering was required near the west portal of the tunnel.

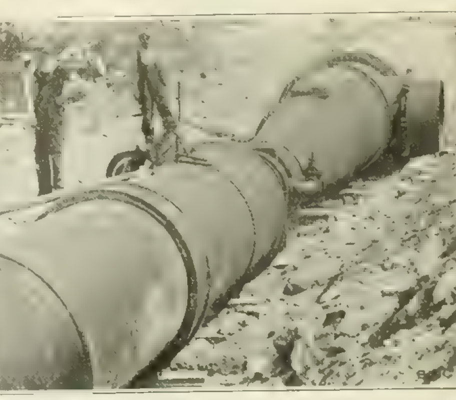


Fig. 10. View of Connection of 60 in. Cast Iron Pipe with 80 in. Steel Pipe, Weston Aqueduct Supply Main.

The cost of the work was as follows:

Item.		Cost per cu. yd.	Cost per lin. ft.	Per cent of total cost.
Superintendence and general labor	$ 173.49	$0.60	$0.09	4.5
Labor	976.50	3.34	0.48	25.2
Teaming	4.00	0.01	0.002	0.1
Sand	221.60	0.76	0.11	5.7
Cement	1,338.45	4.58	0.65	34.6
Small tools, etc.	146.65	0.50	0.07	3.8
Incidental expenses and insurance	86.92	0.30	0.04	2.3
Rental, transportation and repairs of grout machine	26.26	0.09	0.013	0.7
Plant—				
Transportation, erection, repairs, operation and dismantling	826.27	2.83	0.40	21.4
Interest and depreciation	63.39	0.22	0.03	1.7
Total cost	$3,863.53	$13.23	$1.884	100.0
Value of work	3,696.50	12.66	1.810	
Loss	$ 167.03	$0.57	$0.074	4.3

Item 12.—Cement Mortar Lining of 80-In. Steel Pipe (363 *lin. ft.*).—The mortar lining for the 80-in. steel pipe was made 2 ins. thick and was cast in place by pouring a thin mortar into the space between the steel pipe and a central collapsible steel form of the Blaw type, which was held in correct position by means of adjustable bolts, located around the circumference of the form and which were brought to a bearing upon the steel pipe so as to provide the desired 2-in. space for the mortar. The forms were made in sections 7 ft. long, each section consisting of five circular segments bolted together with an adjustable wooden key piece at the top, as shown in Fig. 9. The mortar was poured through 2-in. holes in the top of the pipe, the lining being cast in sections 14 ft. long, without interruption in the flow of mortar after the pouring of a section was once started.

The end of the section was closed by means of a hose extending around the circumference and expanded by means of water pressure, forming a bulkhead at the end of the annular space between the steel pipe and the form.

Before setting up the forms, the interior of the pipe was cleaned to bright iron with a sand blast and it was then painted with a cement wash in the same manner as the outside of the pipe, described under Item 9. The mortar was mixed in the proportion of 1 part of Portland cement, two parts of sand and water amounting to about 25 per cent of the volume of these materials.

On account of the short length of pipe to be lined, the mortar was mixed by hand in barrels supported on a wooden platform above the top of the pipe. One cubic yard of mortar required four barrels of cement, and was sufficient for lining 7.9 ft. of the pipe.

The cost of the work has been sub-divided to show the cost of forms separate from the cost of mixing and pouring the lining, as shown in Table I.

Items 13 and 15.—Laying 60-In. Cast Iron Pipe (935 *lin. ft.*).—The work of laying the 60-in. case iron pipes included the teaming of the pipes about two miles, unloading them from the wagons and laying them in trenches, including the furnishing of all materials required. The excavation and refilling of the trenches was paid for under Items 3, 4 and 5. The cost of the portion of the work included under these items was as follows:

Item.		Cost per lin. ft.	Per cent of total cost.
Superintendence and general labor	$ 215.54	$0.23	8.3
Labor	1,213.13	1.30	47.2
Teaming	358.80	0.38	13.9
Jute	24.00	0.03	0.9
Lead	565.10	0.60	22.0
Blocking and wedges	131.50	0.14	5.1
Small tools, etc.	20.67	0.02	0.8
Incidental expenses and insurance	45.10	0.05	1.7
Plant, interest and depreciation	2.74	0.003	0.1
Total cost	$2,576.58	$2.753	100.0
Value of work	1,973.81	2.113	
Loss	$ 602.77	$0.64	23.4

The cost of teaming the pipe was $0.38 per ton mile.

Item 14.—Laying 80-In. Steel Pipe (363 *lin. ft.*).—The 80-in. steel pipe was delivered to the contractor in sections 20 ft. in length and was hauled by him to the work, a distance of about two miles. It was necessary to roll the pipes on skids for an average distance of 100 ft. from the point of delivery to place them in position. The steel pipes were furnished by the Hodge Boiler Works of East Boston, and were placed, riveted together, lined and covered by the contractor for building the tunnel. Each 20-ft. section of the pipe was made of three alternately large and small courses, each course being formed of a single sheet of flange steel 6 ft. 11 ins. wide and 5/16 in. thick. The longitudinal joints were lapped 4⅜ ins. and double-riveted with ¾-in. rivets spaced 2⅞ ins. from center to center. The circular joints were lapped 2½ ins. and single-riveted with ¾-in. rivets spaced about 2⅜ ins. on centers. At intervals of about 40 ins., pads 6 ins. in diameter and ½ in. in thickness were riveted on top of the pipe, through each of which was drilled and tapped a hole for a 2-in. diameter steel plug. As previously stated these 2-in. holes were used for introducing the Portland cement mortar for lining the steel pipe. At the junction between the 76-in. mortar-lined steel pipes and the 60-in. cast iron pipes, 76x60-in. cast iron branches were set and the 60-in. outlet capped for future use when an additional main shall be required. Figure 10 shows the details of the connection of the steel and cast iron pipe. The cost of the work under Item 14 was as follows:

Item.		Cost per lin. ft.	Per cent of total cost.
Superintendence and general labor	$ 135.83	$0.374	12.2
Labor, laying pipes	390.12	1.08	35.0
Labor, riveting	374.43	1.03	33.6
Teaming	73.53	0.20	6.6
Small tools, etc.	85.00	0.234	7.6
Incidental expenses and insurance	54.45	0.15	4.9
Plant, interest and depreciation	1.78	0.004	0.1
Total cost	$1,115.14	$3.072	100.0
Value of work	1,226.21	3.376	
Profit	$ 111.07	$0.304	9.9

The cost of teaming the pipe was $0.62 per ton mile.

Item 16.—Extra Work.—The extra work required under the contract included the excavation and timbering of six old shafts on the Cochituate Aqueduct tunnel, where the excavation for the new tunnel broke through into these old shafts, and also some miscellaneous work. For this work the contractor received the actual cost of the work plus 15 per cent, and the total amount paid under this item was $925.89.

Results of an Efficiency Investigation of the Aurora, Indiana Water Purification Works.

The water works system of Aurora, Ind., was installed in 1904 and is owned by the Indiana Public Service Co. The supply is drawn from the Ohio River and is subjected to coagulation, sedimentation, filtration and hypochlorite treatment before it is delivered to the consumers. This plant was visited in June, 1911, by Mr. Jay A. Craven, at that time a member of the engineering staff of the Indiana State Board of Health, and samples were taken of the raw, settled and filtered water. Examinations of these samples indicated such a poor efficiency of the filter beds that the installation of hypochlorite treatment was recommended. The inefficient condition of the filters was laid to the irregular feeding of the coagulant.

Early in January, 1914, a rather serious epidemic of typhoid fever developed in Aurora. Examinations of a number of sets of samples taken from the taps of the public water supply disclosed the presence of sewage bacteria in many cases. The bacterial content of the samples indicated something seriously wrong with the purification operations. At the request of W. A. Winn, general manager of the water company, an inspection of the equipment and operation of the plant was made by a representative of the Indiana State Board of Health. The results of the investigation are here described from information in an article by John C. Diggs, assistant engineer, in the monthly bulletin of the board for April, 1914. The investigators' findings indicate the importance of competent technical supervision of the operation of water filtration plants.

During the inspection, washing of the filters, feeding of chemical solutions and general operations of the plant were studied. On March 3, 4 and 5 samples were taken from various sources of the plant and bacterial examinations made. The results of the tests are given in Table I. A laboratory was set up above the office of the water company, thereby permitting an examination of the samples soon after collection.

During the course of the investigation of the plant some minor changes in operation were made at once. Other alterations which could not be made immediately were recommended to be carried out as soon as possible.

Following is a description of the essential features of the works: Two Van Wie centrifugal pumps of 1,000,000 gals. capacity each draw water from an intake extending 200 ft. into the river and elevate it to the two sedimentation basins which have a capacity of 1,000,000 gals. The chemicals, iron sulphate and lime, are here applied. From the sedimentation tanks the water flows by gravity into two New York Continental Jewell filters of 500,000 gals. capacity each. These filters were used at the Louisiana Purchase Exposition at St. Louis and after the close of the fair were moved to Aurora. Leaving the filters, the water flows into a 50,000-gal. clear well of cypress staves. Two Smith-Vallé pumps of 1,000,000 gals. capacity each force the water into a storage tank located on one of the highest hills. This tank has a capacity of 280,000 gals. The hypochlorite solution is added to the suction line drawing the filtered water from the clear well.

One of the first objectionable features to be noted was the irregular feeding of the iron sulphate solution used as a coagulant into the raw water. The iron sulphate was put in solution in two cypress tanks of about 800 gals. capacity each. From these tanks it was pumped into the raw water main leading to the sedimentation tanks. No definite amount of the chemical was put in solution, but a shovelful was added at such times as was deemed necessary in the judgment of the engineer. From the appearance of the coagulated water, the slight turbidity of the filter effluent and the condition of the filter beds upon dropping the water from them, it appeared that for a water as turbid as the raw water was at the time of the visit, an insufficient amount of chemical was used. It was advised that the solution tanks be moved from their position under one of the filters to a separate room from which the solution could be fed by gravity to the raw water mains. The solution should also be made

TABLE I.—BACTERIAL EXAMINATION OF SAMPLES FROM AURORA WATER WORKS, AURORA, IND., IN MARCH, 1914.

Date.	Sample.	Bacteria per c.c. Agar at 37°.	Gelatin at 20°.	Presumptive B. Coli.	Remarks.
March 3	1	450	......	+	Top of filter No. 1, from river.
March 3	2	500	.....	+	Top of filter No. 2, near river.
March 3	3	160	....	+	Effluent, filter No. 1.
March 3	4	1,500	.	+	Effluent, filter No. 2
March 3	5	40	..	+	Tap at pumping station.
March 3	6	100		+	Tap in city.
March 4	7	580	.	+	Top of filter No. 1.
March 4	8	550		+	Top of filter No. 2.
March 4	9	190		+	Effluent of filter No. 1.
March 4	10	200	.	Gas	Effluent of filter No. 2.
March 4	11	85	..	—	Tap at pumping station.
March 4	12	20	..	—	Tap at pumping station, 16 lbs. hypo.
March 5	13	20	8,000	Gas	Tap at pumping station, 14 lbs. hypo.
March 5	14	25	3,000	+	Tap at pumping station, 7 lbs. hypo.
March 5	15	450	16,000	+	Effluent, filter No. 1.
March 5	16	480	18,000	+	Effluent, filter No 2.
March 5	17	200	8,000	—	Tap at pumping station, 9 lbs. hypo.
March 5	18	6,000	150,000	+	Raw water at river's edge.
March 5	19	4,000	40,000	+	Top of filter No. 1
March 5	20	3,000	35,000	+	Top of filter No. 2.

up to a definite strength and this solution fed into the raw water at a rate for proper coagulation. The same condition was found to exist in the case of the lime solution.

For hypochlorite treatment a half per cent solution of chloride of lime was used. This was fed into the mouth of the suction line drawing from the bottom of the clear well cistern through an automatic feed tank. A micrometer screw valve was used for adjusting for different rates of pumpage. This adjusting screw was entirely out of order, making an accurate rate of feed absolutely impossible. The pumpage varies from 6,000 to 30,000 gals. per hour and with such a device for regulating the solution, the application of the chemical at anything like a regular rate was impossible. This, of course, resulted in the addition of an excess amount of hypochlorite solution at times, while at other times an amount too small for proper sterilization was added. To eliminate this objection the adjusting screw was replaced by a ⅛-in. valve. This gave very satisfactory control and by means of tables indicating gallons per hour for each rate of pumpage, satisfactory chemical treatment was obtained.

Insofar as the filtered water was slightly turbid and the bacterial efficiency of the filters decidedly low, it seemed wise that a careful examination of the sand beds be made. An examination disclosed the fact that the filtering materials were so displaced that the filter was of value only in removing the coarser sediment from the water. For the purpose of removal of bacteria it was entirely useless. Gravel was collected in the center of the sand bed extending even above the surface of the surrounding sand. This gravel layer extended to the bottom of the filter and permitted the water to flow freely through it. Near the edge of the filter were similar mounds of gravel. Between the center of the filter and the outer edge was a section which was in very good condition for a filter, but of course, any efficiency that this portion may have had was entirely offset by the condition of the surrounding material. In addition to all of this, a very large part of the sand had been washed out of the beds. Filters of this sort should have 4 ft. to 4½ ft. of filtering material. One filter had 3 ft. and 4 ins. of gravel and sand and the other 2 ft. 6 ins. This fact alone would have been the cause of very poor efficiency.

In an attempt to break up these mounds of gravel and to level the sand stratum over the gravel layer, the filter was washed with the wash water valve open full. A rise of water of 15 ins. per minute was obtained and this pressure, with the rake revolving part of the time, continued for 15 minutes. As this washing continued an attempt was made to force a rod down through the bed, but the mounds of gravel could not be penetrated. The wash water failed to loosen the material and, for the time being, it seemed wise to depend on chemical treatment alone for bacterial reduction.

This condition of the filters may have taken its start from one of three causes; the stoppage of some of the strainer caps preventing the passage of wash water, too rapid revolution of the rake, or the displacement of the sand and gravel layers when the strainer system was cleaned, some two years since. It seemed hardly possible this displacement of gravel was entirely due to a stoppage of the filter heads. This would have resulted in no such regular ridge formation as existed in this case. It seemed most likely that the condition can be attributed to the last two causes. When the strainer heads were examined two years ago the sand was thrown back in any fashion to best get at the work, without regard to gravel and sand. When the sand was washed with the rake going at twenty revolutions per minute the hydraulic sorting took place, some strainer heads being buried so deeply that it was impossible to force water through them. Over these places the coarser material gradually accumulated and slowly overspread adjacent strainer heads. By continued rapid raking the gravel was drawn to the center until it stood even higher than the sand immediately surrounding it. It is quite clear that such a bed would be a failure as a filter. It will be necessary that the gravel be placed over the filter heads and that the bed be filled to the proper level with suitable sand.

Rate control valves designed to prevent sudden changes of the height of the water above the filters are so worn and out of repair that they work very ineffectively.

In practice the plant is run at full capacity from six to eight hours per day in which time the storage tank is filled. Continuous running at a regular rate would give far better results as far as filtration and chemical treatment are concerned.

To get the plant in an efficient working condition from a point of economy, as well as water purification, it seemed advisable that it be thoroughly overhauled and remodeled in agreement with the more advanced ideas in water purification engineering. Certain changes about the plant are already under way and several of the recommendations which are made are in agreement with plans already considered. Changes recommended however, will cover only those points which will assist in bettering the water supply and will in no way deal with certain other points which might be covered by tests made by an efficiency engineer. It does, however, seem highly advisable that such an engineer be employed by this plant while planning the alterations.

To get the plant in proper working condition, it is necessary that the filters be overhauled thoroughly, the gravel placed back in its proper position and the layer of sand be brought back to a suitable thickness. In the meantime, it will be necessary that the health of the consumers be safeguarded by treating the water with hypochlorite solution. The rate of 12 lbs. of calcium hypochlorite per 1,000,000 gals. of water appears to be the most suitable rate for feeding the chemical. The coagulant feed must be carefully regulated. Iron sulphate as a coagulant should be replaced by alum as soon as arrangements can be made.

During the investigation 30 samples were collected. Bacterial counts were made on agar at 37°. Tubes of lactose broth were plated and the presence of B. Coli or other gas forming bacteria determined. Judging from the results of this test, it is necessary to treat the water with hypochlorite at the rate of 12 lbs. per 1,000,000 gals. to remove objectionable bacteria.

RIVERS AND HARBORS

Constructing a Submarine Riprap Embankment to Retain Sand Fill.

The project of improving Governor's Island in New York Harbor involved increasing by 103½ acres the area of the island by filling in behind a sea wall built across a shoal southwest of the island at about 2,000 ft. from shore and connected at each end to the island. The inclosing dike was planned to be a masonry sea wall on a riprap embankment foundation. Since the foundation riprap was to be deposited on soft bottom and since it has to retain sand filling the behavior of the embankment is a matter worth recording and the following statement is extracted from an article by Assistant Engineer H. N. Babcock in "Professional Memoirs" for May-June, 1914:

Borings showed a generally soft bottom except close to the old island, and after some deliberation the conclusion was reached that a foundation of riprap sunk to a natural sustaining depth in the mud would be quite as secure as crib work, easier to repair if required, and much less expensive. At that time a riprap wall to retain sand filling had not been tried in this vicinity, if anywhere, and doubts were expressed as to whether the embankment would not run through the wall. The officer in charge was not unduly tied down to precedent, and with his approval, sanctioned by the Chief of Engineers, the design was changed to provide a riprap foundation with masonry wall on top. The plan was successful and has since been adopted, even to the extent of copying the specifications, by the New York Dock Department at Rikers Island and at the new Brooklyn Shore Drive revetment.

The foundation settled during construction an average of about 4 ft.; it has settled somewhat since and in places may not yet have quite reached a permanent level. This is no more than would have occurred with crib work, and perhaps not as much.

The leakage through the wall was very small, roughly estimated at 500 cu. yds. It occurred only where mud lay against the riprap, and there only while the bank of the fill was between low and high water and the mud was stirred up by ripples. It was enough to noticeably stain the water in the immediate vicinity, outside the wall. When the fill was built up to or above high water, the voids in the stone filled up permanently and the leakage ceased.

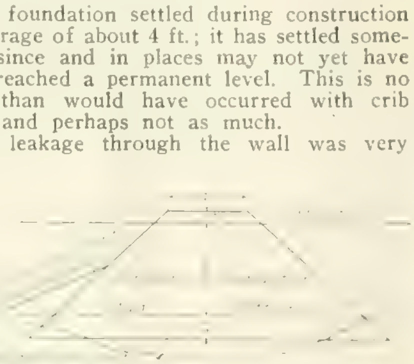


Fig. 1. Section of Riprap Foundation Embankment for Sea Wall.

The wall was built of large and small stones mixed, so as to make it as compact and tight as practicable for stone placed under water.

Riprap foundation was begun in November, 1901, and finished in November, 1904, except at a gap 350 ft. wide left at the outer end to admit scows bringing material for the embankment. The gap was filled in and the foundation finally completed Jan. 31, 1911.

The foundation was planned with side slopes of 1 upon 1. For 2,200 ft. on the Buttermilk Channel side and where exposure to waves was least, the top was 12 ft. wide at 2 ft. above low water. The rest had a top width of 15 ft., 3 ft. above low water. The total amount of stone placed for this work was 404,115 tons (2,240 lbs.). As the stone settled into the bottom additional riprap was put on the outer slope to the amount of 33,496 tons, making a total of 437,611 tons. The length of riprap is 7,207 ft., and the depths in which the riprap was placed ranged from 0 to 26 ft.

While laying the riprap an effort was made to record the amount actually placed in different sections of 200-ft. length, so as to determine the amount of settlement in each section. This proved impracticable. The stone was measured by the barge load and when, as was usually the case, a barge discharged into the different sections, there was no way of estimating how much went to each. In two cases the amounts as recorded would indicate a negative settlement. By dividing the wall into two sections, one on the Buttermilk Channel side extending across the gap and 3,703 ft. long, the other on the Hudson River side, 3,504 ft. long, the inaccuracies of overlapping are reduced to a minimum. The Buttermilk Channel wall averaged 45.0 tons to the linear foot which (at 18 cu. ft. to the ton) would build a wall to contract dimensions in water 20.1 ft. deep. The average depth in this section was 16.6 ft., and the average settlement on top of the masonry wall to date is 0.6 ft., making the average sinking of riprap into the bottom (20.1—16.6 +0.6) 4.1 ft.

The Hudson River wall averaged 61.5 tons, or 1,107 cu. ft. to the linear foot, which would be the estimate for such a wall in water 23.6 ft. deep. The average depth before work was 20.5 ft., and the average settlement on top of the masonry wall is 1.0 ft., indicating an average settlement of riprap (23.6—20.5+1.0) 4.1 ft.

The exact agreement of these two is accidental. It would seem that the Hudson River wall, with its greater weight, would sink deeper, and presumably it did, the exact slope of the sides as built being difficult to determine. But it is obvious that for this structure, or one under similar conditions, the original estimates of the amount of riprap required before it reaches a fixed level should be based upon depths about 4½ ft. greater than actually observed depths. Figure 1 shows an average cross section of the riprap foundation for this sea wall.

The riprap for this work was purchased under three contracts at prices per ton of 2,240 lbs. for stone delivered in place, 35 cts., 58 cts. and 38 cts. The stone was obtained from New York City excavations. The construction of the subway at the time made it possible to get such stone in large quantity and at low prices.

The masonry wall is of granite on a concrete base laid in the top of the riprap. The wall has a coping course 3 ft. wide laid at 10.4 ft. above low water. It was built under three contracts, at prices of $18.75, $20.25 and $21.00 per linear foot. It has settled, with the riprap, an average of 0.8 ft., and at one point 2.4 ft. When it has reached a permanent level—presumably in two or three years—the coping course for nearly one-half its length should be lifted and a leveling course placed underneath.

The entire cost of this sea wall and foundation, exclusive of inspection, was:

Rip-rap, 437,611 tons	$195,233.50
Masonry wall, 7,219 linear feet, including repair of damage by collisions	143,543.20
Total	$338,776.70
Average cost, per foot	47.00

Dock Wall, Gangway and Gangway Bridge Barge Canal Terminals, Troy, N. Y.

(Staff Article.)

The terminals of the Barge Canal on the east bank of the Hudson River at Troy, N. Y., occupy about 960 ft. of water front in two city blocks between Adams, Washington and Liberty streets. Along this waterfront will be constructed a concrete bulkhead wall filled and paved inshore and having at its north end about at Liberty St., a gangway and adjustable gangway bridge. Details of the bulkhead wall and of the gangway construction are illustrated by the accompanying plans redrawn from the official contract plans.

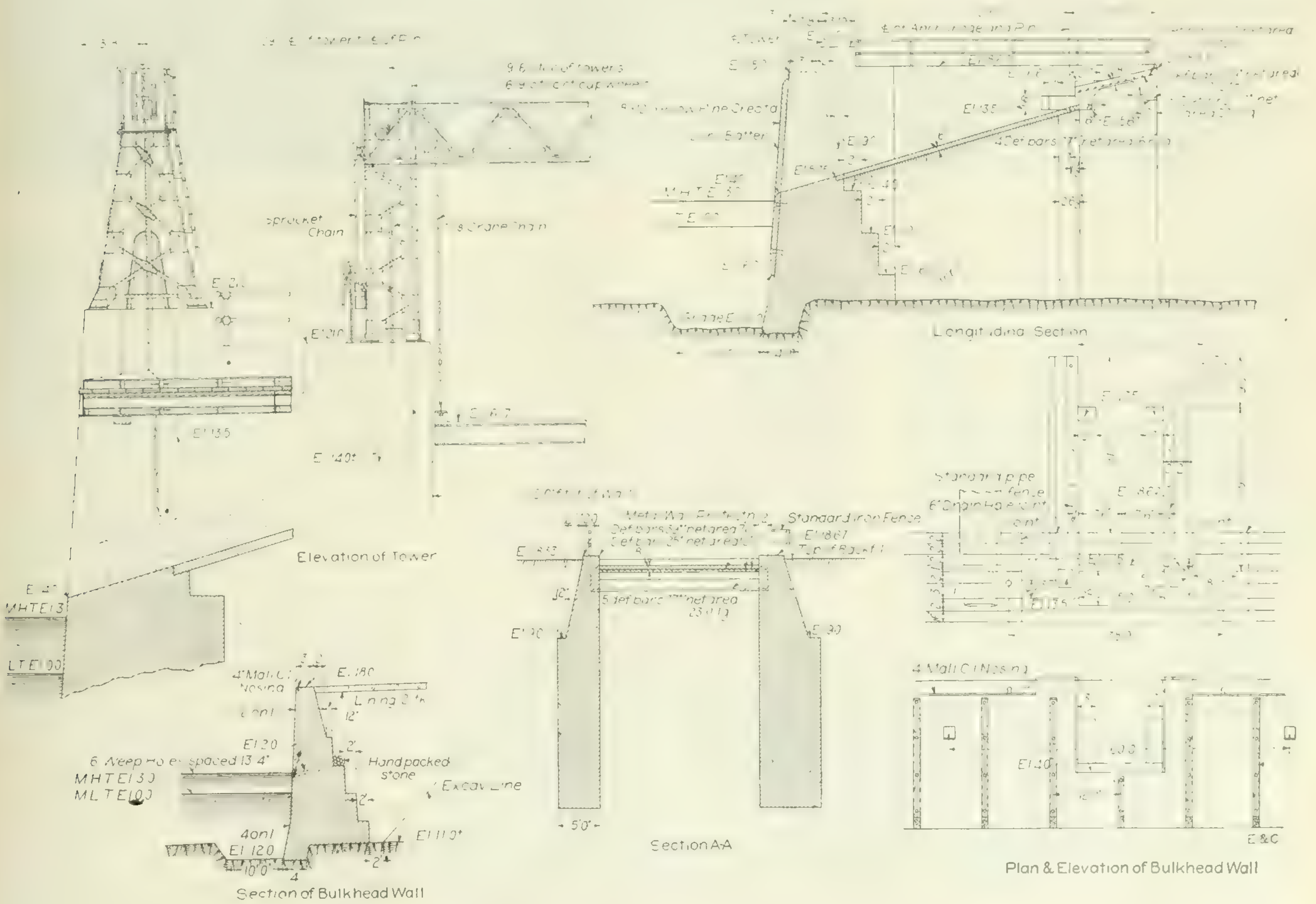


Fig. 1. Wall Section and Gangway Plans, Concrete Dock Wall.

The bulkhead wall is 960 ft. long and is divided by construction joints into 24 sections 40 ft. long. For 800 ft. this wall has snubbing posts spaced 50 ft. apart; for 756 ft. it has line hooks 48 and 50 ft. apart set in niches and seep holes are spaced every 13⅓ ft. The other details of the wall are indicated by the drawings.

Details of the gangway and gangway bridges are indicated by the drawing which also shows clearly the general operation of the bridge.

The general specifications for the concrete work are as follows:

First-class concrete shall be made of 1 part of Portland cement, 2 parts of clean sand and 4 parts of crushed stone. Second-class concrete shall be made of 1 part of Portland cement, 2½ parts of clean sand and 5 parts of crushed stone or gravel. In determining the proportions of the ingredients, 100 lbs. of cement shall be considered as 1 cu. ft.

Stone shall be free, before being crushed, from soil, mud or dust. Soft or shaly stone shall not be used; stone shall be of hard, durable, insoluble stone. Crushed stone for first-class concrete shall be in fragments that will pass through a 1½-in. circular hole, and that will not pass through a ⅛-in. circular hole. Crushed stone for second-class concrete shall be in fragments that will pass through a 2½-in. circular hole, and that will not pass through a ⅛-in. circular hole.

All sand shall be composed of grains varying in size from fine to coarse, not over ⅛ in. in size; it shall be clean, sharp, and shall be screened and washed, if required. Sand which contains not more than 2½ per cent of its volume of silt or loam need not be washed, provided that the total amount of silt or loam in the aggregate of sand and gravel or sand and broken stone does not exceed 3 per cent of the volume of these materials when mixed together in the proportions to be used for the concrete. Sand which contains not more than 10 per cent of its volume of gravel need not have the gravel removed, provided the amount of broken stone or gravel for the concrete be reduced by an amount similar to that contained in the sand.

A wet mixture shall be used, as tending to produce a uniform, dense and impervious concrete; the amount of water shall be such that little or no free water collects on the surface. Concrete upon which concrete is to be deposited shall be thoroughly scrubbed with wire brooms and water from a hose under sufficient pressure thoroughly to remove all laitance, loose and foreign material, this work to be done immediately before depositing new concrete. In joining new concrete to old, or to concrete that has already set, the work already in place shall have its surface cut over thoroughly with picks to remove all inert material. This surface shall then be washed and be scrubbed with wire brooms, and shall be spread over with a thin layer of mortar before the new concrete is placed. In order to bond the successive courses, horizontal keys running lengthwise of the wall at least 12 ins. deep, of a total width of at least one-fourth of the width of the joint, shall be formed at the top of the upper layer of each day's work, and at such other levels as work is interrupted, until the concrete has taken its initial set. Rough stone may, at the discretion of the Engineer, be embedded, instead of using the keys, provided they afford a bonding area equivalent to that specified for the keys.

Whenever concreting is suspended on any section for more than one hour, all edges which will be exposed in the finished work shall be brought to a level and be struck off with a straight edge and a trowel. No concrete shall be slid down a chute or thrown to the place where it is to be laid, except by special permission of the Engineer. In any given layer the separate batches shall follow each other so closely that each one shall be placed and compacted before the preceding one has set, so that there shall be no line of separate between the batches. After the concrete has begun to set, it shall not be walked upon in less than twelve hours.

The Engineer may withhold permission to lay concrete during freezing weather until the work is protected by housing or until the ingredients entering into the composition of the concrete shall be heated so that when the concrete is mixed and ready to be deposited, it shall have a temperature of not less than 75° F. In warm weather, concrete shall be covered with canvas or otherwise protected from the sun, and kept wet until thoroughly set.

The plans as illustrated were designed in the office of the State Engineer, John A. Bensel, Albany, N. Y.

Wave Damage to the Harbor Beach Breakwater, Lake Huron.

A description of the Harbor Beach, Michigan, breakwater, with costs of construction, was published in our issue of Sept. 25, 1912. During the severe storm of Nov. 9-10, 1913, this breakwater, which was exposed to the direct force of a storm from the north, was badly damaged and in an article in "Professional Memoirs" for May-June, 1914, Lieut.-Col. Mason M. Patrick, U. S. Engine Corps, describes the nature of this damage. We take the following extract from Mr. Patrick's article:

The main breakwater is 4,695 ft. long and the superstructure consists of 185 blocks numbered consecutively from the upper to the lower end. The cribs had been leveled off as well as possible and then, as shown in the cross-sections, Fig. 1, there were placed footing blocks which were 10 ft. long, 4.5 ft. wide and 4 ft. high. The outer row was laid as headers, the two interior rows as stretchers and under the joints of the adjacent monolithic blocks were placed cross blocks 4 ft. wide, 4 ft. high and from 7.5 to 11.5 ft. long. All of the footing blocks had joggles or panels molded in their adjacent vertical faces and in their upper faces for binding them to each other and to the mass concrete. This mass concrete in each block amounted to about 200 cu. yds., and is estimated to weigh about 340 short tons.

Blocks 1 to 15, inclusive, suffered little damage. Blocks 16 to 60, inclusive, were all more or less damaged. Most of these blocks are broken at the base of the parapet on its inner side and have settled to the front, the cracks varying in width from about two inches to over a foot. In a number of cases, the concrete of the slope from the parapet to the outer edge of the block has been broken in several places. (See Fig. 2.) On nearly all these blocks there has been much spalling. Blocks 61 to 161, inclusive, show some subsidence and some spalling, but the damage to this portion was not great. Blocks 162 to 172, inclusive, are in about the same condition as blocks 16 to 60, the crack at the inner base of the parapet varying in width from 1 in. to over 20 ins. Along all the portions of the breakwater where the damage is greatest, in many cases the footing blocks on the outer side have disappeared, the rubble filling between the outer row of blocks and the first inner row has been washed out and the mass concrete above has settled to the front.

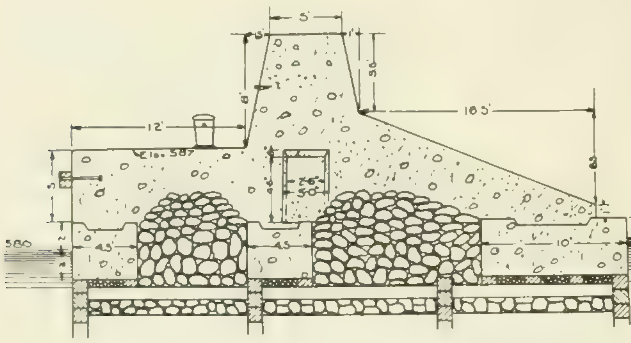


Fig. 1. Cross Section of Concrete Superstructure Harbor Beach Breakwater.

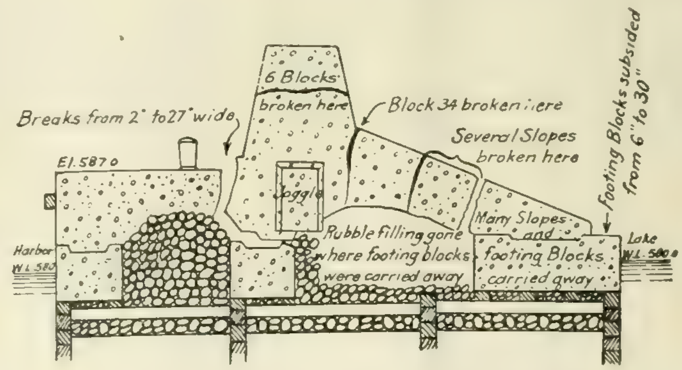


Fig. 2. Typical Section of Harbor Beach Breakwater Showing Damage Done by Waves.

Table of Costs of Reinforced Concrete Dock Structures.

The following tabulation of costs of reinforced concrete docks is taken from a paper by Harrison S. Taft in Proceedings American Society of Civil Engineers, Vol. XL, p. 963:

Location.	Type.	Cost per square foot.
Pier No. 8, Puget Sound Navy Yard.	Concrete columns. Steel deck-beams. Concrete deck-slab	$3.11
Naval Station, Philippines	Concrete columns. Steel deck-beams. Concrete deck-slab.	2.60
Balboa, Panama Canal	Concrete columns. Concrete beams. Concrete deck-slab.	3.28
Oakland, Cal.	Concrete piles. Concrete beams. Concrete deck-slab.	3.27
Brunswick, Ga.	Concrete piles. Wooden deck system.	1.40
Charleston Navy Yard, S. C.	Concrete piles. Wooden decking.	2.60
United Fruit Company, Panama	Concrete-protected wooden piles. Concrete deck-beams. Concrete slab.	2.13
Brooklyn, average of two docks	Wooden piles. Wooden caps. Concrete deck-slab.	0.90

ROADS AND STREETS

Notes on Repaving in Philadelphia.

Repaving problems present many interesting features, since in their consideration errors of judgment in laying the original pavement may be corrected. The 1913 report of the bureau of highways of Philadelphia, Wm. H. Connell, chief, discusses some of these problems.

It is considered necessary to use a comparatively noiseless type of pavement in the vicinity of hospitals and public schools. For this purpose wood block paving was used in 44 different localities within this city. Both the schools and the hospitals, where these pavements have been laid, praise the improvement from the standpoint of the comfort of the patients in the hospitals, and the increased efficiency in school work due to the elimination of noise from the streets.

A portion of Market St. which is subjected to extremely heavy traffic and was formerly paved with old and very rough granite blocks, was repaved with a granite block pavement of modern type. The new type of granite block, Fig. 1, forms a pavement almost as smooth and easily cleaned as asphalt and yet sufficiently durable to withstand the wear of heavy traffic.

Another improvement that has received considerable attention in the business section of the city is the substitution of modern dressed granite blocks for wood blocks, Fig. 2, at the intersections of the street railway tracks. The wood block pavement at these intersections was constantly being repaired, and seldom lasted over six months. Aside from the fact that it is a very severe test on wood block at the intersections of street railway tracks, the track construction in this city is extremely hard on any type of pavement, as the ties are simply laid on the ordinary sub-soil on the bed of the street, which results in so much vibration of the rails that the pavements throughout the city suffer.

Asphalt has been extensively used to replace cobble and rubble pavements and has resulted in an improvement in sanitary conditions. Macadam pavements are treated with some type of bituminous binder. This results in a reduced cost of maintenance, the elimination of the dust nuisance, a reduced cost of sprinkling and an improvement in appearance and smoothness.

The practice adopted in connection with

bituminous surface treatments consisted of applying from ¼ to ½ gal. of bituminous material to the square yard, and then spreading over the surface from 15 to 30 lbs. of fine washed gravel or clean trap rock chips, the quantity and character of materials depending upon the condition of the road, the nature and character of traffic to which it is subjected, grades, and local and social conditions. In each particular instance a careful study is made of these conditions before deciding upon the character of the material to be used. In 1912 none of the macadam roads were in a condition fit for applying bituminous surface treatments until after they had been resurfaced or repaired late in the season, consequently most of the bituminous material used consisted merely of a dust laying material and was applied to roads in poor repair simply for the purpose of laying the dust. In 1913 a more permanent character of bituminous material was used which does away with the dust nuisance and is designed to preserve the roads. During 1913 40.81 miles of roads were treated at an average cost of $0.073 per square yard. Plain macadam resurfacing averaged $0.46 per square yard.

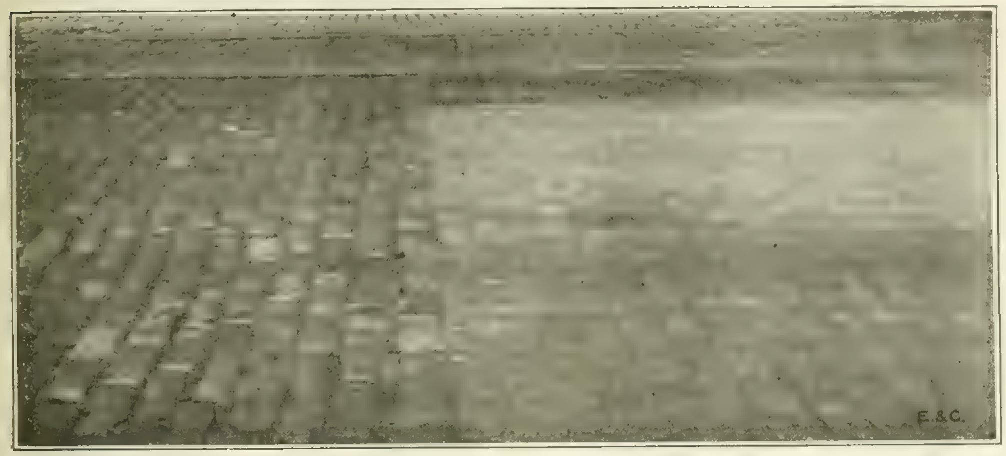


Fig. 1. Showing Difference Between Old Granite Blocks and Modern Dressed Granite Blocks.

Long-Leaf and Short-Leaf Pine and Light Oil Treatment for Wood Blocks.

The properties of long-leaf and short-leaf pine and the danger of using too light an oil treatment for wood paving blocks were discussed by R. E. Beaty in the Proceedings of the American Society of Civil Engineering pp. 730-32.

In European practice it has been customary to use a very light treatment in preparing blocks, that is, to treat them with 10 lbs. of oil per cubic foot, or in some instances merely to dip them into the preservative.

The wood principally used in France is known as "Landes" pine; in England "Baltic deal" is used. An examination of some of these blocks shows very uniform wear, and that the wood very much resembles southern short-leaf, yellow pine, with the difference that the wood is much more nearly uniform in density and strength of fiber.

Tests made by the Borough of Manhattan, and by the Department of Forestry, of the United States Department of Agriculture, show very clearly that although some American pines very much resemble the timber used in France and England, they are of much less uniform quality. The characteristics of southern pines will be considered in more detail later.

In the speaker's opinion, it will be taking a grave risk to try to follow the European practice of using a light treatment of oil. This method will have the effect of leaving some of the blocks in the pavement untreated. This condition is indicated to some extent in foreign countries, where numbers of blocks have to be removed on account of rotting.

All the speaker's observations of experiments and—what is of more value—of the actual condition of 250,000 sq. yds. of blocks after treatment, in charges of from 400 to 500 sq. yds., has convinced him that, as the quantity of oil is decreased below 20 lbs. per cu. ft., it becomes increasingly difficult to get either a uniform distribution of oil between the individual blocks making up the charge, or a satisfactory penetration into all the blocks. This difficulty can be overcome to some slight extent by the slow application of the oil, but this is not commercially desirable, for the reason that lengthening the time of treatment would often curtail the output of plants fully 33 per cent. Attempts at light treatment have been observed many times. In one case, 16,000 sq. yds. were being treated with 16 lbs. per cu. ft. The specific gravity of the oil used was 1.06 at 38° C. At first a 16-lb. treatment was tried, but it was found necessary to increase this to 18 lbs., because many of the blocks, when the lighter treatment was applied, showed a penetration of only ¼ in. from the surface. The inference is, that in injecting 16 lbs. or less into the blocks, the duration of time the pressure is applied to the treatment cylinder is too short to allow the pressure to be raised to a height sufficient to cause oil to penetrate the denser and harder blocks before all the oil is used up. In experiments reported in December, 1912, on various samples tested for the Department of Public Works, Manhattan, it was shown that the quantity of oil taken up per cubic foot by the blocks, under a pressure of 150 lbs. per sq. in., which pressure was necessary to inject an average of 20 lbs., varied widely among the different species of southern yellow pine blocks making up the samples. Some blocks received the equivalent of 28 lbs. per cu. ft., and others received only 12 lbs. At less than 50 lbs. cylinder pressure, 10 lbs. can be injected. If thus treated, many blocks in a charge would receive practically no oil. This lack of uniformity in material, though not so marked in the "Landes" pine and "Baltic fir," indicates a cause for decay in individual blocks.

There is in very general use, by some wood block manufacturers, the term "Southern Yellow Pine" to describe the timber used in manufacturing their product. This nomenclature, as used in the 1913 Proceedings of the Association for Standardizing Paving Specifications, is intended to cover all the five or more varieties of pine found in the South, and it was so stated in the "discussion" at the time of their adoption. That the term is misleading, indefinite, and inaccurate is readily seen by a reference to Bulletin No. 13 of the Division of Forestry, of the United States Department of Agriculture, in which five species of pine, several of them occupying large and definite areas of the Southern States, are given, as follows: "Long-leaf" pine (*"Pinus palustris"*, Miller); "Cuban pine (*"Pinus heterophylla"*); "Short-leaf" pine (*"Pinus Echinata,"* Miller); "Loblolly" pine (*"Pinus taeda,"* Linn); and "Spruce" pine (*"Pinus glabia,"* Walt).

These species vary widely in physical characteristics, such as average percentage of resin, number of annual rings, weight per cubic foot, average percentage of heart wood, strength in compression of fiber parallel to the grain, etc. Two of these definite groups will be compared; the "Long-leaf" and the "Short-leaf" yellow pine. Tests by the U. S. Department of Agriculture showed the following relative values:

	Long-leaf.	Short-leaf.
Strength in cross-breaking, lbs.	10,900	9,230
Strength in compression, lbs. per sq. in.:		
Average lowest	5,650	4,800
Average highest	6,850	5,900

In summarizing the results of tests on various pines the Department expert states: "From these results, though slightly at variance, we are justified in concluding that Cuban and Long-leaf are nearly alike in strength and weight, and excel Loblolly and Short-leaf by about 20 per cent."

These values were checked by experimental tests, at Columbia University, for the Borough of Manhattan. The results of these tests are not given here as the tests were made to demonstrate the value of a requirement for a minimum number of annual rings in timber used in the manufacture of blocks. It is interesting to note, however, that the average strength of three samples of long-leaf was more than 10,000 lbs. per sq. in. in compression, parallel to the fiber, and some samples of short-leaf were as low as 5,600 lbs. per square inch.

Other tests, made at this time by the highway department, indicated clearly that "short-leaf" pine, as compared with "long-leaf," has a greater range of expansion and contraction, and is more difficult to impregnate uniformly with preservative. Each of the species considered has its uses, but, as they often vary widely in price, as well as in physical characteristics (as has been shown), then, for the mutual protection of the bidder and the ultimate user, engineers should specify clearly the kind of timber to be supplied.

Fig. 2. Dressed Granite Blocks Used at Street R. R. Crossings on Streets Paved with Wood Blocks.

It would be necessary to use a certain quantity of oil in order to protect the blocks against bulging or "buckling." The proposition to use untreated timber is also unwise, because it has been observed that, if certain varieties of pine, if untreated, are exposed to the weather, they will be destroyed by fungi and other destructive agents in less than 4 years.

Recent Revisions in the Standards of the New York Highway Commission.

The standards of the New York Highway Commission have been described both in the technical journals and in recent text books. Recent changes in these standards noted in an official organ published by the commission, called the "New York Highway News," are given here.

A cross-section called the general excavation section, Fig. 1a, shows the maximum payment lines for excavation. This section eliminates all uncertainty as to payment for both earth and rock excavation. Figure 1b shows in detail the various shapes of embankment used.

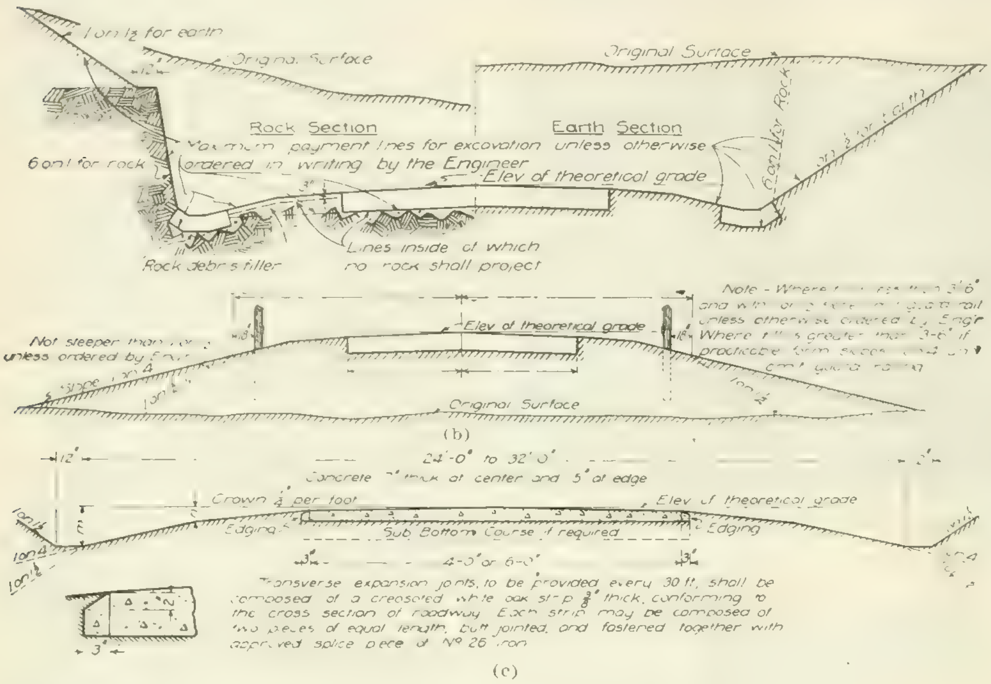


Fig. 1. Modified Cross Sections for New York State Roads.
(a) Payment lines for excavation; (b) Shapes of embankments; (c) Concrete surface.

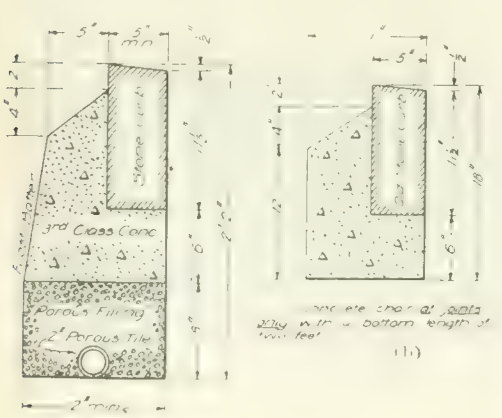


Fig. 2. Stone Curbing.
(a) New type; (b) old type.

New plans for concrete roads, Fig. 1c, call for a flat subgrade and a thickness of concrete of 7 ins. at the center and 5 ins. at the edges. An improved form of edging is shown in the figure. This is an improvement over the old square and sharp edging which caused accidents by the breakage of vehicle wheels when regaining the pavement.

Figure 1c illustrates a sub-bottom course used if required. The sub-bottom course will come in use more extensively in the construction of all types of roads for 1914. Sub-bottom has heretofore been used only on heavy grades and occasionally on soft soil. The proportions formerly used for cement concrete pavement were 1 part cement, 2½ parts fine aggregate and 5 parts crushed stone or selected gravel used as coarse aggregate. This has been changed to 1 part cement, 1½ parts fine aggregate and 3 parts coarse aggregate. This richer mixture has been found more suitable and gives a much more durable type of concrete pavement.

The new specification for stone curbing calls for a concrete saddle, Fig. 2a, which runs the entire length of curbing and a porous underfilling. Figure 2b shows the old type which had a concrete chair at the joints only. Frost batter on the saddle is a new feature.

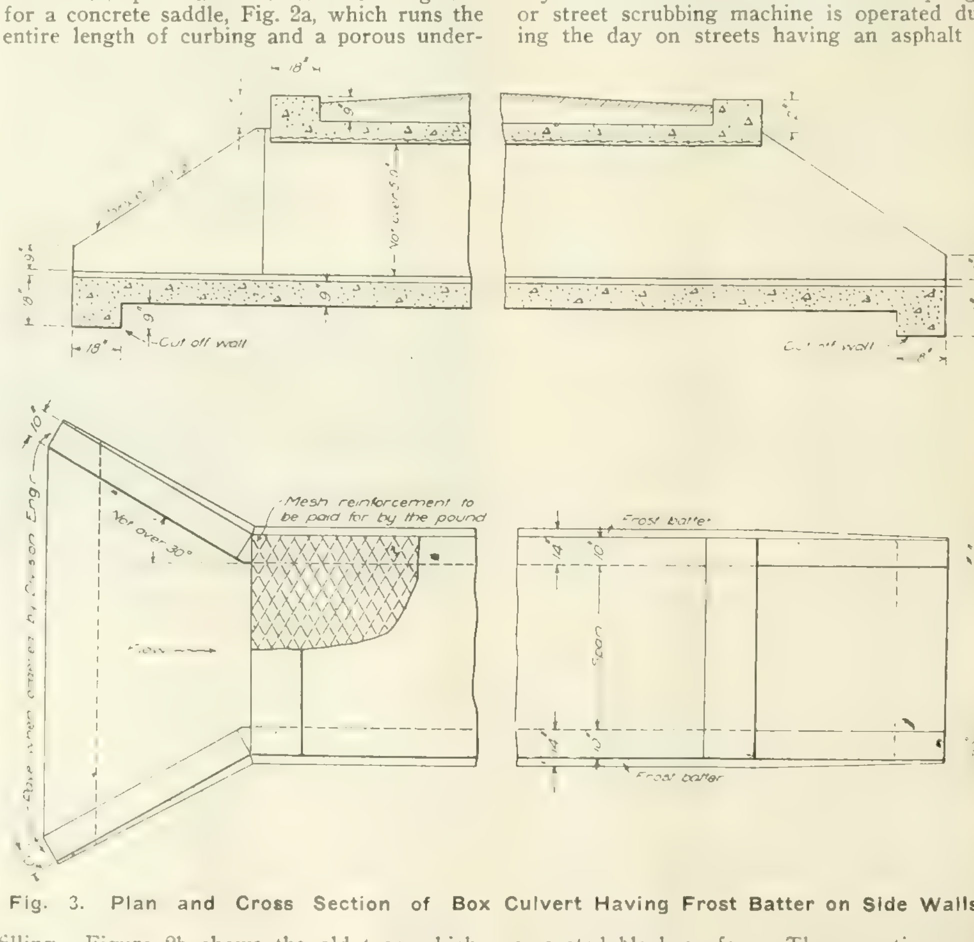


Fig. 3. Plan and Cross Section of Box Culvert Having Frost Batter on Side Walls.

Slight changes have been made in concrete culverts, Fig. 3, namely, frost batter on side and head walls and the placing of cut-off or toe walls on both ends of the culvert bottom. The quantity of concrete used has not been increased. It is expected that this change will tend to prevent cracking and heaving from frost action. Humps or ridges across roads are often due to the lifting of the culvert bodily at that point by frost action.

Figure 4a shows a new and improved type of cast iron grating for leaching basins and catch basins. The former rather complicated design is shown in Fig. 4b.

Figure 5a shows a new type of concrete sign post, which is an improvement over the concrete sign post shown in Fig. 5b. The new post is six sided, making it possible to place guide signs at many angles. Also the six reinforcing rods are placed near the surface where the greatest strain comes. Formerly a single 2-in. pipe in the center of the post, as shown in Fig. 5b, was the only reinforcement and it proved to be inadequate.

Figure 6 shows a new type of catch basins. The new specifications state that catch basins may be built of 1:2½:5 concrete or of acceptable brick at the option of the contractor. The brick construction is shown in the figure.

Cost of Street Cleaning in St. Paul, Minn., in 1913.

The street cleaning service of St. Paul, Minn., states the report of the Commissioner of Public Works, consists of the regular removal from all streets and alleys of vegetable, animal and other refuse; the sweeping and flushing of paved areas and the removal of snow and ice. Paved streets in the business district are flushed every day, weather conditions permitting. This district has a total length of 7.12 miles of streets and an area amounting to 153,400 sq. yds.

Four power flushing machines are operated two shifts a day of eight hours each, the night shift working on business streets and the day shift on other streets. One squeegee or street scrubbing machine is operated during the day on streets having an asphalt or creosoted block surface. The operating crew consists of one foreman for each shift, four or five drivers and two gutter sweepers The power flushers clean 162 blocks in a sixteen hour day or 237,000 sq. yds. per day, and the squeegee 30 blocks in an eight-hour day, or an area of 42,000 sq. yds.

During 1913 flushing was done in every month of the year excepting January and February. The average cost of flushing once per 1,000 sq. yds. was $0.51. In detail the cost was as follows:

Flushing items.	Bills.	Labor.
Gasoline	$2,059.73	
Repair to flusher	2,382.30	
Labor		$8,299.75
Repair to squeegee machine	311.95	
Labor		833.00
Totals	$4,753.98	$9,132.75

Forty-nine miles of paved streets or an area of 1,138,000 sq. yds. have been swept by hand each day. There were employed on the work two foremen, 132 sweepers, 11 shovelers and 11 teams at a cost of $332.39 per day. The average daily collection of street sweepings amounted to 120 cu. yds.; the average daily cost per mile was 6.32 or 27 cts. per 1,000 sq. yds. per day. The detailed costs were as follows:

Hand sweeping items.	Bills.	Labor.
Brooms, etc.	$1,768.28	
Superintendent and asst.		$ 1,485 00
Sweepers		60,990.16
Shovelers		2,244.00
Teams		5,385.10
Totals	$1,768.28	$70,104.26

There are in this city 469 miles of improved streets and 20 miles if improved alleys that receive regular service and 349 miles of unimproved streets and 269 miles of unimproved alleys that receive attention at the spring and fall cleaning up. The total cost of this service, which consists of the cost of removing rubbish, cutting weeds, cleaning macadam streets, flushing paved streets, hand sweeping, paper picking, cleaning alleys and the removal of ice from streets, stairways and sidewalks, and the removal of snow and dirt from bridges and the sanding of walks and bridges, was $30,078.83.

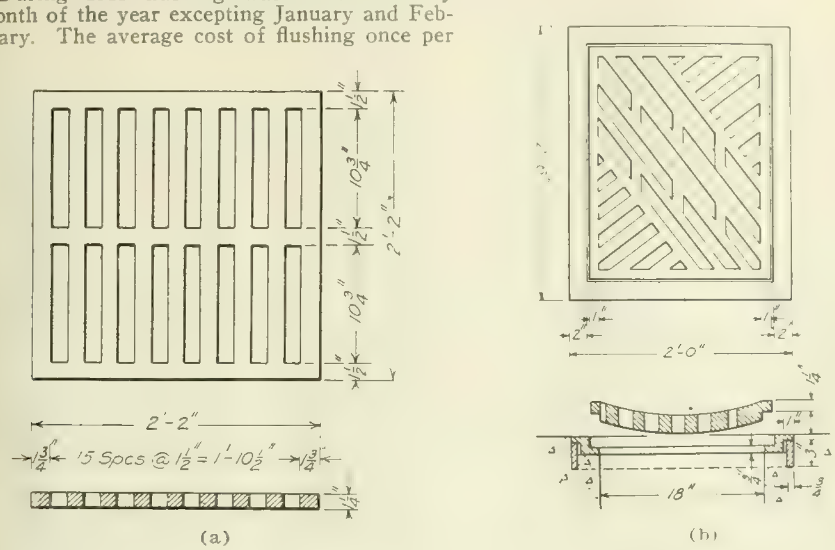


Fig. 4. Gratings for Catch Basins.
(a) Old type; (b) New type.

Bituminous Surfaces for County Roads: Permanence and Desirability.

The question of the ideal surface for country roads is a moot subject and reduced to its last analysis is often a matter of personal preference. The ideas of A. B. Fletcher, state highway engineer of California, an engineer of broad experience in the construction of country roads, as expressed in a letter to a California correspondent, published in an official organ of the commission, are given here:

At the outset I have to confess that I know of no type of pavement which can truly be called "permanent," and the expression must be considered as only relative. I know of no pavement or roadway which does not require from the day it is constructed more or less expenditure for maintenance. If no other work is needed for the first few years the roadsides, gutters and culverts have to be looked after.

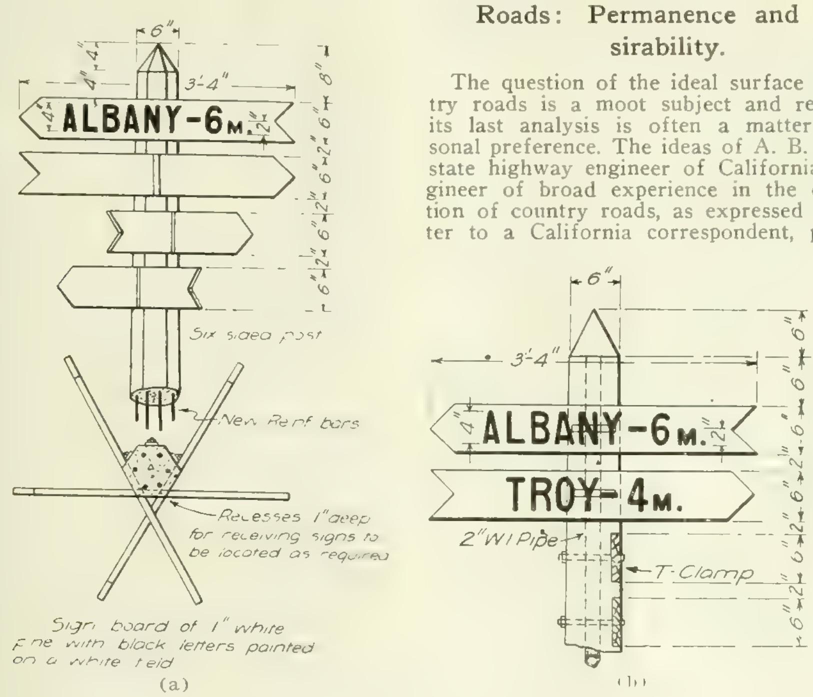


Fig. 5. Concrete Sign Posts.
(a) New hexagonal type; (b) Old type.

As concerns the pavement proper, I believe of the types within the means of the more opulent states and counties at the present time that most nearly approaching the ideal consists of a concrete base of thickness sufficient to properly sustain the traffic which it must carry with a wearing surface of asphaltic concrete from 1½ to 2 ins. thick.

Such a pavement 20 ft. in width and with the base only 4 ins. thick would probably cost under most conditions not less than $14,500 per mile for the paved portion of the road, assuming that the concrete can be laid for $6.25 per cubic yard and the asphaltic concrete surface for 6 cts. per square foot. To this must be added the cost of grading, culverts, drains and any special treatment of the shoulders or beams on the sides of the pavements. Few of the states and hardly any of the counties with which I am familiar can afford so great an expenditure per mile.

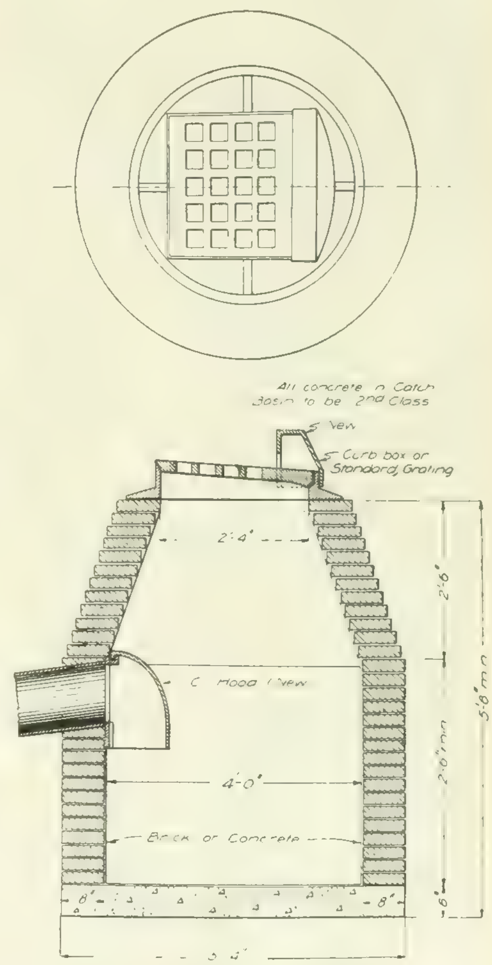


Fig. 6. Catch Basin with Brick Side Walls.

In the instance mentioned a thickness of concrete base of 4 ins. was assumed. In my opinion so thin a base should be adopted only when the subgrade is of excellent quality both as concerns the material composing it, and the rolling done upon it to make it compact and unyielding, and when the inspection of the construction is adequate. Eternal vigilance is necessary to prevent construction of the base thinner than the specifications provide for. If the base be increased in thickness to say 5 ins. one-fourth must be added to its cost or about $2,000 per mile, making a total probable cost of $16,500 per mile.

A shortage of funds lead the California commission to seek a type of road cheaper in first cost, notwithstanding a consequential higher maintenance expenditure during the early years of its existence and the type called "concrete base with bituminous top" was adopted rather generally.

In this method of construction, the base is the same as if the 1½ to 2-in. covering were to be applied, but instead a thin coating of asphaltic oil of special quality is put on to the concrete by spraying machines at the rate of about ½ gal. to the square yard. Clean stone screening or coarse sand are then added in sufficient quantity to absorb the oil. The process requires much care in the selection of the materials used and in their manipulation, but the result is a bituminized coating about ⅜ in. thick. The cost of such surface work ranges from 5 to 10 cts. per square yard, or $600 to $1,200 per mile, roughly, for a 20-ft. pavement, depending on the cost of materials and local conditions. This means that more than 90 per cent of the cost of the work on the road goes into grading, culvert work and the concrete base, all of which may be considered as practically permanent, and the remainder into the thin wearing surface.

Such a wearing surface should last from two to four years before it requires renewal, which renewal should cost considerably less than the original application. The thin surface is best adapted to rubber-tired vehicles, but it wears well under a considerable volume of mixed traffic consisting of both rubber and iron-tired vehicles. I would have no hesitancy in recommending the thin road surface for a road covering as many as from 500 to 600 vehicles a day, provided a considerable portion of the vehicles are rubber tired.

If the concrete is covered with a thick 1½ to 2-in. coating of asphaltic concrete, under ordinary conditions, there will be but little cost for maintenance except on the roadsides and for cleaning culverts for the first five years, but after that period the surface will require attention from time to time in patching and applying seal coats, and when the road needs a new surface, it will doubtless be found that the maintenance cost has been in excess of 2½ cts. per square yard per annum.

Another type of road which the commission has constructed to the extent of nearly 20 miles is the so-called "oil macadam" road. When properly built, I consider this type of work worthy of adoption in many cases, but it has been found, in this state at least, that it is very difficult to secure uniformly good results. There are many contributing causes to the failures or partial failures and the commission has favored the cement concrete type because of the greater certainty of result and because also the concrete so far has cost about the same as "oil macadam."

My experience in the East leads me to believe that the "oil macadam" pavement costs for mainteance not much less per mile per year than the cement concrete base with thin asphaltic top, and this is borne out in the case of the San Joaquin County work in California.

Much work is being done in the cities and to some extent in the country of the type known as "asphaltic concrete." This pavement in its original form was patented and is often called "bitulithic." With a hard, unyielding base a thickness of 4 ins. may be used with good results, but unless the subgrade is nearly perfect, the upper surface of the road is likely to become wavy under the traffic. This sort of pavement costs from 12 to 16 cts. per square foot, $12,500 to $17,000 per mile, 20 ft. wide, for the pavement alone. About 20 miles of such pavement was put down several years ago in San Joaquin County of an average width of 11 ft. I am told that, including the grading, this work cost about $11,000 per mile.

My personal preference is for a concrete base with an asphaltic concrete top from 1½ to 2 ins. thick rather than for the full thickness of asphaltic concrete. In my judgment a "first class all-the-year-round highway" can be constructed to stand the mixed traffic on our county highways and continue of durable character at a reasonable cost for upkeep, but I question if the cost can be kept below $300 per mile per annum on a 20-ft. roadway. Such a road with a concrete base 4 ins. thick and 20 ft. wide covered with the thin coat of asphaltic oil and screenings will probably cost under your conditions not less than $9,000 per mile for the pavement alone.

Notes on Roads and Streets.

Mileage and Average Cost of Roads in Louisiana.—A recent report of the State Highway Commission of Louisiana, W. E. Atkinson, state highway engineer, gives the mileage and average cost of road constructed under the supervision of the highway commission:

STATEMENT SHOWING MILEAGE OF ROADS CONSTRUCTED WITH CONVICT LABOR AND COST PER MILE FROM APRIL 20, 1908, TO APRIL 20, 1914.

Parish.	General character.	Mileage.	Average cost per mile.
[1]Natchitoches	Earth	37.23	$ 779.51
[2]DeSoto	Sand clay	50.39	869.17
[3]Ouachita	4¼ miles graveled, 13 miles earth	17.25	1,454.44
Rapides	Earth	3.57	916.94
[4]East Baton Rouge	Earth	22.22	1,053.13
Orleans	Earth	15.50	1,039.36
City of Baton Rouge	Wide gravel	.82	6,125.71
St. John the Baptist	Earth	8.00	900.00
Total earth roads constructed		99.52	909.45
Total sand-clay roads constructed		50.39	869.17
Total graveled roads constructed		5.07	3,670.32
Total		154.98	

[1]3 miles sand-clay. [2]5 miles "natural gravel." [3]Ouachita Parish—Average cost 4¼ miles graveled road, $3,196.58; average cost 13 miles earth road, $884.90. [4]This item includes the cost of 1.1 miles of drainage canal, costing approximately $3,093.70.

saved by this practice is used for additional grading. About 32 miles of road have been completed and 15 additional miles graded. Gravel and top-soil are used for surfacing. M. H. Stacy is secretary of the road commission and R. T. Brown superintendent of construction.

Kentucky Highway Commission.—State highway commissions are by no means innovations in this country. In 1837 the state highway force of Kentucky was constituted as follows: Chief engineer at a salary of $5,000 per annum; two engineers at a salary each $2,000 per annum; one engineer at a salary of $1,600 per annum; five assistants, salary of each, $1,500 per annum; four assistants, salary of each, $1,100 per annum; making a state highway corps consisting of 13 members.

Cost of Paving in New Bedford, Mass., in 1913.—A recent report of C. F. Lawton, superintendent of streets, states the cost of paving in New Bedford, Mass., in 1913 and former years.

COST OF PAVING IN NEW BEDFORD, MASS., IN 1913.

Type	Sq. yds.	Stone used per sq. yd. tons.	Cost per sq. yd.
[1]New macadam	81,757	0.35	$0.54
Resurfacing macadam	114,920	0.23	0.38
[2]Bitulithic	9,635	....	2.74
[3]Endurite	16,251	....	2.05
[4]Granite block	16,203	....	3.56
Granolithic sidewalk	12,616	....	....
Labor on foundation	......	....	0.75
Concrete	......	....	0.84
Stone curbing per lin. ft.	......	....	1.02
[5]Crushed stone per ton.	73,830	....	1.06

[1]1912, $0.54; 1911, $0.60; 1910, $0.52; 1909, $0.53; 1908, $0.54; 1907, $0.54. [2]1912, $2.53; 1911, $2.44; 1910, $2.96. [3]1912, $1.63 (grading not included). [4]1912, $3.57; 1911, $3.51; 1910, $3.68; 1909, $3.85; 1908, $3.71. [5]Three permanent municipal plants and one portable plant.

Necessity of Bank Protection for Bridge Approaches in California.—In a paper on "County Bridges," presented before a meeting of the division engineers of the California State Highway Commission, Walter C. Howe, engineer for Division V, stated that the lesson taught by floods of last winter was a very valuable one to the counties. Many of the bridge failures were due to faulty pile foundations and consequent settling of piers; others were due to improperly designed abutments, but the principal and most important lesson taught was the urgent necessity for proper protection of the river banks to prevent continued erosion. On every river in this division the erosion of the banks from the 1914 floods is the one big problem, and this problem should be solved before the acceptance of bridges menaced by this danger, for it may easily happen that a 700-ft. bridge accepted today may be an isolated structure tomorrow, and the expense of all the additional spans necessary to bridge this gap made by the flood waters would be large.

Masonry Abutments and Wood Superstructure.—In the construction of a system of improved roads in Orange County, N. C., under a $230,000 bond issue, the practice has been adopted on lightly traveled roads on which the average expenditure per mile has been limited, of constructing masonry abutments and piers suitable for steel structures but using a wood superstructure. The money

Unit Prices on Street Paving in Albany, N. Y., in 1913.—A recent report of the bureau of engineering for the city of Albany, N. Y., F. R. Lanagan, city engineer, states that 21 contracts for street improvement, 2 contracts for repaving and 1 contract for curb and walks were let during 1913. The total quantities and average unit prices were as follows:

Item.	Quantity.	Unit cost.
Excavation, cu. yds	56,351	$0.36
Vitrified block on concrete foundation, sq. yds	47,267	1.94
Dressed granite block on concrete foundation, sq. yds	1,852	3.55
Concrete pavement, sq. yds	5,350	1.35
Concrete pavement with Bit. top, sq. yds	3,272	1.78
Straight granite curb, lin. ft	25,297	0.92
Circular granite curb, lin. ft	1,509	1.41
Straight concrete curb, lin. ft	9,745	0.70
Circular concrete curb, lin. ft	634	1.13
Concrete sidewalk, sq. ft	121,754	0.17
Flag sidewalk, sq. ft	49,555	0.22
Brick sidewalk, sq. ft	132,625	0.13
Sod, sq. yds	5,060	0.18
Driveways, sq. yds	2,125	2.26

The cost of inspection on street paving and sewer work amounting to $402,000 was $9,060, approximately 2¼ per cent.

English Specifications for Road Tars.—Directions and specifications for the tar treatment of roads issued by the Road Board of Great Britain state that for tar No. 1 (suitable for the surface tarring of roads) the distillate between 170° and 270° C., or 338° and 518° F. (middle oils), shall remain clear and free from solid matter (crystals of naphthalene, etc.) when maintained at a temperature of 30° C. for half an hour. This requirement, however, may be waived in the case of tar supplied direct from gasworks; but tar from which the naphthalene has been extracted is preferable to tar containing much naphthalene. The tar must contain not less than 12 per cent and not more than 21 per cent by weight of free carbon. The free carbon is to be determined by complete extraction with benzol and carbon bisulphide of the bituminous matter from a weighed portion of the tar. The residue left is to be taken as free carbon. Specifications for tar No. 2 (suitable for making tar macadam, and for surface tarring in very hot weather, when the road crust is exceptionally dry) stipulate that the tar may contain not more than 25 per cent of its volume of the tar (or distillates or pitch therefrom) produced in the manufacture of caburetted water gas. The free carbon limit, which formerly was 18 per cent of the weight of the tar, has been altered to not less than 12 per cent and not more than 22 per cent by weight. Prepared pitch from tar distilleries must contain not less than 16 per cent and not more than 28 per cent by weight of free carbon; and commercial soft pitch not less than 18 per cent and not more than 31 per cent. In the latter case the pitch may contain not more than 25 per cent of pitch (formerly 10 per cent) derived from tar produced in the manufacture of carburetted water gas for both tar and tar oils.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

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Sand-Clay Road Construction in the Southern States.

A sand-clay road is a type of scientifically constructed earth road. It is an earthy hardpan produced or improved by the skillful combination or manipulation of sand and clay. The basic principle on which the strength of the surface is founded is that a mechanically interlocking aggregate of sand or gravel is cemented together by an adhesive earthy binder.

Within the past decade the sand-clay surface for country roads has been very generally adopted throughout the southern states. This type of surface appears to fit traffic and climatic conditions of the south and to meet the requirements of low first cost which distinctly rural and thinly populated districts demand in road construction. It is also being used to some extent in other sections of the country, especially in the repair of sandy stretches of road. In fact, some of so-called gravel roads of Indiana are nothing more than sand-clay roads.

There is nothing new in the fundamental theory of sand-clay roads or in the ordinary methods used in their construction. For generations it has been the practice in this country and abroad to use a "packing" earth both as road surfacing and as filler for stone roads. However, distinct advances have been made in recent years in the development of economic methods of handling the various combinations of sand and clay which result in packing or hardpan earths. In the last decade of the nineteenth century, Richland County, South Carolina, began the systematic construction of this type of road, using for the most part a natural mixture of sand and clay which exists abundantly in that county. A few years later representatives of the United States Office of Public Roads constructed many demonstration sections of sand-clay road and developed a technique of artificial mixing. Further analyses of correct mixtures and methods of securing them were made quite recently by various investigators.

In Georgia sand-clay surfaces are more generally used and have probably reached a higher stage of development than in any other state. Hence a study of sand-clay road construction there should emphasize the important points of construction; although methods suitable to Georgia conditions might not fit those of other sections. At the present time convict forces are used almost exclusively in construction work. Their use has resulted in a low first cost since practically the entire cost of building a sand-clay surface is made up of labor expense. With labor at from fifty to eighty cents per actual working day comparatively low first costs are possible.

An interesting point in connection with the article on sand-clay road construction contained in this issue of ENGINEERING AND CONTRACTING is the discussion of maintenance methods and costs. Practically no published data exist on the cost of maintaining sand-clay roads. It would seem from the data given that maintenance costs are variable—as might be expected with a friable type of surfacing—and that they are low in comparison with the cost of maintaining other types of surface.

A quality possessed to a marked degree by an earth surface is that of self-healing. That this should lead to economy in maintenance is evident. Another important factor in low cost of maintenance is the durability of the material. As long as the surfacing material remains on the road, is not blown or washed from the surfaced way, it is ordinarily good surfacing material. The stability of the surface, however, is inferior to that of more rigid types of surfacing.

It must be borne in mind, however, that sand-clay as a surfacing is suited only to light traffic. Under heavy traffic, especially steel-tired traffic, it disintegrates rapidly and it is almost impossible to keep the surface in a satisfactory state of repair. But for lightly traveled roads it is now and will probably remain an ideal surface which may be constructed economically in sections where the cost of labor is relatively low.

Uniform License Laws for Engineers.

A committee of six national engineering societies is now engaged in drafting a license law for engineers. When the committee will report to the society membership is unknown, but it will very likely be within the year. Whether he believes in licensing engineers and whether he may agree in detail with the provisions of the law as it will come from this committee, we think that every engineer is called upon to uphold the hands of this committee in its general plan.

As outlined, the main features of the license law planned by the committee are: (1) Requirements for registration of about the same character as those for associate members of the national societies of civil and mechanical engineers: (2) examinations to be made by a board of engineering examiners; this board to consist of nine members appointed by the Governor and serving without compensation, but with expenses reimbursed; a small number, not exceeding three, to constitute a quorum for examinations; (3) registration not required of those serving as assistants without professional responsibility to employers or clients; (4) a well-paid secretary and an office always open at the capital of the state; (5) engineers from other states having similar provisions for registration and duly registered in such states to be promptly admitted to practice without examination on payment of a small fee; and (6) engineers from other states not having such registration, but with a record of responsible work for a considerable period to be permitted to practice for a limited time on payment of a somewhat larger fee; this is to permit a reasonable time for examinations.

Whatever its wisdom the tendency toward legislation having the licensing of engineers for its object has become marked; several states have enacted license laws and others are planning to do so. It is extremely important that this legislation shall as far as possible be uniform in its requirements. If it is not, the practice of engineering will be hampered by unnecessary restrictions and communities will be hindered in their freedom to choose engineers best qualified to perform the service required.

New Types of Country Road Surfaces and the Bad Weather Factor.

Formerly, perhaps the most variable and troublesome element of risk in estimating the cost of building hard surfaced country roads was the difficulty of determining with any degree of accuracy the effect of weather conditions on lost time. In building gravel and macadam roads the experienced contractor estimates no item more liberally than the item of lost time due to bad weather.

New types of pavements, however, which are coming into vogue are reducing the importance of this factor. Brick and concrete roads and bituminous surfaces on a concrete base are essentially rigid types of construction. When the base is in place construction may proceed with but little regard to the weather. Asphaltic concrete may be laid in comparatively damp weather. Brick surfacing may be laid under similar climatic conditions, provided the sand cushion is not placed too far in advance of brick laying. Ordinarily a damp subgrade is desirable in the construction of either concrete surfacing or concrete base.

Moreover, the percentage of labor cost to total cost has been materially reduced by the increased use of relatively more expensive materials. This condition undoubtedly leads to more accurate estimating, since the cost of materials is a much more stable factor than the factor of labor expense. The importance of the foregoing is self-evident.

The adaptability of the plant required for brick and concrete road building to other quite different uses and its consequent increased salvage value is worth considering. Road rollers and stone crushers are of use only in road construction. Concrete mixers, wagons and motor trucks are used for many other purposes.

There is no question but that more plant and more expensive plant is required in the construction of these higher types of surfaces. A careful analysis, however, will show that the ratio of plant expense to total cost is no greater on construction at $9,000 to $15,000 per mile than on construction that costs at the rate of $1,000 to $5,000 per mile. Moreover, the reduction in cheap labor expense which is generally effected by the use of machinery, permits the employment of a much more reliable class of labor, although the rate of wage may be higher.

Country and suburban road construction is rapidly becoming an inviting field for the large contractor. Equipment requirements for building relatively higher types of surfaces are beyond the credit and capital of the small contractor. And yet with this change the opportunities for the contractor of limited means to secure sub-contracts for grading or hauling are increasing and much work will undoubtedly be handled in this manner in the future.

These conditions are doubtless familiar to all engineers and contractors actively engaged in road building. But changes have come about so rapidly and are today taking place so fast that even the specialist in this work, strive as he may to keep posted by assiduous reading and observation, is apt to fall behind. Also a vast number of men engaged in other lines of work still cling to the accepted notions of ten years ago, although developments of recent years have materially changed old methods and types of construction.

The influence of the use of new materials on the old variable of lost time will be felt. The lost time factor is still important but instead of being almost entirely dependent on bad weather, other factors such as delayed shipments, breakdowns of machinery, and strikes must be considered, which factors formerly were of relatively small importance.

WATER WORKS

Design Features of the Rapid Sand Filtration Plant of the Kensington Water Co. at New Kensington, Pennsylvania.

(Staff Article.)

The Kensington Water Co. supplies water to the Boroughs of New Kensington, Parnassus and Arnold, Pa. These towns lie along the east bank of the Allegheny River, 20 miles north of Pittsburgh. A railroad siding from the Allegheny Valley Branch of the Pennsylvania Railroad was built to the site of the plant near the village of Valley Camp, by the water company and was used to handle the construction materials, and is now used to deliver operating supplies to the plant.

The three towns are essentially industrial communities and have many large works stretched along the river front which furnishes many excellent manufacturing sites due to the unusually favorable conditions of lying above the high water mark and having good railroad and water facilities. To all appearances the three Boroughs are one town for, with few exceptions, the streets extend through from one Borough to the other. Their combined population was 12,100 in 1910.

The water supply is obtained from a pumping station on the river bank. Water enters the pump well from two large cribs built in the bottom of the river. These cribs are of timber construction and are rock filled. Water is admitted to this well from the cribs by gravity and is controlled by valves on the intake lines which are partly closed during high water. The new works here considered comprise the filter plant and settling basin which treat all of the supply before it is pumped into the force main.

Fig. 1. View of Water Purification Works of the Kensington Water Co. at New Kensingsington, Pa.

The filter plant is located at the pump station above the high water mark. It has a capacity of 3,000,000 gals. per 24 hours, or twice the present normal consumption, which permits the plant to be operated under present conditions on a 12-hour shift and furnishes ample margin for future growth for a considerable time. The plant is, however, so designed that it can be added to at any time.

The filter plant is located just outside of the pump station on the south side, with the tops of the filters above the high water mark and with a clear water basin extending under all the filters. The grade of the land around the pump station was 20 ft. below the grade of the railroad track in front of pump station, so that a retaining wall was built at this end

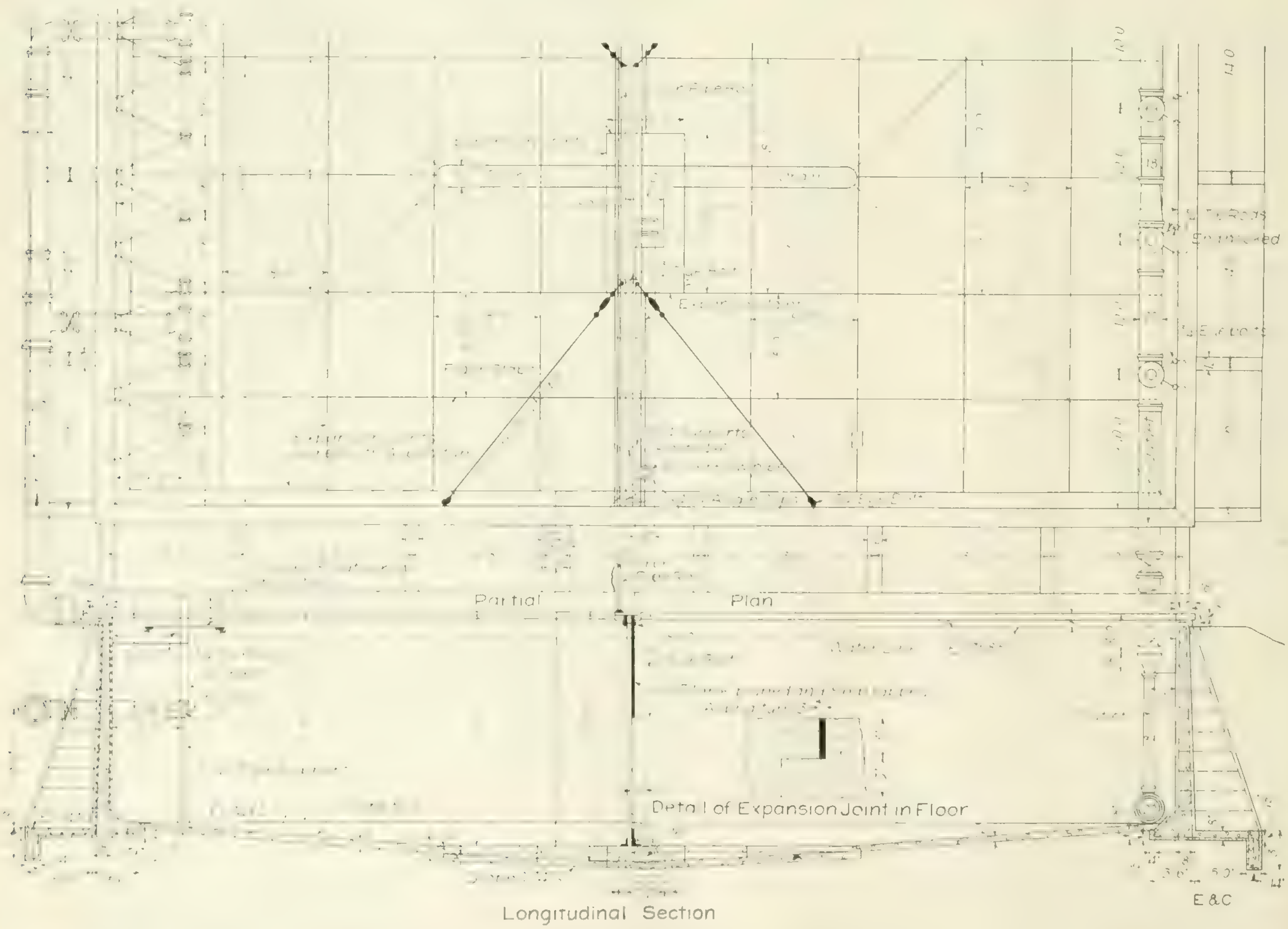
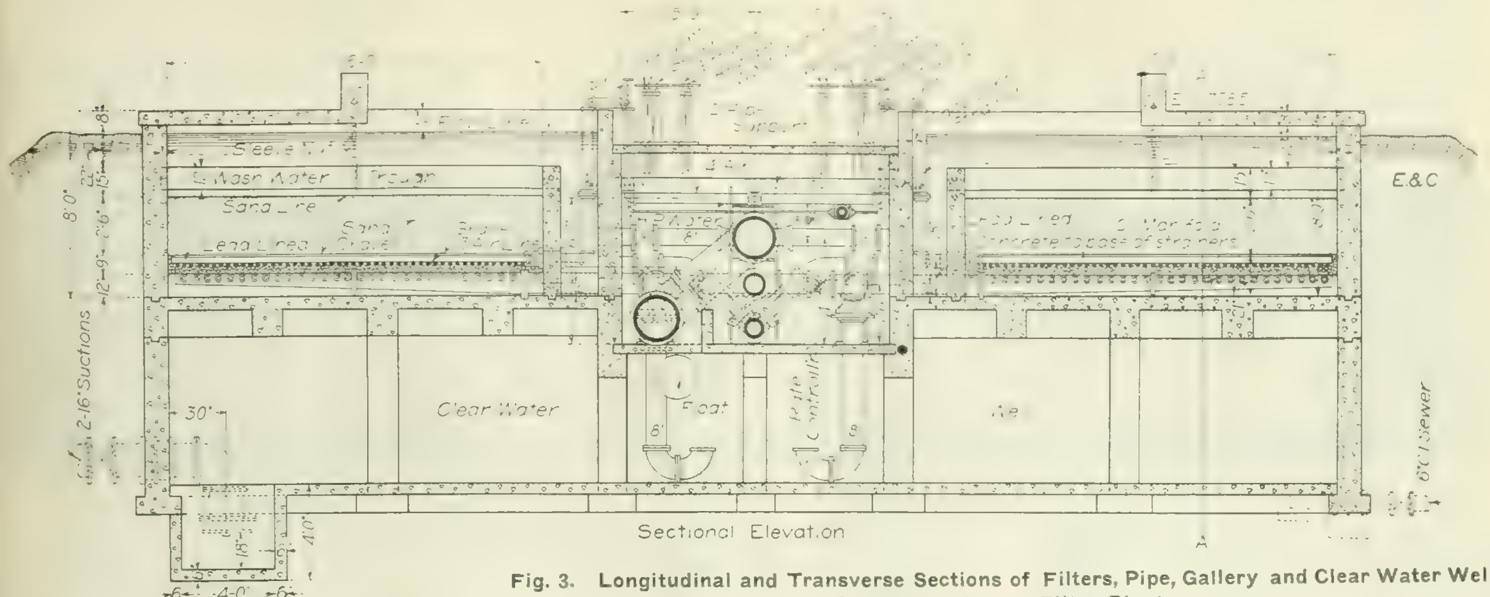


Fig. 2. Part Plan and Longitudinal Section of the Settling Basin of the Kensington Water Co. at New Kensington, Pa.

of the filter plant, and the coagulant house was constructed behind this retaining wall with its upper floor just above the upper grade line next to the railroad. A siding extends from



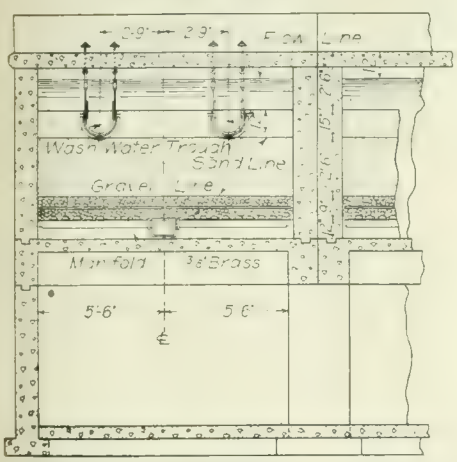


Fig. 3. Longitudinal and Transverse Sections of Filters, Pipe, Gallery and Clear Water Well of New Kensington Filter Plant.

the railroad to the coagulant house and to a coal pocket located between coagulant house and boiler house so that materials can be unloaded directly on to the storage floor of the coagulant house.

The settling basin is located on the north side of the pumping station with its top above high water mark. The basin is located far enough from the pumping station to permit of a future addition to the station and the boiler house in this direction. A general view of the plant is shown in Fig. 1.

Water is taken from the present wet well in the pump station and is pumped to the settling basin through a 20-in. cast iron main, which connects to the two new low service pumps each of 3,000,000 gal. capacity located in the old pump well. The old Nordberg pump is used for high service pumping and is connected up by a 16-in. suction line to the clear water basin. The old Snow pump is located in the pump pit next to the Nordberg pump and is connected by an independent suction line to the clear water basin. It is used as an auxiliary for high service pumping. Two new triple expanding, direct acting pumps are installed for high service and low service respectively.

THE SETTLING BASIN.

The settling basin is built of reinforced concrete and is 80 ft. long by 50 ft. wide by 16 ft. deep. It has a capacity of 500,000 gals., which permits a four hours sedimentation. A partial plan and a longitudinal section of the settling basin are shown in Fig. 2. Water is admitted at the end away from the pump station through two cast iron grids extending across the entire end of the settling basin and furnishing eight 6-in. inlet openings, spaced uniformly across that end at a distance of 2½ ft. above the flow line. A wooden float of 2-in planking is built around these inlets to assist in aeration and the thorough mixing of water. Water is taken off from the opposite end of the settling basin through four 8-in. outlets with flaring tops, located 2 ft. below the flow line and connecting with a 20-in. outlet main which extends to the filters. There is a 16-in. by-pass connection between inlet and outlet lines of settling basin to permit water to by-pass around the settling basin in case of an emergency. Figure 5 shows the basin in operation.

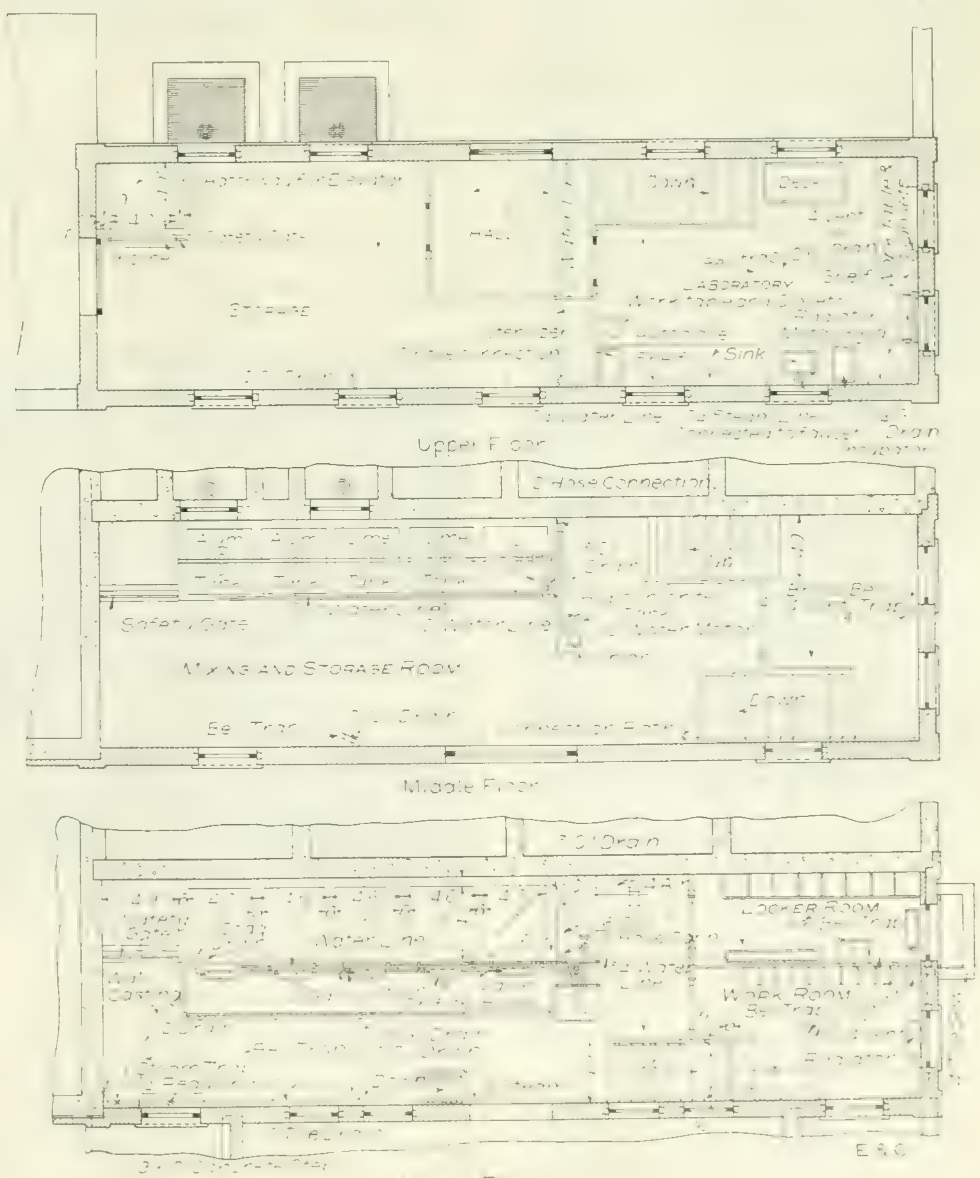


Fig. 4. Floor Plans of Coagulant House of New Kensington Filter Plant.

The walls of the settling basin are of the buttress type with earth carried up to the coping. The bottom of the basin slopes on a 5½ per cent slope to the center where it drains through a 16-in. sewer line, controlled by a gate valve to the river. There is also a 6-in. overflow pipe of cast iron located above the flow line and connecting with this drain line. The basin is baffled with a 1½-in. wooden baffle located across the center half way between the inlet and outlet ends and extending from the top of the basin to a depth of 7 ft. below the flow line. This baffle is securely held in place by ½-in. guy ropes provided with turnbuckles, and connected to the side walls of the basin.

The inlet and outlet cast iron manifolds shown in Fig. 2 are held securely in place by means of ¾-in. tie rods bolted to the concrete walls. A 3-in. wrought iron pipe column is placed under each riser on the inlet end. The wooden baffle built across the center of the basin is carried by vertical struts bolted to the floor with angle clips. All the lumber used in baffles and floats consists of well-seasoned, long leaf yellow pine equal to No. 1 Common. The outlet openings are funnel-shaped, and are constructed of ¼-in. riveted steel thoroughly coated with an asphaltum steel pipe coating.

The expansion joints in the plant consist of No. 28 gage copper plate extending the full length of the joint. Joints between adjacent sections of the floor in the settling basin and clear water basin are filled with asphaltum. These joints are ⅜ in. thick and extend the full depth of the upper layer of concrete. Before placing this asphaltum, the joints were thoroughly dried with a plumber's torch so that the asphaltum would adhere to the concrete, and after the joints cooled their surfaces were burned.

FILTER PLANT.

The filters are rectangular in plan and consist of six units each 11 ft. x 16 ft. in plan by 8 ft. deep. They are arranged in two groups of three units each with a pipe gallery 11 ft. 6 ins. wide built between them. The entire filter group covers a space 52 ft. wide by 39 ft. long, with an additional length on the end next to the railroad of 16 ft. for the coagulant house. Each filter box is built of reinforced concrete as a monolith with 12-in. walls. The filter superstructure extends over half of the filter area so as to permit the entire filter to be inspected from the operating platform. The rest of the filter is covered with a reinforced concrete roofing which serves as a sand platform and is just above the grade line. The net area of the surface of each filter is 176 sq. ft., giving a capacity of 500,000 gals. per 24 hours when operated at a rate of 125,000,000 gals. per acre per day. The flow line in the filter is 1 ft. below the flow line in the settling basin. A cast iron manhole frame with a cover containing glass sidewalk lights is placed in the reinforced concrete roof of each filter box. Sections of the filters, clear water well and pipe gallery are shown in Fig. 3.

Filtering Medium.—The filtering medium consists of 30 ins. carefully graded quartz sand resting on 10 ins. of hard flint seashore gravel and underlaid with a standard cast iron manifold equipped with strainers of the exterior orifice type spaced 6 ins. center to center in both directions. The effective size of the sand is 0.35 m. m. and the uniformity coefficient is 1.5. The gravel has a greater specific gravity than the sand and was screened through a ⅝-in. mesh and retained on a 3/16 mesh. The gravel stones are comparatively round and freedom from flat or light particles was sought.

Between the gravel and the sand there is a standard brass air system consisting of a manifold of ⅜-in. brass laterals spaced 4 ins. center to center and drilled with 3/64 in. holes every 2½ ins. These laterals are supplied by a 4-in. air pipe from a blower located at the end of the pipe gallery. The wash trough system consists of two steel troughs extending the full length of the filter and each is 15 ins. wide. They connect to a cross trough of reinforced concrete extending across the end of the filter next to the operating floor and to the full depth of the filter and drain into the sewer outlet through an 8-in. connection. The tops of the wash troughs are 15 ins. above the top of the sand line in the filters.

Collection and Wash System.—Raw water is admitted to each filter through an 8-in. valved connection to the raw water main in the pipe gallery. It empties into the trough system and thence overflows onto the filters. Wash water is admitted to the manifold system in the bottom of the filter through a 6-in. wash connection to the high service main controlled by a pressure reducing valve which limits the maximum pressure to 25 lbs. There is a 4-in. rewash connection provided to the outlet from each filter for wasting filtered water immediately after a filter has been placed in operation. This is connected to the sewer. Each filter is controlled by a standard adjustable rate controller, located in the pipe gallery and connected by a draft tube to the clear water basin so as to permit of the use of negative head. A standard loss of head gage and sampling pump is located on the operating platform in front of each filter and all valves are operated by standard wheel stands located on this platform.

The collecting system is designed for an even distribution of not less than 10 gals. of wash water per square foot of filtration surface per minute. The cast iron laterals are so coated as to expose no raw surfaces either on sides or ends. The air system is designed for supplying not less than 3 cu. ft. of free air per square foot of filter area per minute. Provision is made for introducing filtered water into the air distributing pipes. The arrangement of pipes conducting air to the filters is such that they pass an elevation in excess of the highest water in the filters to prevent any back flooding of the blower. They enter the filters in a manner that does not necessitate their passing downward through the sand. Openings in the distributing pipes are so arranged as to first empty of all water. The wash overflow system is so arranged that the horizontal flow at any point does not exceed 3½ ft., and has a carrying capacity sufficient to convey 10 gals. per square foot of filter surface per minute with a minimum drop at the rear end of the filters of 1 in. They are placed at the least minimum height at which sand will not be carried over when washing at the above rate. The strength of the trough was made sufficient to resist the upward thrust of the water when empty or to carry the trough full of water or sand, and in addition a 200-lb. concentrated load at any point. The troughs are hung by adjustable rods from the beams over the filters as shown in Fig. 3.

Fig. 5. Close View of New Kensington Settling Basin Showing Aerating Inlets in Operation.

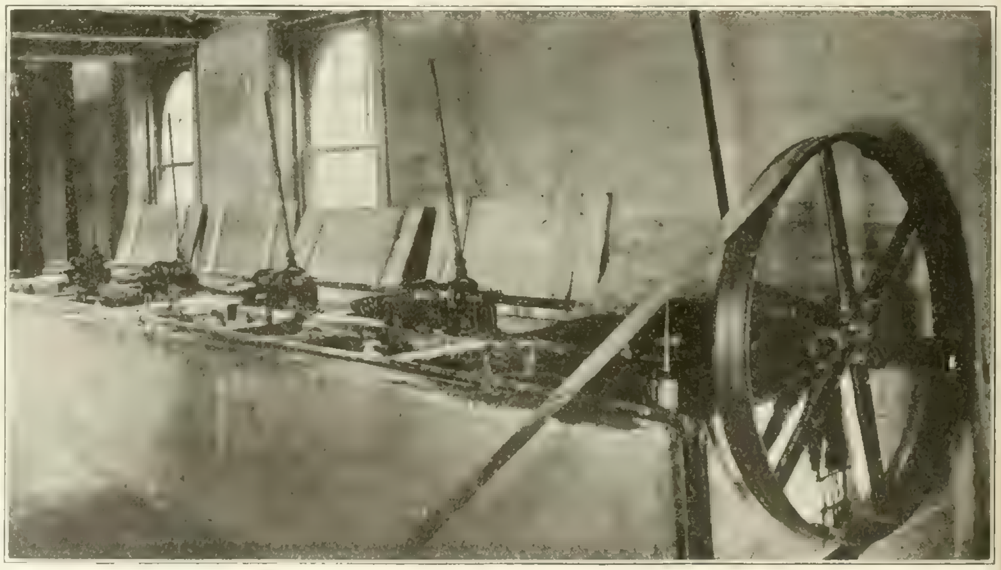


Fig. 6. View of Chemical Solution Tanks and Mechanical Stirring Device of New Kensington Filter Plant.

Pipe Gallery.—A section of the pipe gallery is shown in Fig. 3. It is situated between the filters and is connected to the coagulant house basement where a stairway extends to the pipe gallery from the operating floor above. The gallery contains all piping, valves, and controlling apparatus for the filters and a balanced float valve chamber for controlling the raw water. This chamber is built in the bottom of pipe gallery connecting with the clear water basin below the floor of the gallery and a large copper float in this chamber is connected to a balanced valve on the raw water line in the pipe gallery so as to close off the raw water when the clear water basin is full.

Clear Water Basin.—The clear water basin lies below the filters and pipe gallery, as shown in Fig. 3, and is also constructed of reinforced concrete. It is 50x37 ft. in plan with an effective depth of 7 ft. and has a capacity of 100,000 gals. or 50 minutes supply for the high service pump. The roof is of reinforced concrete, beam and slab construction, and supports the group of filters above. Entrance to the clear water basin is made through the float chamber in the bottom of the pipe gallery. Ventilation is obtained by 4-in. vent pipes carried up above the operating floor of the filter gallery and protected by return bends. A sump is located inside of the clear water basin next to the pump house and is connected to the high service pumps by two 16-in. suctions. The clear water basin is drained by means of a 6-in. cast iron drain line from the basin to the creek. There is no by-pass through which raw water can be sent around the clear water basin.

Coagulant House.—The coagulant house is a three-story structure with its upper floor just above the grade line along the railroad, and its bottom floor on a level with the operating floor for the filters. The floor between these is the charging floor for chemical tanks and the upper floor is used for storage. All floors are built of reinforced concrete and the walls are of brick and concrete. On the upper floor there is also located a laboratory which is equipped with the usual apparatus required for making simple determinations and controlling the operation of the plant. There is a hand elevator from this floor to the charging floor and to the lower floor for transporting chemicals to the coagulant tanks and materials to the pump station. Figure 4 shows the three floor plans of the coagulant house.

Chemical Apparatus.—The chemical apparatus is of the gravity type. There are five solution tanks constructed of reinforced concrete 4 ft. x 4 ft. 6 ins. in plan and 6 ft. deep. Two are used for alum, two for lime and one for chloride. In the top of each tank there is built a dissolving compartment connected with hot and cold water piping in which chemicals are mixed by hand and allowed to drain into the solution tanks. The tanks are agitated by a mechanical stirring device as shown in Fig. 6, consisting of paddles and chains connected to a vertical shaft in each tank and driven by a horizontal shaft extending over the tops of the tanks to a water motor operated with pressure from force main. Each tank is arranged so that the apparatus can be thrown out of commission by a lever. All the tanks drain through a 2-in. drain line connected to the sewer and are piped up to independent feed boxes located on a platform on the lower floor sufficiently high to feed by gravity to the raw water main in the pump station. Each of these boxes is controlled by a glass float and supplies the solution through an adjustable orifice with a sight feed. The boxes are standard orifice boxes of cast iron with white porcelain linings and each is provided with a bronze float inlet valve and a calibrated orifice plate on the outlet to be adjusted by a graduated index wheel. Each orifice box feeds by sight into the feed lines to the raw water main outside of the building. A view of the orifice boxes is shown in Fig. 7.

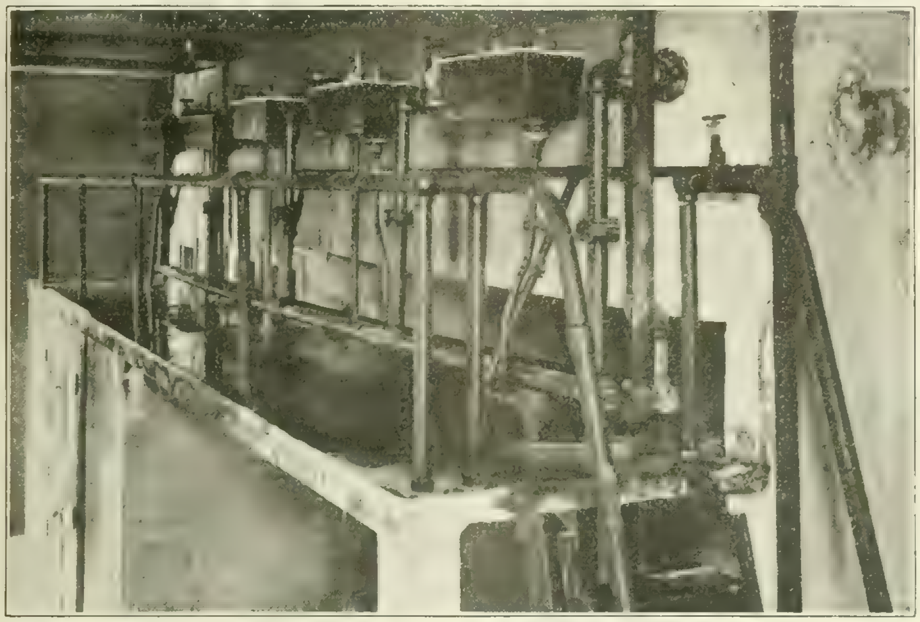


Fig. 7. View of New Kensington Filter Plant Orifice Boxes.

Controlling Apparatus.—The clear water outlet from each filter is provided with a rate controller capable of regulation as to capacity from 40 per cent overrating to 25 per cent underrating. A loss of head gage is furnished for each filter. Each gage is equipped with a direct-reading scale graduated in feet and tenths. These gages are of the manometer type. They have brass trimmings and are placed on a cast-iron base in front of each filter. A sampling pump is furnished for each filter and is located on the same stand with the loss of head gage. This pump is connected up so as to take samples from the clear water outlet of the filter. A view of the Humphreys loss of head gage and sample pump is shown in Fig. 8.

Air Blower.—The air blower is located in the basement of the coagulant house at the end of the pipe gallery on a concrete foundation. It has a capacity sufficient to deliver 525 cu. ft. of air per minute against 3 lbs. pressure. It is driven by a hydraulic motor of sufficient horsepower to operate blower at full capacity with a water pressure of 125 lbs. A drain pipe is laid from the hydraulic motor to the clear water basin and a connection to the motor is taken off from the wash water line with suitable piping connections properly valved. The motor is directly connected to the blower.

On the lower floor of the coagulant house there is a locker room and a work room. Both buildings are heated by steam and are lighted with electric lights. The superstructures are of brick and are covered with Spanish tile roofs. The retaining wall on the railroad side of the coagulant house is of reinforced concrete of the buttress type, similar to the retaining wall for the settling basins. The pocket between the coagulant house and the boiler house is also of similar design.

PERSONNEL.

The plant here described was designed by Messrs. Chester & Fleming, consulting engineers, Pittsburgh, Pa., who also supervised construction operations. Mr. George Hendrickson represented the engineers as resident engineer in charge. Mr. H. S. White was general contractor. The filter equipment was furnished by the Pittsburgh Filter Manufacturing Co. of Pittsburgh, Pa.

The Removal of Iron and Manganese from Well Water Supply at Lowell, Mass.

An article published in this section for July 15, 1914, described the results of experiments on the removal of carbonic acid from the Cook well water at Lowell, Mass. The article also stated the city's present water supply problems. The present article describes, briefly, the results of experiments on the removal of iron and manganese from the water drawn from the system of wells known locally as the Boulevard Wells. This article also gives a brief description of the design of the plant proposed for the removal of these metallic substances from the city's water supply. As in the case of the former article the information here given is taken from the report on the improvement of the water supply by Mr. F. A. Barbour, consulting engineer, Boston, Mass.

Fig. 8. View of Humphreys' Loss of Head Gage and Sample Pump Installed at New Kensington, Pa., Water Filtration Plant.

Contracts for putting down 169 of the driven wells in the Boulevard system were awarded in 1895. In 1900 52 wells were added and in 1901 the number of wells was increased by 125. As stated in the article above mentioned the Cook Wells were condemned in 1899 because of the action of their waters on the lead service pipes in use in the city. This increased the draft from the Boulevard well field and coincident with the increased draft a gradual deterioration in the quality of this supply resulted, due to increasing amounts of iron and manganese, and evidenced physically by pronounced color and turbidity. In an endeavor to obtain more and better water, 118 additional wells were driven in 1911, the result being a temporary improvement in quality, but within a few months, metal contents again increased to a point which rendered the supply disagreeable and altogether unsuitable for municipal service.

The progressive deterioration of the Boulevard supply is indicated by the increase in iron and ammonia. This indicates overworking of the soil from which the supply is drawn. The report gives a table of 5-year averages of chemical analyses of the Boulevard well water for the period from 1896 to 1913. In 1896-1900 the free ammonia amounted to .087 and the albuminoid ammonia to .032 p.p.m., respectively. The corresponding figures for 1911-13 were .372 and .067. The iron content increased from .322 p.p.m. in 1896-1900 to 1.468 p.p.m. in 1911-13. It is planned to check this progressive deterioration by extending the well system and balancing the amounts drawn from different parts of the field.

The Boulevard supply has always been hygienically safe, a fact shown by the low typhoid death rate of the city. To make it entirely acceptable it is necessary to remove the iron and manganese, and to determine how this could be done most efficiently and economically experimental work was undertaken. A small test plant was erected at the Boulevard station. This plant consisted essentially of a prefilter of coke or gravel and a sand filter. A series of 18 experiments was performed. During the first 15 experiments the depth of gravel or coke was 2 ft. 6 ins. For the last three experiments the depth of coke in the prefilter was 7 ft. 8 ins. Aeration was employed in connection with the first 15 experiments by means of a pan aerator at the top of the prefilter. In the sand filter the depth of sand was 2 ft. 1 in. The effective size of the sand was 0.34 and its uniformity coefficient was 2.56. This sand rested upon a layer of coarse sand 3⅛ ins. thick and this, in turn, rested on 6 ins. of graduated screened gravel.

The reason for the experimental work is found in the fact that different waters containing metals require quite different treatment. Iron, which is found in the form of ferrous hydrate, can be readily oxidized, and if there are no interfering substances, such as manganese, carbon dioxide, or organic matter, which hold the iron in semi-solution, or in a colloidal condition, it can be precipitated and removed by aeration and sand filters. In some waters, however, excessive aeration is possible, and a retention of a certain portion of the carbon dioxide is necessary to prevent the organic matter from interfering with the precipitation of the iron. Also, it has been found that manganese interferes, and the removal of the last traces of iron, if the necessary conditions are not provided for the coincident removal of the manganese, is impossible. It therefore follows that, while the deferrization of some waters involves simple aeration and sand filtration, others require more highly developed preliminary treatment by passage through coarse filters, operating either as tricklers or contact beds. The experiments performed therefore comprehended a progressive study leading to the determination of the method necessary for the successful treatment of the water.

Field analyses made of the raw water during the progress of the week show that the carbonic acid content ranged from 18.4 p.p.m. to 25.8 p.p.m.; the iron from 1.040 p.p.m. to 2.080 p.p.m., and the manganese from 1.700 p.p.m. to 3.400 p.p.m. The amount of dissolved oxygen in the raw water, expressed as per cent. saturation, ranged from 9.0 to 22.8.

Until this investigation was undertaken, no regular analyses had been made for manganese in the Boulevard supply, and it is of interest to note that the amount of this metal present in the water, which is largely responsible for its unsatisfactory condition as delivered to consumers, exceeds the iron present by about 40 per cent.

The procedure in the 18 experiments and the results obtained are recorded in detail in the report The period of experimentation was from Aug. 19, 1913, to April 25, 1914. Enough has been said to indicate the general scope of the experiments and we here record the conclusions based upon them.

CONCLUSIONS FROM EXPERIMENTS.

The experiments indicate that the iron and manganese in the Boulevard water cannot be removed by aeration and direct application to sand filters, nor by aeration, sedimentation and sand filters.

That excessive aeration and the operation of the prefilter as a trickler is not adapted to the treatment of the Boulevard water.

That the iron and manganese can be successfully and economically removed by limited aeration, passage through a coke prefilter not less than 8 ft. in depth, operated as a contact bed at a rate of 67,500,000 gals. per acre daily, and subsequent filtration through sand at a rate of 10,000,000 gals. per acre daily.

On these conclusions, plans of a purification plant have been prepared, based on the use of coke prefilters 10 ft. deep, an intermediate sedimentation basin of 1 hour's capacity, and sand filters 3.0 ft. deep. The depths of both filters are greater than in the experimental apparatus, and the addition of the intermediate sedimentation basin will serve to reduce the work to be done by the sand filter. It is believed the experiments were sufficiently prolonged, and on such a scale as to permit safe conclusions to be drawn, and that there can be no doubt that the plant designed will sucessfully remove the iron and manganese and render the Boulevard supply entirely satisfactory.

THE PROPOSED PLANT FOR REMOVING IRON AND MANGANESE.

The proposed purification plant consists of 6 coke prefilters, 10 ft. in depth and 2/5 acre in total area; a settling basin, divided into 2 units, with a total capacity of 500,000 gals.; 6 sand filters, with a total area of 1 acre; and a filtered water reservoir of 1,000,000 gals. capacity.

Over the inner walls of the prefilters a superstructure is to be built, so as to provide an operating gallery under cover. Immediately in front of this building, and connected thereto, a headhouse and laboratory is located, and practically all the operations involved in the control of the plant are concentrated in this central structure, which contains the main valves and the recording apparatus.

The present Holly pump will be used to draw water from the wells and lift it to the prefilters. After passing through the coke, the water will flow through orifice boxes, by which the prefilter rate will be determined and recorded, thence through the settling basins to the sand filters, from which it will return through effluent pipes and Venturi meters to the headhouse, and finally flow through a conduit in the central wall of the settling basins to the filtered water reservoir, from which it will be lifted by new steam turbine driven centrifugal pumps, located in a small extension of the present station, into the force main.

At a rate of 75,000,000 gals. per acre per day through the prefilters, and a 10,000,000 gal. rate through the sand filters the areas provided are equal to a 10,000,000 gal. daily output. Allowing for cleaning and for the possible desirability of a lower rate through the coke, the plant is believed to be ample for an average daily supply of 7,500,000 to 8,500,000 gals., or sufficient for the needs of the city until 1935. It is to be noted that the depth of coke is 40 per cent greater and that of the sand 50 per cent greater than the corresponding dimensions of the testing apparatus, and that there is interposed between the prefilter and sand filters a settling basin which will materially reduce the load on the final filters. Provision is made for elasticity of operation to meet the varying conditions which may develop in the raw water.

The estimated cost of the prefilters, settling basins, sand filters, filtered water reservoir and superstructures is 147,385. The estimated cost of the new low lift pumping equipment and pipe connections to the purification plant is $179,833.

The figures indicate that, including interest, depreciation and operating charges, the iron and manganese can be removed from the Boulevard supply for an average cost of $7.65 per 1,000,000 gals. during the next 25 years.

BRIDGES

Standard I-Beam and Pile Highway Bridges of the Iowa State Highway Commission.

(STAFF ARTICLE.)

The Iowa State Highway Commission has prepared standard plans for a number of different types of bridge construction. In this article we shall give essential data on the design of I-beam bridges with concrete floor slabs, pile bridges, and wood piling abutments for steel highway bridges.

STANDARD I-BEAM BRIDGES WITH REINFORCED CONCRETE FLOOR SLABS.

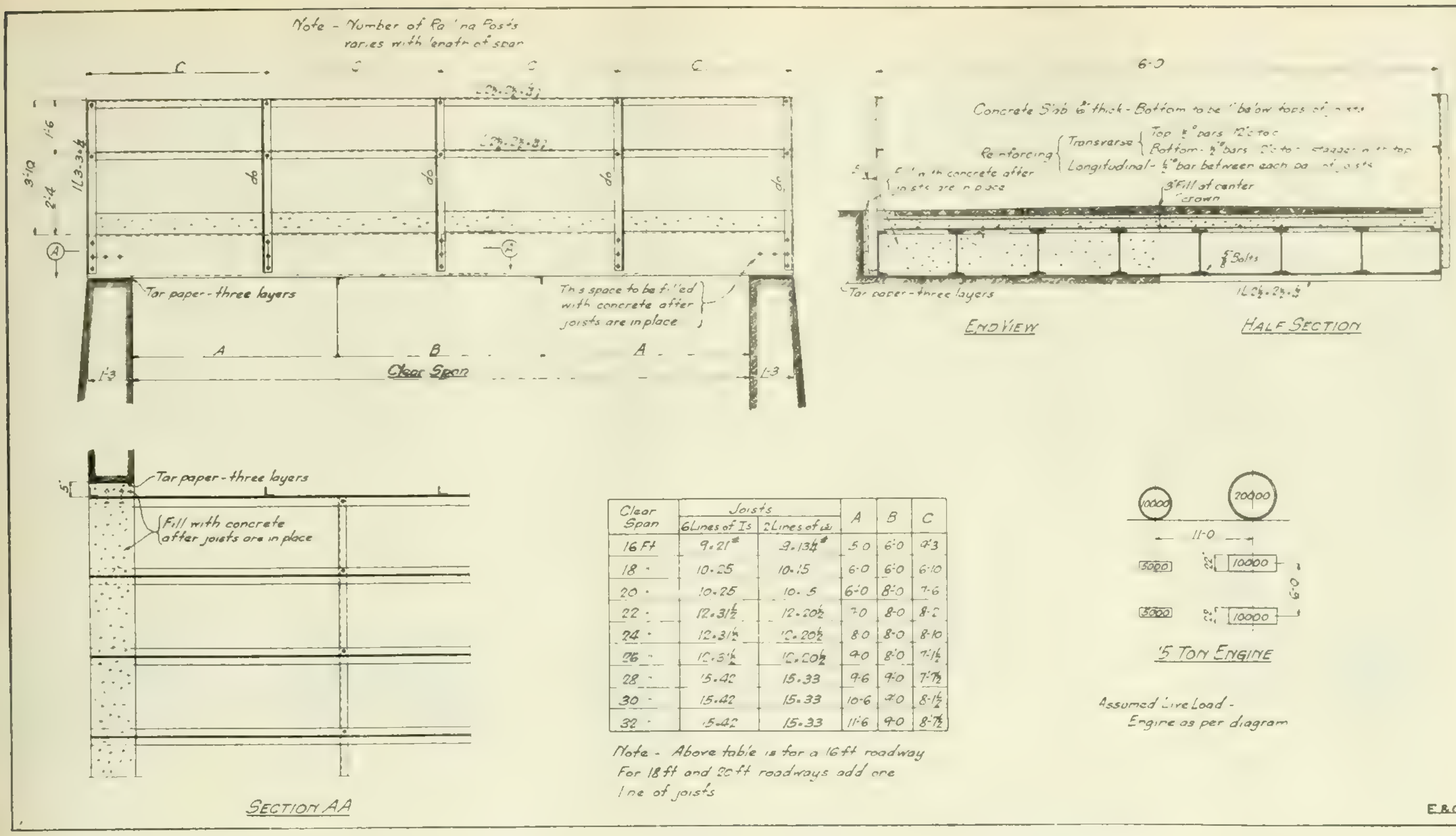
The standard I-beam bridges with reinforced concrete floors cover clear spans from 16 ft. to 32 ft., inclusive.

Loads.—In estimating the dead load the weight of the earth filling is taken at 120 lbs. per cubic foot, and that of the concrete at 150 lbs. per cubic foot.

The live load for the beam spans consists of a uniform load of 100 lbs. per square foot or a 15-ton traction engine, two-thirds of its weight being concentrated on the rear axle and one-third on the front axle. The distance between the front and rear axles is 11 ft., and the distance center to center of rear wheels is 6 ft., these wheels being 22 ins. wide. One-third of a wheel load is assumed to be carried by each joist.

Concrete Proportions.—The concrete for the floor slabs consists of 1 part cement, 2 parts sand, and 4 parts crushed stone or screened pebbles passing a 1½-in. screen. For reinforced concrete abutments and wing walls a 1:2:4 mix is required, while for the footings a 1:2½:5 mix is used. For gravity abutments, the body of the abutments consists of a 1:2½:5 mix, the coping of a 1:2:4 mix, and the footings of a 1:3:6 mix.

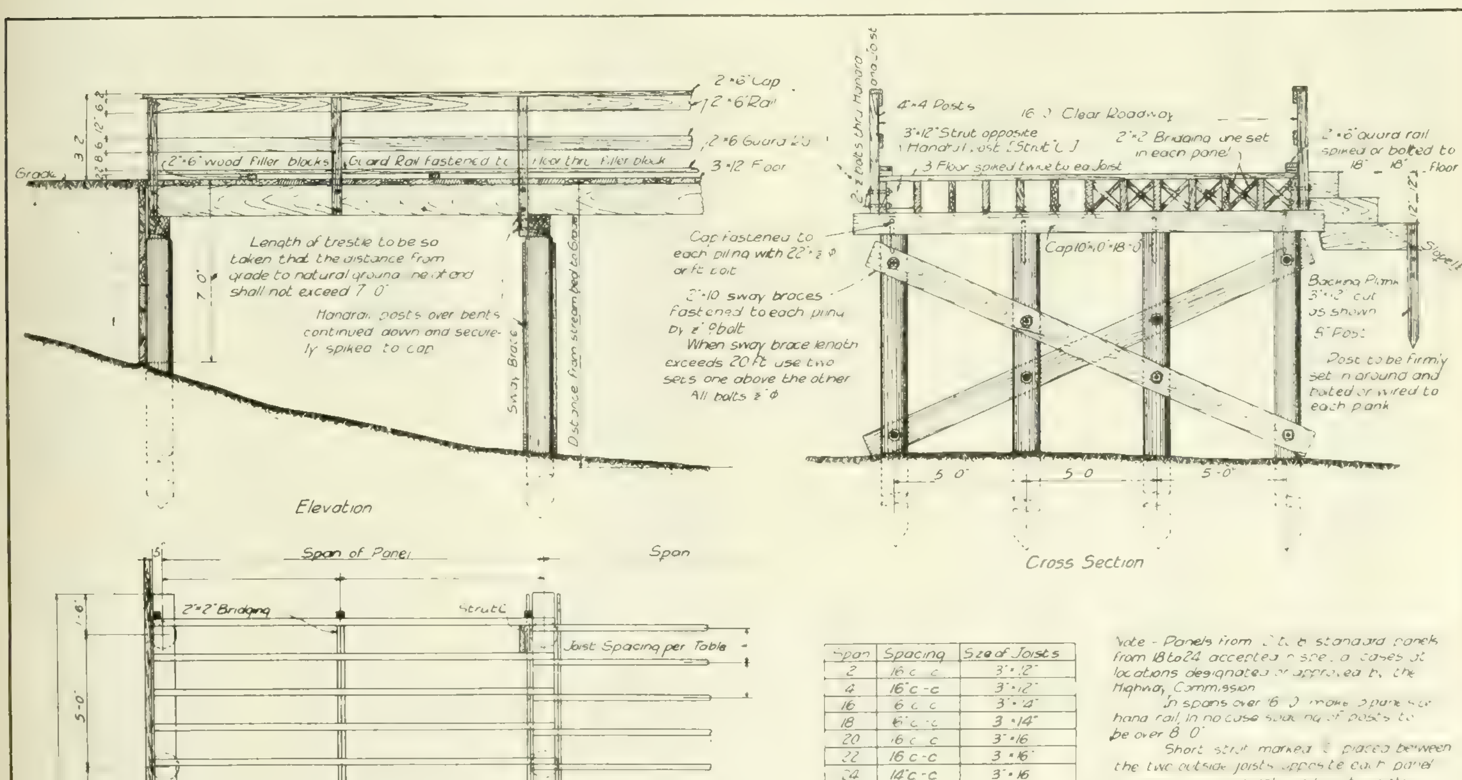
Allowable Stresses.—The allowable unit stress on the extreme fibers of the I-beams is 16,000 lbs. For the reinforced concrete floor slabs the allowable tension in the reinforcing steel is 16,000 lbs. per square inch, and the allowable compression in the concrete is 600 lbs. per square inch. For the abutments the allowable tension in the steel is 16,000 lbs. per square inch, the allowable compression in the concrete, 500 lbs. per square inch, and the allowable bearing on the bridge seats, 400 lbs. lbs. per square inch.

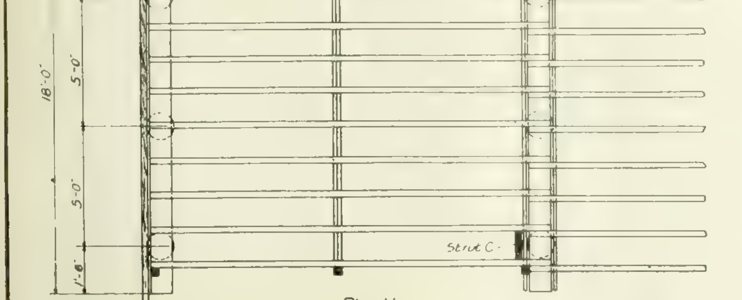


Clear Span	Joists, 6 Lines of Is	Joists, 2 Lines of ⊔	A	B	C
16 Ft	9-21#	9-13¼#	5-0	6-0	4-3
18 "	10-25	10-15	6-0	6-0	6-10
20 "	10-25	10-15	6-0	8-0	7-6
22 "	12-31½	12-20½	7-0	8-0	8-[illegible]
24 "	12-31½	12-20½	8-0	8-0	8-10
26 "	12-31½	12-20½	9-0	8-0	7-1½
28 "	15-42	15-33	9-6	9-0	7-7½
30 "	15-42	15-33	10-6	9-0	8-1½
32 "	15-42	15-33	11-6	9-0	8-7½

Note - Above table is for a 16 ft roadway. For 18 ft and 20 ft roadways add one line of joists.

Fig. 1. Standard I-Beam Bridges With Reinforced Concrete Floors of the Iowa State Highway Commission.





Span	Spacing	Size of Joists
12	16" c-c	3"×12"
14	16" c-c	3"×12"
16	16" c-c	3"×14"
18	16" c-c	3"×14"
20	16" c-c	3"×16"
22	16" c-c	3"×16"
24	14" c-c	3"×16"

Fig. 2. Standard Pile Highway Bridge (Class B Floor Loading) of the Iowa State Highway Commission.

Design Drawings and Data.—Figure 1 gives complete data for designing the superstructures of I-beam bridges with reinforced concrete floor slabs having spans varying from 16 to 32 ft. and widths of 16, 18 and 20 ft.

PILE HIGHWAY BRIDGES.

Figure 2 shows drawings and data for designing standard pile highway bridges with "Class B" floor loading. For this loading the uniform live load is 100 lbs. per square foot, and the minimum concentrated live load is a 10-ton traction engine having the distribution of loadings directly proportional to that specified for the 15-ton engine loading given in

Fig. 1. The data given in Fig. 2 apply to spans varying from 12 to 24 ft. and to a 16-ft. clear roadway. The specifications for timber piles require that they shall have a minimum diameter at the top of 8 ins. and a minimum diameter at the butt equal to 1 in. for each foot of length. The safe load on a pile is to be determined from the formula, $P = \frac{2wh}{s+1}$, in which w = weight of hammer, h = fall in feet, and s = final penetration in inches.

WOOD PILING ABUTMENT.

Figure 3 shows a standard design of a wood piling abutment for a steel highway bridge. This design is for a clear roadway of 16 ft. By referring to Fig. 3 it will be noted that the piles are capped with a 15-in. 33-lb. channel, which is connected to the piles by means of two 8-in., 11¼-lb. channels, the latter being bolted to each pile by a ¾-in. bolt. Other details are clearly shown by the drawings.

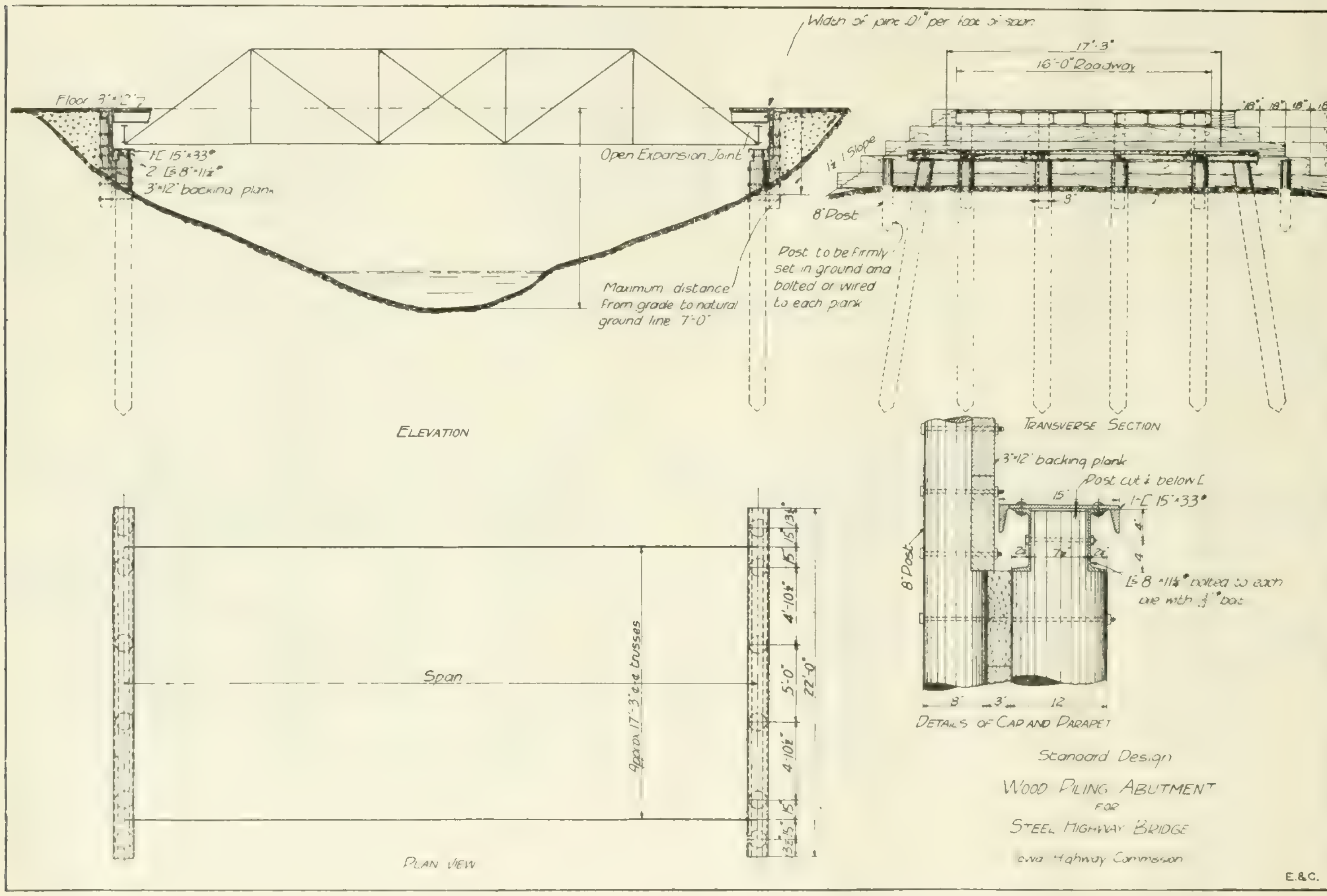


Fig. 3. Standard Wood Piling Abutment for Steel Highway Bridge of the Iowa State Highway Commission.

Mr. Thos. H. MacDonald is highway engineer and Mr. J. E. Kirkham is consulting bridge engineer of the Iowa State Highway Commission.

Construction Features of the Quebec Bridge Substructure.

In our issues of Oct. 1, 1913, and Jan. 7, 1914, we published data on the design and erection of the new Quebec Bridge superstructure. In this issue we shall consider the construction features of the substructure of this important bridge. The article is based on a paper by H. P. Borden, assistant to chief engineer, Quebec Bridge Commission, in the "Canadian Engineer."

The contract for the construction of the piers of the Quebec Bridge was awarded in February, 1910. The work has been continued constantly since that date, and is now practically completed. The contract as finally completed is divided into the following units: North abutment (alterations), 404.5 cu. yds.; north intermediate pier, 1,665.6 cu. yds.; north anchor pier, 17,736 cu. yds.; north main pier, 31,870.4 cu. yds.; south main pier, 38,279.4 cu. yds.; south anchor pier, 16,073 cu. yds.; south abutment (alterations), 61.1 cu. yds.; total, 106,090 cu. yds.

At the start a careful study was made by the board appointed by the government, to determine whether it was possible to use the old masonry. After a thorough investigation it was found that, owing to the increased weight of the steelwork, all the old masonry, with the exception of the abutments, would have to be taken down and new piers constructed. It was therefore decided to move the whole bridge to the south about 65 ft., retaining the original longitudinal center line. This brought the north main pier further into the water and the south main pier the same distance towards shore, the same center to center length of span of 1,800 ft. being retained. It was impossible to place the south main pier nearer the river on account of the wreckage which lies in the water at that point.

BORINGS.

Before the contract was awarded a series of borings was made at and about the location of the two main and anchor piers. Nineteen borings in all were taken, each boring penetrating at least 15 ft. into solid rock in order to make sure that it was bed rock rather than a boulder that had been struck. These borings showed that bed rock would be encountered approximately at elevation 0.0 on the location of both north and south main piers, which elevation was about 101 ft. below extreme high water and 70 and 85 ft. below the bed of the river on the north and south sides respectively. The formation of the bed of the river on the two sides, however, was found to be totally different. On the north side heavy boulder formation was encountered for the entire depth, the boulders being closely packed together with coarse sand and gravel. On the south side the borings showed sand formation for the entire depth with only a sprinkling of boulders at various points. The bed rock was a hard sandstone, called "Sillery grit," overlaid with a red and gray shale. On the south side 2 ft. of hardpan overlaid this shale.

ORIGINAL CAISSON FOR NORTH MAIN PIER.

The caisson for the north main pier was started first and was constructed at Sillery, about three miles down the river. This caisson was 180 ft. long by 55 ft. wide. It was constructed of 12x12-in. southern pine with a cutting edge of the same material 30 ins. square. This cutting edge was shod with a 6x12-in. oak timber instead of the steel shoe commonly used. It was claimed in this case that if any distortion of the caisson took place the steel shoe would tend to prevent the caisson from readily readjusting itself (as would be the case with a wooden shoe) and that the wooden shoe gave sufficient service during the process of sinking. The caisson had a working chamber 8 ft. high in the clear, divided by longitudinal and transverse bulkheads into 18 compartments. It was built in the winter under a construction shed, thus enabling the men to work without interruption. The caisson was built over launchways having a 10 per cent grade which led out into deep water. The walls of the caisson were built up about 40 ft. before it was launched. When ready for launching the caisson was lowered to its inclined position on the launchways by means of heavy jacks. When everything was ready an impetus was given by jacks placed horizontally at the rear, the launching being effected without mishap.

The caisson was towed to the bridge June 14, 1910, and was placed in position over the site, which had been previously dredged to an average depth of about 20 ft. in order to push ahead the work of sinking as fast as possible. The work of filling it with concrete was started immediately, and about 2,000 cu. yds. of concrete had been deposited before the caisson began to touch bottom at its corners. The caisson leaked considerably, but it could readily be kept dry by means of two pumps. At this time, however, an accident happened to the boiler equipment and before it could be repaired the caisson had been filled with water to such an extent that it grounded on an uneven bottom. The result was that the caisson was seriously strained and the seams opened up to such an extent that it was found impossible to keep air in the working chamber. It was decided to remove the concrete from the caisson and tow it to St. Joseph de Levis and have permanent repairs made there during the coming winter.

In view of this accident a reconsideration of the masonry design was made by the board, with the result that it was decided (in consequence of the difficult sinking on the north side) to use two caissons for the north main pier and to use the reconstructed larger caisson for the south side, where the sinking operations would be much simpler and the material to be penetrated would be, as shown by the borings, composed mostly of sand. This entailed the abandonment of the enlargement of the south main pier and required the sinking of a caisson south of the old pier and entirely distinct from it. It was therefore decided to sink this new large caisson—or caisson No. 1 as it has been designated—65 ft. nearer the shore, or south of the existing main pier, and to sink the caissons for the north main pier the same distance towards the river, or south of the existing north main pier, thus making the span 1,800 ft.—the same as that of the original bridge. This change in the plans allowed the board to keep the center line of the bridge coincident with that of the old structure, which was a very important item as it would avoid the large expense of changing the location of the railroads approaching both ends of the bridge.

It was found that caisson No. 1 could be satisfactorily repaired in dry-dock, and on May 28, 1911, it was floated out and towed up the river about nine miles to the site on the south side which, being exposed at low water, had been carefully leveled off. At extreme high water there is about 15 ft. of water over this prepared bed. As the caisson (due to its construction) had a very deep draught, a false bottom was constructed with a view to decreasing this draught before floating into position. The result was that the caisson floated with a draught of 11 ft. and was placed in its exact position for sinking without serious difficulty. The openings in the various shafts were then left unobstructed in order that the rise and fall of the tide would not lift the caisson from its permanent bed. The caisson was left in this position throughout the season of 1910, the work of the contractor being directed towards the sinking of the caissons on the north side of the river.

CAISSONS NOS. 2 AND 3 FOR NORTH MAIN PIER.

Caissons Nos. 2 and 3, for the north main pier, were constructed at Sillery on the same location as caisson No. 1, the same details of construction being followed throughout. Each of these caissons was 85 ft. long by 60 ft. wide. Caisson No. 2 was started June 15, and caisson No. 3 on June 29, 1911. Both of these caissons reached their permanent location at elevation 20.0 about Oct. 20, 1911.

The average rate of progress of sinking the westerly caisson (No. 2) was 0.37 ft. per day, and that of the easterly caisson (No. 3), 0.47 ft. per day. It was the original intention to sink these caissons to rock, but as the work progressed the sinking became more difficult, and finally, when the caissons had reached elevation 20.0, it was considered that the foundations at this point were quite satisfactory for many times the load that the piers would be called upon to carry.

Bearing tests were made at this point to determine the supporting value of the foundation. A cube of granite 2 ft. square was placed on an average section of the bottom and over this was placed a lever composed of two I-beams supported on pin bearings. The short end of the I-beams was supported against the roof of the caisson. A hydraulic jack was placed to exert a definite load at the end of the longer lever arm. A load of 59 tons per square foot showed a settlement of only ⅛ in., practically no settlement at all being noticed at 20 to 30 tons. As the average working load at the foot of this pier was only 8 tons per square foot, it was considered that the board would not be justified in carrying the foundations to a lower level.

In the operation of sinking these caissons the contractor met with considerable difficulty owing to large boulders fouling the cutting edge, and in several places this cutting edge was forced inward from 6 to 10 ins., and, as it was feared that if the sinking was continued in the same manner the cutting edge would be further distorted and sinking operations endangered; the method of sinking was then changed so as to avoid any such contingency.

Timber blocking was placed beneath the bulkheads and also at the center of the chambers. A trench was then excavated all around and below the cutting edge and for a distance of several inches outside the exterior surface of the caisson. This trench was excavated to a depth of about 2 ft., after which it was filled with blue clay, in bags. When all was ready the blocking was under-scoured with water jets, and the caisson lowered on a cushion of clay. The clay tended to act as a lubricant and also prevented considerable air leakage, and as all boulders were removed from beneath the cutting edge before the caisson was lowered, all further damage to the cutting edge was prevented, and it was found that the sinking was carried on even more rapidly.

After the caisson had reached its final location the working chamber was filled with concrete composed of 1 part cement, 2 parts sand, and 4 parts small crushed stone. This concrete was made much drier than the concrete used in the main caisson, it being found that concrete deposited under compressed air gave better results when very dry than in a more or less liquid state.

The concrete was deposited in terraces, the men working towards the center from the sides and ends. Great care was taken to ram the concrete thoroughly around the roof timbers so that a bearing would be assured under the roof of the working chamber. After the working chamber was filled as carefully as possible by hand the shafts were filled with concrete. As a further precaution, a rich grout was forced in through 4-in. blow pipes by compressed air, under a pressure of 100 lbs. per square inch. One hundred fifty-four bags of cement were used in grouting caisson No. 2, and 274 for caisson No. 3.

Caissons Nos. 2 and 3 were sunk with a 10-ft. space between the two ends, thus making the overall length of the two caissons 180 ft., the same as for caisson No. 1. After they had been filled with concrete, the space between them was dredged with a clam-shell bucket to a depth of 25 ft. below high water, the boulders and hard sand being excavated with considerable difficulty. Shutters 40 ft. high, made of 12x12-in. timbers, were placed vertically against the outside walls of the adjacent caissons so as to close each end of the space between the caissons and overlap about 12 ins. on their sides. The bottoms of the caissons were banked on the outside with clay dumped in the river and covered with heavy rip-rap. The shutters were securely bolted to the caisson walls down to low-water level, thus forming cofferdam walls which enclosed the space between the caissons. This space was then filled with concrete deposited under water up to an elevation of 7 ft. below low-water mark. After the concrete was deposited the water was pumped out, and the space between the caissons was then bridged by six old steel girders, 6 ft. deep, resting in pockets left in the concrete in the adjacent ends of the caisson, the wooden walls of the caisson having been cut away to allow this to be done. Afterwards the concrete was deposited in a continuous mass in and between both cofferdams and caisson, thus forming a monolith upon which the masonry shaft of the pier could be carried. The masonry of the pier was then built up inside of the crib work, which was kept in place until the mason work had extended above high water.

CAISSON FOR SOUTH MAIN PIER.

The sinking of the large caisson for the south main pier was started July 28, 1912, and was completed Oct. 24, 1912, or at the rate of 0.75 ft. per day during the entire period. The material encountered at this point was, as indicated by the borings, chiefly sand, and required that the pier be carried down to rock, which was reached at elevation 0.0, 101 ft. below high water, and 86 ft. below the bed of the river. The difficulty experienced on the north side in keeping the cutting edge intact, and also on account of the fact that the caisson had previously been overstrained, and the fear that it might yet be weak, led the contractors to take unusual precautions to prevent the possibility of any accident happening to the caisson during the sinking operations. For this reason, special appliances were devised for relieving the cutting edge from carrying all the load, and by the use of sand jacks the total weight of the caisson was distributed over the entire bottom area. The manner of using these sand jacks was one of the most interesting features connected with the sinking of this caisson, and merits a detailed description.

The jacks themselves were of very simple construction. The cylinders of the sand jacks had an internal diameter of 31 ins. and were 36 ins. long. They were constructed of ¼-in. steel plate with a 4-in. lap joint, two angles 1½x1½x¼-in. being used to reinforce each cylinder at top and bottom. The piston was a block of yellow pine, 2½ ft. square and 5 ft. long. For a length of 4 ft. at one end the piston was round with a diameter of 29 ins., thereby allowing 1 in. play in the cylinder. The lower end of the piston was reinforced with a 2½x⅜-in. welded iron band. During its operation the piston was attached rigidly to the roof of the working chamber by long screw bolts, and remained there permanently during the entire period of sinking.

In preparing for a drop the first step was to excavate a hole under the piston. The cylinder was first filled about two-thirds full of sand; it was then placed in position under the piston and was blocked up hard against it by means of timbers. While this was being done the caisson was supported on timber blocking under the bulkheads and other points. At the bottom of the sand jack there was a 2-in. iron pipe extending entirely across the cylinder, the center of which was split and opened up to allow the sand to escape. This type had no bottom to the cylinder, the timbers acting as a support for the sand. Another type used had a steel bottom and two 3-in. holes, with sliding cover at each side at the foot of the cylinder. The operation in both cases was the same.

When everything was ready for a drop the timber blocking supporting the caisson was undermined by a water jet, and the full load was taken by the sand jacks. A man was stationed at every jack, and at a given signal (made by the flashing of electric lights) each man turned a hydraulic jet with 60 lbs. pressure into the hole at the bottom of the cylinder, thus washing out the sand, which was caught in canvas bags of uniform size. When a canvas bag was full the lights flashed again, and the water jet was turned off. Another bag was then obtained, and at the signal the jet was again turned on and the bag filled. Each cylinder contained about 16 bags of sand. This operation was continued until the required settlement was obtained. By adopting the signal system and by emptying the sand into bags it was possible to guarantee that the whole caisson was being sunk at a uniform rate, and that there was no reasonable possibility of any part of the caisson being strained due to its being sunk more rapidly than another portion. As a rule, a drop of from 1½ to 2 ft. could be effected at each operation, the recurrence of the operations depending entirely on the nature of the material to be removed. When the drop had been finished the blocking was again placed under the bulkheads to take the load of the caisson, and the holes under the sand jacks were deepened in order that the operation might be repeated. The greater part of the material excavated in this caisson, which was sand, was forced out through blow pipes.

ANCHOR PIERS

No difficult problems were encountered in the construction of the north and south anchor piers and the north intermediate pier. Both anchor piers were constructed on a location south of the existing anchor piers. For the north anchor pier a cofferdam had to be constructed around the foundations, since the foot of the pier was below high-water mark. The south anchor pier was well above high-water mark, so that all excavation was in the dry.

The anchorage girders were embedded in concrete and the first length of anchorage eyebars were set in place, two shafts being left in each anchor pier for connecting up the anchor eyebars of the main anchorage. It is the intention ultimately to embed the bottom section of eyebars in concrete, but this will be deferred until they receive the full dead load stress.

CONCRETE AND MASONRY

The concrete used in the caisson and backing of the piers was a 1:2½:5 mix, by volume, except that used in the working chamber, which was 1:2½:4 mix. The cement was required to pass a tensile test for neat cement of 450 and 540 lbs., for 7 and 28 days, respectively; and for 1 part of cement and 3 parts of sand, 140 and 220 lbs., respectively. For the main piers new quarry cut-stone was used. For the anchor and intermediate pier the specification allowed the use of stone from the old piers. The greater portion of the old stone demolished from the old masonry was consequently used in the construction of these piers. The abutments were not radically changed, it being only necessary to raise the ballast walls and to make minor alterations to suit the new design.

The masonry in the pier shafts consists of grey granite rock-faced ashlar, laid with alternate headers and stretchers and backed with concrete, in which were embedded displacer stones usually about 1 cu. yd in size. Headers were required to have a length of at least 2½ times their width, with a minimum length of 7 ft. Bed joints were ½ in. throughout, while vertical joints were ⅜ in. for a distance of 12 ins. back from the face and could not be more than 4 ins. wide at any point.

All stones in the rounded ends of the main piers were clamped together and connected vertically by dowels. The upper 18 ft. of these piers were built with cut granite backing. About 40 per cent of the stones in these backing courses were made to project up through the course above, in this way giving a very strong vertical bond. The bridge seats proper are built 2 ft. higher than the surrounding upper coping course, and are 4 ft. deep, extending to the bottom of this coping, thus providing heavy stones under the main bearings.

The anchor piers are in plan about 136 ft. long by 29 ft. wide at the bottom, with a batter of 1 in 24, and reduced in section for 41 ft. at the center to a vertical wall 18 ft. thick, thus forming pilasters at the ends, through which the anchor wells are built.

CONTRACTOR'S PLANT

Owing to the importance of the work the contractor spared no effort or expense to provide a plant up-to-date in every respect.

On the north side a large wooden trestle was built around the four sides of the caissons, all supported on piles and cribs. As the current here reaches a velocity of 7 miles per hour, and as there is an average tide of 16 ft. and a maximum tide of 20 ft., it was necessary to have this trestle very strongly built. Platforms extended to the shore from both the upstream and downstream ends of the pier. These platforms carried standard-gage double tracks, which formed loops around the caisson and connected with the concrete plant 600 ft. inshore and located at the foot of the cliff.

The power plant, the dining-room for the "sand-hogs," and the two-story bunk house were also located at the water's edge—just upstream from the pier. All supplies and materials required were received by rail or team at the top of the cliff, about 160 ft. above high-water level, from which point they were delivered by gravity to the concrete plant and to the service tracks at the foot of the cliff. A service elevator was operated over tracks by a cable and a hoisting engine placed at the top of the cliff which made an angle of about 45° at this point, thus affording a connection between the tracks at the top and those at the bottom. A stairway provided means for the men to reach the upper and lower levels. The board of engineers' office was located at the top of the cliff.

At the foot of the cliff were situated the mechanical plants which furnish the power for the various operations. To supply compressed air five "Ingersoll-Sargeant" compressors were employed. Four of these had a capacity of 1,250 cu. ft. each and one a capacity of 2,500 cu. ft. per minute. These compressors discharged into a 12-in. main, from which 7-in. branches led into the two caissons. Each branch was fitted with a gate valve, so that the air could be cut out of either caisson at will. The main pipe was carried in a sluice of running water about 400 ft. long, which kept the temperature of the air down to about 75° F. As a consequence, the temperature of the working chamber rarely exceeded 90° F., although the service shaft (on account of the heat generated by the setting of the concrete around it) generally exceeded 100° F. For this same reason the temperature of the working chamber reached as high as 110° F. when being finally filled with concrete.

The compressors were at first supplied with power from six 100-HP. horizontal boilers. As the work proceeded it was found that the demand on the compressors was greater than was anticipated. As a consequence, an extra 100-HP. boiler was installed, together with one 500-HP., one 75-HP. and one 250-HP. boiler, making a battery of 10 boilers, aggregating 1,075 HP. These boilers were all coupled up, and in addition to the compressor plant supplied power to the power-house, rock crusher and concrete mixing plant. There were also one 100-HP. vertical and two 50-HP. horizontal boilers on the platform near the caissons. These were used to furnish power to six 15-ton stiff-leg derricks, which were used for handling stone, concrete, etc., during the sinking operations. They also furnished power to one 8-in. high-pressure pump used for washing material in the working chamber and to two 4-in. pumps, which supplied water to the high-level tank on the top of the hill, thus furnishing the water supply for the whole plant.

The plant was supplied with electric light from its own power set situated near the boiler-house. It was equipped with a 30-kw. C. G. E. generator, capable of operating 16 arc lights and 100 incandescent lights (16 c. p.). There was also a blacksmith and machine shop in connection, so that all minor repairs to plant and equipment could be made on the job.

The concrete mixing plant was placed just at the foot of the cliff. Half-way up the slope was the rock crushing plant. The rock used for the concrete was obtained from an adjoining cut, and it was brought to the brow of the hill in cars which dumped into a chute leading to the crusher plant. The stone was fed into two gyratory crushers, which had a capacity of about 500 cu. yds. in 12 hours. After passing through the crushers the stone was led over an inclined screen of 2-in. mesh, and thence into a storage hopper bin of about 200 cu. yds. capacity. These chutes led from this bin to the concrete mixing platform below, the mouth of each chute being directly over a mixer. From this platform the sand, stone, cement and water were fed in the proper proportions to the mixers underneath the platform, which in turn dumped into self-discharging buckets on trucks, which were hauled to the caissons by horses. Three "Ransome" mixers were used on the work, two having a capacity of ⅔ cu. yd. and the other 1⅓ cu. yd. Owing to the conditions under which the work was carried on the mixers never had a chance to work to their full capacity; their best day's work being 450 cu. yds. for the 24 hours.

The sand used in the concrete was conveyed to the concrete mixing platform in the same manner as the stone, i. e., by means of a chute from the upper level, where it was unloaded from hopper-bottom cars. The chute was 8 ft. wide by 6 ft. high and was kept practically full all the time, the sand being taken from the lower end as required. The coal for the boilers was also delivered from the upper level through a chute, which emptied into 2-cu. yd. side-dump cars at the boiler-house level. By means of a track these cars delivered the coal to each boiler-house as required. On the top of the coal chute was a double line of rails with balanced trucks, which conveyed the cement from cars at the upper level to the storage shed at the level of the concrete mixing platform. The cars could therefore be unloaded as they arrived and the cement placed where required for use with the minimum amount of handling.

For the convenience of the "sand hogs," who were compelled to work in shifts through the whole 24 hours, the contractor erected both sleeping and dining quarters for a large number of his men. On the lower level a bunk-house had been provided to accommodate about 100 men, with a dining room that would seat as many more. On the upper level was a similar house with bunks for about 60 men and dining quarters of about the same number. On the dock the contractor erected a number of buildings, which included an office and bath accommodation for the inspectors, a hospital with a doctor in continual attendance, where first aid could be administered in case of serious accidents, or regular treatment in case of minor troubles. There was also provided a coffee house, kept at a high temperature, where the "sand hogs" could change their clothes and receive hot coffee at the end of their shift in the working chamber. In addition to the above there were the usual stores, offices, etc., for the contractor's own use. In connection with the hospital arrangements there was also provided a steel hospital tank connected with the compressed air system, to which men suffering from the "bends" could be immediately transferred and treated.

For serving each caisson four 30-in. shafts for material and two 30-in. ladder shafts were employed. For ejecting the sand and smaller stones four 4-in. blow pipes were used. The larger boulders were broken up and hoisted through the material shaft in buckets having a capacity of ⅔ cu. yd. Four 7-in. compressed air pipes supplied air to the working chamber and served the blow pipes. Two 6-in. pipes supplied the water for washing the sand. One 2-in. pipe supplied high-pressure air for drilling, etc., and a second 2-in. pipe carried the wires for the electric lighting of the working chamber and ladder shaft.

As soon as the sinking was completed on the north shore as much of the plant as could be spared was moved to the south side. The men's dining rooms and sleeping quarters were placed on skids, launched into the river, floated across, and placed in position on the other side. The layout for the mixing plant, sand chute, coal chute, etc., was practically the same as on the north side of the river, all the materials being led to the lower level by gravity. The stone for the crushers was quarried directly from the top of the cliff so that one derrick could pick up the stone in the quarry and deposit it in the hopper leading to the crushing plant half-way down the cliff. Although the boiler and compressor plants used on the south side were drawn as much as possible from the north side, they nevertheless had to be increased materially. The steam plant included three 125-HP. and one 250-HP. "Heine" boilers, twelve 100-HP.

locomotive boilers, and seven "Ingersoll-Rand" and "Ingersoll-Sergeant" air compressors delivering to two coupled receivers, from which a pair of 12-in. mains led to the caisson and were carried for about 200 ft. in a wooden flume constantly filled by water. This reduced the high temperature developed at the compressors to about 80° in the working chamber of the caisson. There were also two 12-in. "Worthington" high-pressure pumps, which delivered water to the caisson for the hydraulic jets used for excavation.

On account of the very high tide which prevailed at the site, the air pressure in the caissons constantly varied and was controlled by an operator in the compressor house who adjusted it to correspond with the indications of an automatic register showing a continuous tide pressure.

The stone from the quarry on the top of the cliff was delivered by derricks to a No. 8 "McCully" rotary crusher near the top of the bank, which broke the larger pieces and delivered them through a chute to a No. 5 "Allis-Chalmers" crusher, about 25 ft. below it. The second crusher reduced the stone to a diameter of 2 ins., and delivered it through another chute to a storage bin adjacent to the sand bin. Both stone and sand bins delivered by gravity through gates to measured compartments in a triple-charging hopper just below the floor of the working platform. This hopper was lined with steel and had a compartment into which the requisite number of bags of cement were poured by hand. The hopper gate was operated from the charging platform and delivered all of the aggregate for one batch of concrete to one of the two "Ransome" mixers under the platform, which discharged into 1½-cu. yd. bottom-dump "Stuebner" steel buckets, which were set in pairs on two coupled cars drawn by one horse on a 600-ft. service track to the main pier caisson, or to the anchor pier, where they were unloaded and emptied by the derricks installed there.

The compressed air, with a maximum pressure of 40 lbs. per square inch, was delivered to the working chamber of the south caisson through two 12-in. pipes, as stated above, which in turn was distributed into four 7-in. mains.

Water, at 100 lbs. pressure, was distributed around all four sides of the working chamber in a horizontal main from 4 to 6 ins. in diameter, which was provided in each of the 18 compartments, with a valved outlet and a jet pipe with 1-in. nozzle used to loosen the sand and excavate the earth and gravel. Each chamber was also provided with a 6-in. vertical blow-out pipe and with electric lights. The caisson was fitted with six 3-ft. material shafts, each having a "Moran" air lock with four 3-ft. ladder shafts having simple air locks composed of short upper sections with top and bottom diaphragms and with one large man lock. The latter was a 6-ft. horizontal steel cylinder about 30 ft. long, located on the deck of the caisson, and it was built permanently into the solid concrete of the pier, being approached through a 4x4-ft. vertical stair shaft. The lock was large enough to accommodate many "sand hogs" at once, thus greatly expediting the entrance and exit of each successive shift, thus effecting an economy of air consumption and considerably reducing the waste of lock air.

A hospital lock was also established on the shore near the "sand-hog" house. Under moderate pressures, 100 men worked 8 hours in each shift. As the pressure increased the lengths of the shifts were diminished to a minimum of 1 hour. As many additional "sand-hogs" were required to carry on the work great difficulty was experienced in securing enough men, so that eventually the number of men in each shift was considerably reduced. Some of the men lived in an adjacent boarding house provided by the contractors, but the majority of them lived in local villages up to five miles distant.

At the present time the contractor is at work pointing the joints in the masonry and cleaning these piers thoroughly by a sand blast. There is also some work still to be done on the dressing of the bridge seats. This work is very important and has proved a very difficult operation. These bridge seats are about 32 ft. x 26½ ft. and it is necessary that they should be absolutely level to distribute the load from the main steel pedestal, the base of which is shipped in four pieces. It requires about six weeks to complete the dressing on one of these beds, and it has been found that the work can be done with such accuracy that the variation from a level plane need not be greater than 0.02 in.

PERSONNEL.

This work is under the supervision of the Board of Engineers, Quebec Bridge, which is composed of C. N. Monsarrat (chairman and chief engineer), Ralph Modjeski and C. C. Schneider. The contractors for the substructure are M. P. and J. T. Davis, of Quebec.

Results of an Attempt to Increase Temporarily the Clearance of a Pony Truss Highway Bridge by Springing the Trusses.

(STAFF ARTICLE.)

The importance of bracing adequately the trusses of pony truss bridges is strikingly illustrated by the failure of the highway bridge shown in Figs. 1 and 2. The bridge whose failure is illustrated by these views spans Allen Creek, near Albany, Wis. It is a pin-connected structure with a plank flooring and has a span of about 80 ft. The top chords consist of two channels, cover plate and lacing bars.

Fig. 1. View of Highway Bridge Failure Near Albany, Wis.—Pony Trusses Pried Apart to Give Clearance to Truck Loaded With Forms.

Fig. 2. View Showing Extent of Failure of Pony Trusses—Note That Load Did Not Break Through Floor.

The direct cause of failure of the bridge was the spreading of the trusses. The load consisted of a traction engine and a truck loaded with forms for a temporary building. The forms were loaded onto the truck in such a manner that their required clearance was slightly greater than that of the bridge. To increase the clearance of the bridge the men in charge of transporting the material used planks to pry out the trusses sufficiently to allow the load to pass. The failure occurred when the engine was about in the middle of the bridge. In addition to wrecking the bridge one man was killed and two others were slightly injured.

The failure would not have occurred if the pony trusses had been braced sufficiently to resist an appreciable lateral deflection, as the men then would not have been able to spring the trusses by the use of planks.

We are indebted to Mr. W. G. Kirchoffer, engineer, Madison, Wis., for the views and data contained in this article.

ROADS AND STREETS

Construction and Maintenance of Sand-Clay Roads in Georgia—Methods and Cost.

Contributed by John C. Koch, 1332 North Broadway, Baltimore, Md.

COUNTY ORGANIZATION.

One of the most important factors in the development of improved roads in Georgia has been the use of convict labor. For many years the various county authorities have employed their misdemeanor, or short-term (1 to 2 years), convicts in the construction and maintenance of public roads. But until 1908 the felony, or long-term, convicts were leased by the state to private individuals and corporations at $25 per annum. In that year an act of the Georgia legislature abolished the convict lease system and provided that all able-bodied male felony convicts, about 2,000 in number, should be placed at the disposal of such counties as desired to use them on the public roads.

An elective Prison Commission of three members was created to carry out the provisions of the new law. The duties of the commission include: the allotment of state convicts pro rata according to population, to the various counties making application for them; the inspection of convict camps; the safe-guarding of the convicts' rights, insuring more humane treatment than was possible before; the regulation of the hours of labor and of rest; regulations governing punishment of the convicts for disobedience; and many other details of like character.

In return for the labor of these felony convicts all expense of maintenance, clothing, guarding, medical attention and other expense incident to their use on the roads, is borne by the county using them. Each county is an independent unit and can carry on such improvements as seem desirable without the necessity of outside approval or authority. The executive business of the county is generally administered by an elective board of county commissioners, who serve two or four years. Such a board generally consists of three or five members. The county board may, if it so desires, divide the entire county up into road districts, each of which will be under the personal charge of one of the commissioners. When this arrangement is carried out the convict gang works a certain length of time in each district under the commissioner having charge of that district. Such a system is usually very inefficient, because ordinarily the average county has but one convict gang of from 20 to 30 men, and the constant changing of supervising officers results in confusion.

In the counties having exceptionally good organizations, the direction of all road and bridge construction and maintenance is placed under the charge of a superintendent, who also acts as warden, and is responsible to the county board for the safekeeping of the convicts and for the road and bridge work of the county. Sometimes his duties also include the management of the county poor farm, on which generally there are from 10 to 20 paupers maintained. The poor farm is a tract of several hundred acres, and is often used for the purpose of raising cotton, hay, grain and other food stuffs. These products materially reduce the cost of feeding the paupers and convicts as well as the stock. The convicts usually do all the work necessary in operating the farm.

The superintendent appoints his foremen and guards subject to the approval of the county board. It is found that a day guard of two or three men and a night guard of one or two are required for a gang of 20 to 30 men.

CONVICT LAB R.

One of the great difficulties in road work in the south is an insufficient supply of dependable labor. The free labor that can be secured for road work is usually shiftless and inefficient, and is hard to hold for more than a few weeks at a time. The experience of the south has been that convict labor is quite efficient and economical. This is largely due to the fact that it is entirely under the control of the superintendent; the convicts get substantial food at regular hours, something few of them are accustomed to outside of the chain gang; and regular hours of outdoor labor and of rest are observed, which insures good health. Under competent direction they quickly acquire a fair degree of skill in the use of ordinary road machinery.

Cost of Convict Labor.—Based on a year of 300 working days the cost per day of useful labor is as follows:

Item.	Cost per day.
Food	$0.23
Guards	.20
Medical	.02
Clothing	.035
Miscellaneous	.015
Total average cost per convict per day	$0.50

The above figures are based on a typical 25-convict gang, with two day guards and one night guard at a salary of $30 per month each, with free board, estimated at $15 per month per man. Food is purchased at wholesale and raised in part on the poor farm, the cooking, washing and care of the camp is performed by "trusties." Medical attention is provided at a cost of from $1 to $1.50 per visit. Under "Miscellaneous" is included the court costs charged to the county and all other expense connected with the trial of the convicts. Free labor costs from $1.25 to $1.50 per day, and when most needed in road work cannot be had at all. In addition, the better discipline possible with convict labor increases its efficiency by 10 to 15 per cent.

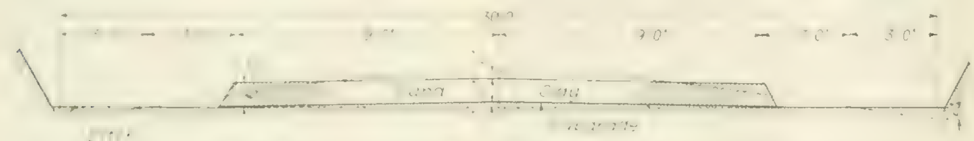


Fig. 1. Cross Section of Surfaced Sand-Clay Road Before Consolidation.

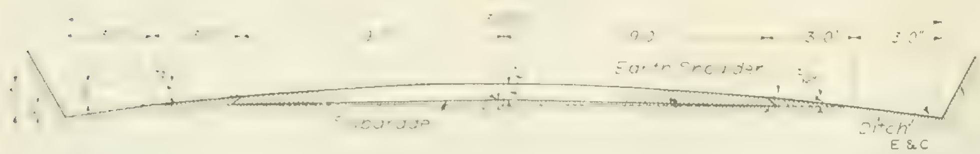


Fig. 2. Finished Cross Section of Surfaced Sand-Clay Road After Consolidation.

SAND-CLAY ROAD CONSTRUCTION.

Sand and clay are the two most widely distributed materials in natural soils. A mixture of the two in proper proportions will produce a surface that will shed water reasonably well, resists the wearing action of traffic to a marked degree, presents a hard, smooth, resilient surface, and is cheap both in first cost and in maintenance expense. In 1909 the mileage of this type of road in 11 states as reported in Bul. 41, U. S. Office of Public Roads, was 13,255. At that time one-half of the total mileage was in the states of Georgia and South Carolina. Since then the growth in mileage has been phenomenal. Ten years' experience with this type of construction in the southern states has demonstrated its economy and suitability under existing climatic and traffic conditions.

The principles underlying the selection of sand-clay mixtures, and methods of analysis of such materials for road construction were fully treated in the Proceedings of the American Society of Civil Engineers, February, 1914. An abstract of that discussion appeared in ENGINEERING AND CONTRACTING, p. 321, Vol. XLI.

Three methods of sand-clay road construction, which include practically all cases encountered in Georgia, will be discussed. The first method, that adopted almost exclusively in the northern part of the state, is characterized by the use of the natural top-soil sand-clay mixture on a clay foundation. The second and third methods are used principally in the middle and southern portions of the state, where the foundation is a sandy loam. In the second method the sand-clay is a natural mixture, and in the third method artificially mixed sand-clay is used.

TOP-SOIL CONSTRUCTION.

In North Georgia there is a super-abundance of clay and the topography is very rough and broken making it difficult to secure reasonable road gradients. The prevailing geological formation is igneous and metamorphic. Granites and gneisses are the more common rock outcrops. The surface soil is composed of decomposed granite or gneiss and consists of a mixture of sand and clay in varying proportions. The ratio of clay to sand content increases with the depth, due to the fact that much of the clay near the surface has been washed out leaving behind the sand particles. The ordinary unimproved roads of this region are clay roads. The material used for surfacing roads is a natural sand-clay mixture occurring as the top-soil of cultivated fields. Hence the name,

Fig. 3. Sand-Clay Roads Surfaced with Top-Soil. (Sand-Clay Mixtures Produced by the Weathering of Surface Soils).

Left [illegible] Upper [illegible] about [illegible] Lower—Six months old, 12 hours after heavy rain. Right—One [illegible] of traffic with several rains.

"top-soil," has been generally adopted to distinguish this type of construction. The method of construction described is that used in Clarke County, which was among the first to use top-soil material.

Method of Construction.—It is assumed that the alignment and grades of the road are satisfactory. The first step is the preparation of the sub-grade, which should be brought to a cross section having a crown of 3 or 4 ins. Sometimes the sub-grade is given a horizontal cross section. The sub-grade is consolidated sufficient by the traffic in a few days, without any delay in the construction operations. The right-of-way of a typical first-class road in this state is only 30 ft., but when high fills or deep cuts are necessary this width is often exceeded.

After consolidation of the sub-grade, a layer of suitable top-soil 10 to 12 ins. thick, measured loose, and 18 ft. wide is spread over the central part of the roadway. This is done in such manner that as the wagons or wheeled scrapers deliver the top-soil it is spread in a smooth, uniform, continuous layer. Experience has shown that if the material is dumped in scattered piles along the road and later spread out the road will be full of irregularities after consolidation, a knob or high point being formed where each load was dumped. On fills the width of the roadway is generally 20 ft.

After about 1,000 ft. of the roadway has been covered with a layer of top-soil, a 4-horse road grader is used to cut shallow ditches at the sides of the 30-ft. right-of-way, the material from the ditches being scraped toward the center of the road to form shoulders, Fig. 4, against the top-soil. By successive passages of the grader, working from the ditches toward the center, the road is brought to the parabolic cross section shown in Fig. 1. The wearing surface of 18 ft. in the center is ample for most country roads and the 3 ft. shoulder between the ditch and the wearing coat is ample protection for the top-soil. The ditches are made 3 ft. wide and 5 ins. deep. Ditches of this size will generally have sufficient capacity, and they can be deepened if necessary. In Fig. 1, the 10-in. layer of top-soil is indicated by the heavy lines, the actual finished shape of the cross section is that shown by the dashed line.

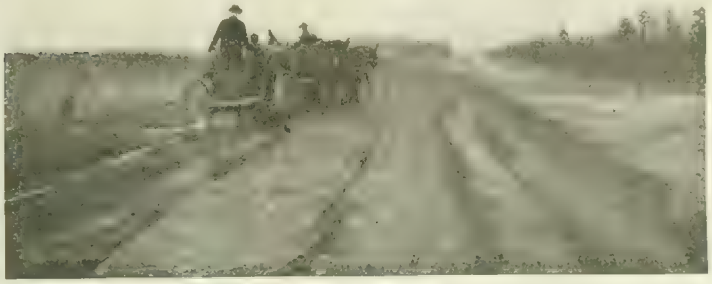


Fig. 4. Forming Ditches and Casting Clay Against Top-Soil to Form Shoulder. Ditch and Shoulder on Right Already Formed.

The crown of the finished top-soil road, after consolidation has been effected is 4½ ins. for the central strip of 18 ft. This has given much better results than the 9-in. crown which has frequently, in other sections, been used experimentally on such roads. The steep crowns cause water to drain from the road surface so rapidly that small depressions are formed at frequent intervals, making the road very rough and uneven. The clay shoulder in time becomes well-incorporated with the top-soil which it holds in place. The 3-ft. shoulder is given a fall of 3½ ins. On grades steeper than four per cent the crown is increased proportionally so that water will drain readily from the road to the side ditches.

Wet weather immediately after placing and shaping the surfacing material, whether it be top-soil or any other sand-clay mixture, aids greatly in consolidating the surface, although it may make the road muddy for several days. The cutting up and puddling action of the traffic at such a time makes the bond between the foundation and the surfacing quite strong, and also mixes the sand and clay thoroughly so that the surface hardens sooner than with dry weather alone.

After the surface has been shaped as described, traffic is allowed to use the road at once. This saves all expense of rolling and the surface becomes compacted as thoroughly as could be effected by any other means. As the consolidation proceeds it is necessary to re-shape the cross section of the road with a 2 or 4-horse grader, at intervals of two weeks for two or three months. This is required because irregularities will appear in the road surface arising from the tendency of wagons to follow the same wheel tracks. For best results these ruts should be smoothed out before the surfacing hardens. If this care is not given the top-soil surfacing during the period of consolidation all irregularities will remain in the finished road and their removal afterwards will be difficult and expensive. The layer of loose top-soil 10 ins. thick eventually becomes compacted to a thickness of 5 or 6 ins.

Maintenance.—There are roads in Clarke County built as described, which have carried a heavy country traffic for four to five years, on which scarcely any repairs have yet been made. The maintenance of this type of road is very inexpensive. The roads are systematically repaired twice a year. The 4-horse grader is used to clean out the ditches and remove the growth of vegetation just outside of the traveled part of the road. At these times the culverts are cleared of debris and repairs to the road surfacing made where needed. The road grader with driver, skilled operator and three convicts will clean ditches and clear off vegetation at the rate of from six to eight miles per day, depending on local conditions.

Repairs to the surfacing are made by a "flying squad" of one guard and eight convicts, equipped with two slat-bottomed wagons and the necessary shovels, picks and other hand tools. The ordinary method of proceedure is as follows: Both wagons start out loaded with top-soil for making the repairs, one on each side of the road. Wherever repairs are needed the top-soil is shoveled from the wagon, enough being placed eventually to pack down flush with the adjacent surface. Consolidation is left to the traffic. As needed, additional top-soil is secured from convenient points. Under usual conditions, such a squad will repair from 6 to 10 miles of road per day, depending upon the distances to top-soil deposits.

Fig. 5. Sand-Clay Roads Surfaced with Natural Sand-Clay. (Sand-Clay Mixtures Derived from Sedimentary Deposits or the Weathering of Rock in Place).

Left—Surface 3 years old. Center—Surface 2 years old. Right—Surface 2 years old

Cost of Construction.—The following cost data have been prepared from information furnished by the superintendent of roads of Clarke County. The salary of the superintendent has been figured at the rate of $1,500 per annum, one-third being charged to the operation of the poor farm and the balance to the road work.

The cost of construction of one mile of the Athens-Jefferson road was as follows:

Item.	Cost.
Top soil (sand-clay), 2 3 acres at $20	$ 46.00
Labor, 25 men 10 days at $0.50	125.00
10 teams 10 days at $1.00	100.00
Foreman, 9 days at $3.60	32.40
Superintendence, 3 days at $5.00	15.00
Total cost of one mile	$318.40

Based on the loose measurement of the top-soil, this mile of road required 3,000 cu. yds. of material. The cost per cubic yard of top soil on this basis is detailed as follows:

Item.	Cost per cu. yd.
Shaping sub-grade	$0.008
Cost of top soil	.015
Plowing	.006
Loading wagons	.023
Hauling (distance av. 1,200 ft.)	.027
Unloading	.003
Spreading	.008
Foreman	.010
Superintendent	.005
Cost per cu. yd. of surfacing	$0.106

The above costs, and other cost data given, are based on the following information: Working day of 10 hours. Cost of food, clothing, guards, medical attention, etc., per convict per working day $0.50. Cost of maintenance per mule per working day $0.50. Daily rates for convicts, mules, foreman and superintendent based on a year of 300 working days, so that these rates include the non-productive expenses incurred by sickness and Sundays.

The length of haul is probably the most variable of the elements of cost. At the rates given above, an approximation of the cost of top-soil construction per cubic yard of material can be made by adding to a fixed charge of 8 cts., ⅙ ct. for every 100 ft. of haul. On this basis the cost of a mile of surfacing where the average length of haul is one-half mile would be $398, and if the length of haul is one mile the cost would be $557.

Cost of Maintenance.—The annual cost of maintenance of 20 miles of top-soil road in Clarke County is given below. Repairs to culverts and bridges are not included.

Repairs to surfacing—	Cost.
Convicts [illegible] days at $0.50	[illegible]
2 teams 5 days at $1.00	10.00
Foreman 1 day at [illegible]	[illegible]
Superintendent 1 day at [illegible]	[illegible]
Cleaning ditches, etc.—	
[illegible] convicts [illegible] days at $0.50	[illegible]
[illegible] teams [illegible] days at $1.00	[illegible]
[illegible] skilled operators [illegible] days at [illegible]	[illegible]
Total for 20 miles	$74.60
Average cost per year for 1 mile	[illegible]

CONSTRUCTION WITH OTHER NATURAL SAND-CLAY MIXTURES.

In Middle Georgia the surface soil is generally a sandy loam, or, less frequently, a mixture of sand and clay. The natural mixtures suitable for road construction are found both as surface soils and as deposits at a depth of from 3 to 8 ft. below the surface. In this region the ordinary unimproved roads are sand or clay roads.

In South Georgia the surface layer of soil to a depth of several feet consists of a fine-grained sand mixed with loam. Underlying this there is generally found a layer of clay containing varying proportions of sand, which is the material generally utilized for sand-clay construction.

Method of Construction.—The sub-grade of the roadway is prepared by shaping a shallow ditch on each side of the roadway with a road grader, the material being drawn toward the center of the road. The remaining portion of the roadway is then brought to a level or slightly crowned cross section. It is now ready for the application of the sand-clay, which is spread in a uniform, continuous layer, 10 to 12 ins. thick over the central 18 ft. of the road, as in the first method described. After about 1,000 ft. of the roadway is covered with sand-clay, the road grader by successive passages shapes the surfacing to approximate the cross section shown in Fig. 1. The consolidation is left to the traffic. At such times as the condition of the surface may require, the cross section is reshaped by the grader. With this class of material it is much easier to re-shape the surface when wet than with the top-soil material used in the first method described. But for best results it is well to re-shape the roadway at intervals of 2 or 3 weeks until consolidation has been effected.

To reduce the expense of hauling surfacing to a minimum, pits of suitable material should be located in advance of construction work, as favorably situated with reference to the road to be improved as possible.

Consolidation is hastened by wet weather during which the road becomes very muddy. If harrowed while in this condition a more thorough mixing is secured and ultimately the surface will be more satisfactory. As soon as the road partially dries out, after each heavy rain if possible, the road grader should be used to maintain the cross section. With such treatment the road will consolidate and present a hard, smooth surface in about 3 months.

After consolidation this type of road may show soft places where there is either an excess of sand or of clay. These may be improved by adding the material deficient, which will correct the trouble.

Fig. 6. Sand-Clay Roads Surfaced with Artificial Sand-Clay. (Sand-Clay Mixtures Produced by the Mechanical Mixing of Sand and Clay During Construction).

Left—Surface 1½ years old. Center—Surface 3 years old. Right—Surface 2 years old.

In rolling country where cuts and fills of some depth have to be made to secure proper grades, it is often possible to utilize the sand-clay mixture from cuts to surface fills. This effects a marked reduction in the cost of construction and is often done. Again, it may be found that the most economical method of construction is to secure the sand-clay from trenches, Fig. 7, several feet in depth just outside the ditch lines and parallel to the road. In such cases it may be an advantage to use the material overlying the sand-clay to raise the roadway. The sand-clay thus secured will cost somewhat less than if hauled from a pit. The cross-section of the consolidated road should be approximately that shown in Fig. 2.

Fig. 7. Securing Clay by Digging Trenches at the Side of Road and Spreading Over Sand Bed in Center of Roadway.

Maintenance.—For proper maintenance this type of road requires frequent attention. The finished shape of the cross section must be maintained to drain well. The clays generally encountered where this type of material is used, are of sedimentary origin and are therefore of finer texture and less resistant to the scouring action of water than the coarser clays found in top-soils of different origin. Also the sands are of smaller grain and less resistant to erosion than those of the top-soils previously described.

The maintenance of the softer varieties of natural sand-clay should be so planned that after every heavy rain, while the road surface is still wet, it will be smoothed with a road drag. The harder varieties will not require such frequent treatment.

The usual type of drag is made of two pieces of 3x8 in. pine or oak, 8 to 9 ft. long, with ¼-in. strips of steel set along the cutting edges, spaced and bound about 30 ins. apart by strong wooden struts. Another type in common use is constructed entirely of metal and has three parallel cutting edges. Both types give satisfactory service.

In maintaining these roads, two drags are used, drawn at such an angle that each shapes a strip of road from 5 to 6 ft. wide. In operation, the two drags first smooth one side of the road; the front one following the edge of the traveled surface and the other travelling near the middle of the roadway. A pair of drags will average from 8 to 10 miles of road per day.

Two or three times a year vegetation is cut and the ditches cleaned with the road grader. In making repairs to the surface it is customary to use the road grader after a rain has softened the surface so that material may be easily shaped. It is not often that new material is hauled for making repairs. At intervals of several years it may be necessary to scarify the old surface and add considerable quantities of new sand-clay and re-shape as before.

Cost of Construction.—The cost of construction of this type of road varies considerably with the length of haul and the amount of stripping necessary. If suitable material from cuts or from shallow trenches along the road can be utilized as surfacing material, the expense of construction is a minimum.

The following is the cost of a mile of natural sand-clay construction in Dougherty County on the Albany-Camilla road:

Item.	Cost.
Labor, 24 men 8 days at $0.50	$ 96.00
9 teams, 8 days at $1.00	72.00
Foreman, 8 days at $3.60	28.80
Superintendent, 3 days at $5.00	15.00
Dressing surface after construction	33.00
Total cost of one mile	$244.80

Convict labor was used and conditions were similar to those described in the cost data previously given. In this instance sand-clay for surfacing was taken from cuts made to reduce grades and consequently the length of haul was low. Part of the surfacing required no haul as it was in cuts where material of a suitable nature was in place after proper grading. The actual quantity of sand-clay hauled was estimated at 2,000 cu. yds. and the average length of haul was 900 ft.

Based on this yardage an analysis of the cost data per cubic yard of sand-clay is as follows:

Item.	Cost.
Shaping sub-grade	$0.008
Plowing	.008
Loading wagons	.036
Hauling	.024
Unloading	.005
Spreading	.007
Dressing	.017
Foreman	.014
Superintendent	.008
Cost per cu. yd. of surfacing	$0.127

Cost of Maintenance.—The roads of Dougherty County are dragged an average of 18 times a year. In addition there is a semi-annual cleaning of ditches, clearing of weeds and making of heavy repairs to the surfacing as needed. The cost of dragging roads averages $0.56 per mile, or $10.08 for the year. Repairs to surfacing, cleaning ditches, etc., for one year on a 10-mile section of road in this county, using convict labor, cost as follows:

Item.	Cost.
Labor, 24 men 5 days at $0.50	$ 60.00
6 teams 5 days at $1.00	30.00
Foreman, 5 days at $3.60	18.00
Skilled labor, 5 days at $2.50	12.50
Superintendent, 2 days at $5.00	10.00
Total for 10 miles	$130.50
Repairs—Average cost per mile	13.05
Dragging—Average cost per mile	10.08
Total average cost of maintenance per mile per annum	$ 23.13

CONSTRUCTION WITH ARTIFICIAL SAND-CLAY MIXTURES.

This method is employed where it is not feasible to use natural mixtures of sand-clay. To determine the proportions of materials to be used to obtain a satisfactory mixture, analyses are made of available materials. From a study of these analyses the proper proportion in which the materials should be used can be readily determined. The details of analysis have been previously referred to.

The basis of an artificial mixture is usually a clay containing some sand but not a sufficient quantity to make a suitable sand-clay surfacing material. This deficiency is made up by adding the required quantity of sand.

The sub-grade is prepared by first shaping the ditches and then bringing the remainder of the roadway to a horizontal cross section with the road grader. A layer of clay 3 to 4 ins. thick is then spread over the central 18 to 20 ft. of the road. Over this is spread a layer of sand of such thickness as to bear the proper ratio to the layer of clay. The unit of comparison of the materials is the number of wagon loads of each kind of material rather than any definite thickness of layer. An attempt should be made to have the materials spread out as uniformly as possible so that the proportions are kept reasonably uniform. An additional layer of clay is then spread over the other layers and a final one of sand, in the same ratio as before. The total thickness of these layers measured loosely will be from 10 to 14 inches.

The sand used for this mixture is usually obtained just outside the ditch lines. Sometimes the road grader will be used to move sand from the middle of the roadway to the ditches, and after the clay is placed on the road this sand is cast back on top of the clay in proper proportions.

Thorough mixture of the sand and clay are best effected when wet, by harrowing and disk plowing. Traffic also assists to some extent in consolidating the mixture into a firm, hard surface. Dry mixing of the sand and clay is not very satisfactory. The surface should be shaped from time to time after each harrowing to the approximate cross section shown in Fig. 2. It may require six months for the road to become well compacted. Here and there soft spots may develop in the surface due either to an excess of clay or of sand in the mixture. These may be remedied by the application of the material deficient, and left to the traffic to consolidate.

Maintenance.—The maintenance of this type of road differs but little from that described for natural mixtures.

[In connection with the maintenance of sand-clay roads constructed by artificial mixing attention is called to the increased danger of "pot holes," or dry weather ruts, which occur more frequently with this type of surfacing than when a natural mixture is used. These ruts occur during extended periods of dry weather and may result from several causes: too large a proportion of sand, irregular mixing, lack of toughness in the clay bonding material, or the thickness of surfacing may be insufficient. These dry weather ruts are usually several inches deep, with vertical sides and characteristic sharp edges. When once started they enlarge rapidly, generally longitudinally. A typical hole of this type may be 4 or 5 ft. long, 5 to 10 ins. wide and 3 to 6 ins. deep. Once started it is quite difficult to prevent the enlargement of these holes while the weather remains dry. After a rain they may be filled by dragging, or new material mixed in the proper proportion added, and in the course of time the point at which the hole occurred cannot be detected.—Editors.]

Cost of Construction.—The following cost data are given for the construction of one mile of the Springfield-Savannah road, Effingham County, Ga.:

Item.	Cost.
Labor, 24 men 19 days at $0.50	$228.00
8 teams, 19 days at $1.00	152.00
Foreman, 19 days at $5.00	95.00
Total cost of one mile	$475.00

Based on pit measurement, the quantity of clay used was about 2,400 cu. yds., and of sand about 300 cu. yds., making a total of 2,700 cu. yds. of material. The detailed cost per cubic yard is as follows:

Item.	Cost per cu. yd.
Shaping sub-grade	$0.004
Stripping clay	.011
Plowing	.004
Loading wagons	.029
Hauling, av. 3,800 ft.	.062
Unloading and spreading	.011
Harrowing, etc.	.008
Sanding road	.009
Dressing with grader	.003
Foreman	.035
Total cost per cu. yd. of surfacing.	$0.176

Cost of Maintenance.—The improved roads of this county are dragged an average of 15 times a year, at an expense of about $0.60 per mile, or a total annual expense of $9.00 per mile.

Other repairs made as required vary considerably in annual charges. The following data are given on a section of road 13 miles long:

Item.	Cost.
Labor, 24 men 4 days at $0.50	$ 48.00
8 teams, 4 days at $1.00	32.00
Foreman, 4 days at $5.00	20.00
Total for 13 miles	$100.00
Repairs—Average cost per mile	7.69
Dragging—Average cost per mile	9.00
Total average cost of maintenance per mile per annum	$ 16.69

Standard Cross Sections for Illinois Roads.

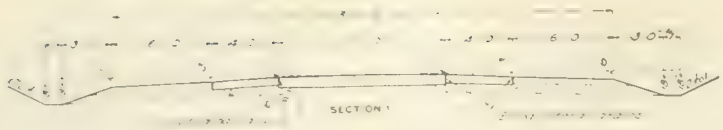
Road construction in progress in Illinois under the supervision of the Illinois Highway Commission is being accomplished according to the most modern methods. This work was planned for the most part by A. N. Johnson, former chief engineer, and is being carried out at the present time by P. C. McArdle, acting chief engineer. The description of the cross sections in use and the reasons for their adoption given here are abstracted from an official publication of the commission.

In general, the sections provide for single and double track roads. A single track road is one on which it is necessary to provide for only a single line of vehicles and most of the state aid roads are of this type. A double track road is wide enough to accommodate two lines of vehicles moving in opposite directions.

Where it is decided that the traffic will be accommodated by a single track road, the paved portion will be 10 ft. wide, if constructed of concrete or brick. This width is sufficient for a single line of vehicles and will also permit two vehicles to pass with only two wheels of one of the vehicles off the paved portion of the road, the other vehicle having all four wheels on the paved road.

Modifications of this section offering a somewhat greater convenience than the single track road alone are the sections which provide, in addition to the 10 ft. paved portion of concrete or brick, a 4 ft. macadam shoulder on either side. It is anticipated, however, that this section at first will be used only on those roads where it would appear that double track roads should be provided, but owing to the limited funds that may be at hand, together with a doubt that might exist as to the necessity for the double track road, it is decided not to build an 18 ft. road. Usually the macadam shoulders will not be provided at first, as they may be placed fully as economically after the road is built and when the traffic needs demonstrate their necessity.

The question frequently is asked, why not build a 14-ft. or 16-ft. road. The answer to this question is, that if we had only to provide for horse drawn traffic 16 ft. would be ample for two lines of traffic, or a double track road. But as the principal roads now carry a considerable proportion of motor traf-



fic, and moreover, as this proportion of motor traffic is fast increasing, it is necessary to take this fact into consideration in the proper design for the road. Experience has proved that to provide safely for two lines of mixed traffic, 18 ft. is the minimum width. It is apparent, therefore, that any width between 10 ft. and 18 ft. is uneconomical because the

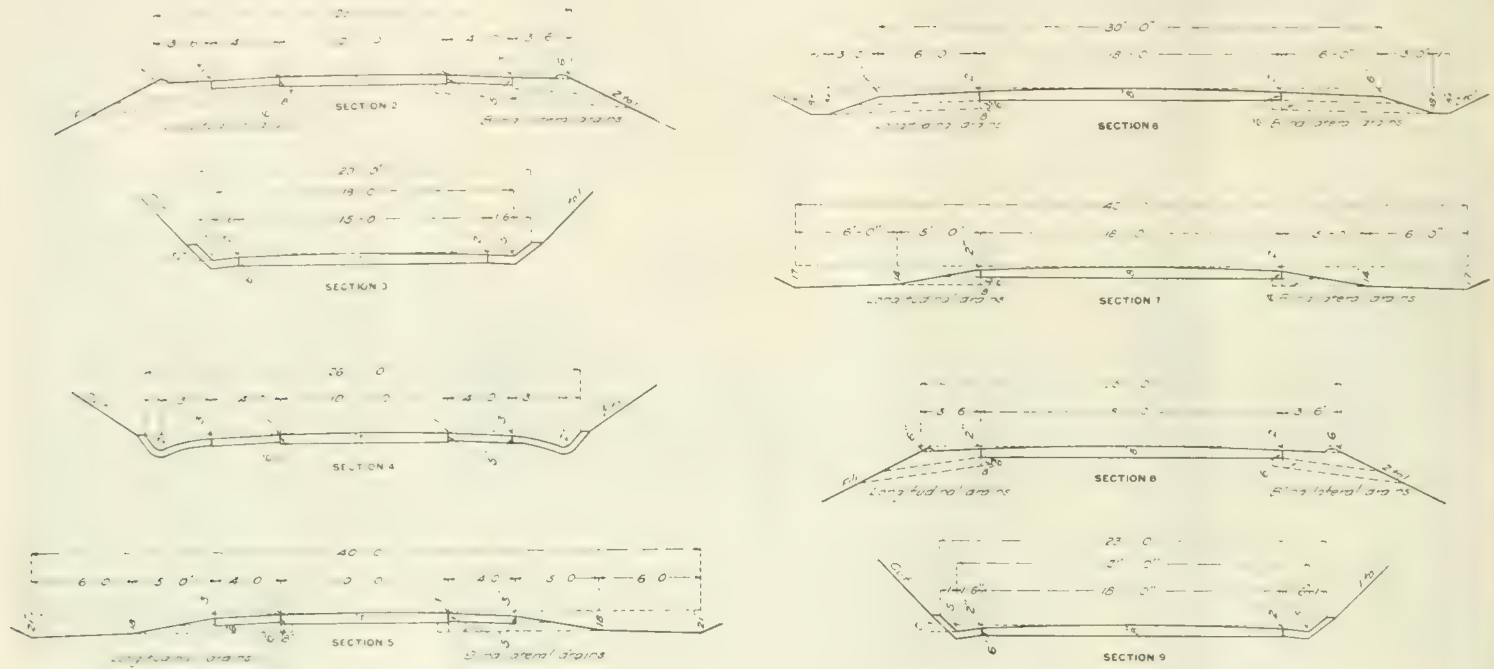


Fig. 1. Standard Cross Sections for Single Track and Double Track Concrete Roads.

Section 1—Ten-foot concrete roadway with 4-ft. macadam shoulders. To be used in a level country. Section 2—Ten-foot concrete roadway with 4-ft. macadam shoulders. To be used on deep fills. For shallow fills use Section 1. Section 3—Fifteen-foot concrete roadway for single track road in deep cuts and on grades that require grouted gutters. Section 4—Ten-foot concrete roadway with 4-ft. macadam shoulders for single track road in deep cuts on grades that require grouted gutters. Section 5—Ten-foot concrete roadway with 4-ft. macadam shoulders and broad side ditches available for traffic where width between fences permits. Especially adaptable for low lying level country. Section 6—Eighteen-foot concrete roadway. To be used in a level country. Section 7—Eighteen-foot concrete roadway with broad side ditches available for traffic where width between fences permits. Especially adaptable for low lying level country. Section 8—Eighteen-foot concrete roadway. To be used on deep fills. For shallow fills use Section 1. Section 9—Eighteen-foot concrete roadway for double track road in deep cuts and on grades that require grouted gutters.

Fig. 2. Standard Cross Sections for Single Track and Double Track Brick Roads.

Section 10—Ten-foot roadway with 4-ft. macadam shoulders. To be used in a level country. Section 11—Ten-foot brick roadway with 4-ft. macadam shoulders and broad side ditches available for traffic where width between fences permits. Especially adaptable for low lying level country. Section 12—Ten-foot brick roadway with 4-ft. macadam shoulders. To be used on deep fills. For shallow fills use Section 1. Section 13—Fifteen-foot brick roadway for single track road in deep cuts and on grades that require grouted gutters. Section 14—Eighteen-foot brick roadway. To be used in a level country. Section 15—Eighteen-foot brick roadway with broad side ditches available for traffic where width between fences permits. Especially adaptable for low lying level country. Section 16—Eighteen-foot brick roadway. To be used on deep fills. For shallow fills use Section 1. Section 17—Eighteen-foot brick roadway for double track road in deep cuts and on grades that require grouted gutters.

traffic either requires the double track road, and therefore the road is too narrow for the traffic; or the traffic does not require a double track, in which case it is wider than is necessary.

The single track roads for macadam construction, which will be used to some extent on the lesser traveled roads in the southern portion of the state, provide 12 ft. for a single track road. The reason for a 12 ft. macadam rather than a 10 ft. is that the traffic on the 12 ft. road will tend to spread itself somewhat more evenly over the surface of the road, and therefore not confine the wear so nearly in one track as would be the case if a narrower road were built. With the hard brick or concrete surface, which will resist the wear, it is not so important that the traffic spread over the road, as in the case of a macadam surface.

Where there are no deep cuts or embankments to be made the cross section in general provides for earth shoulders on either side, flood water may flow over the road surface proper without damage to it, and it will be noted by sections 3, 9, 13 and 17 that the gutter proper is sufficient only to carry a small amount of rainfall, but that the outer edges of the gutter are raised, so that in case of heavy storms the road can care for a very large volume of water.

Where a macadam shoulder is used, or a macadam road is built, it is necessary to prevent the storm water from encroaching upon the road. To meet this condition the gutter must be designed to carry by itself all the storm water, as indicated in sections 29, 30 and 4. Under many soil conditions it is usually found necessary to provide for underdrains in cuts, particularly in deep cuts. Sometimes the under-drainage may be required only on one side of the road, which is usually the case on side hill roads, the upper side of the road requiring the under-drainage.

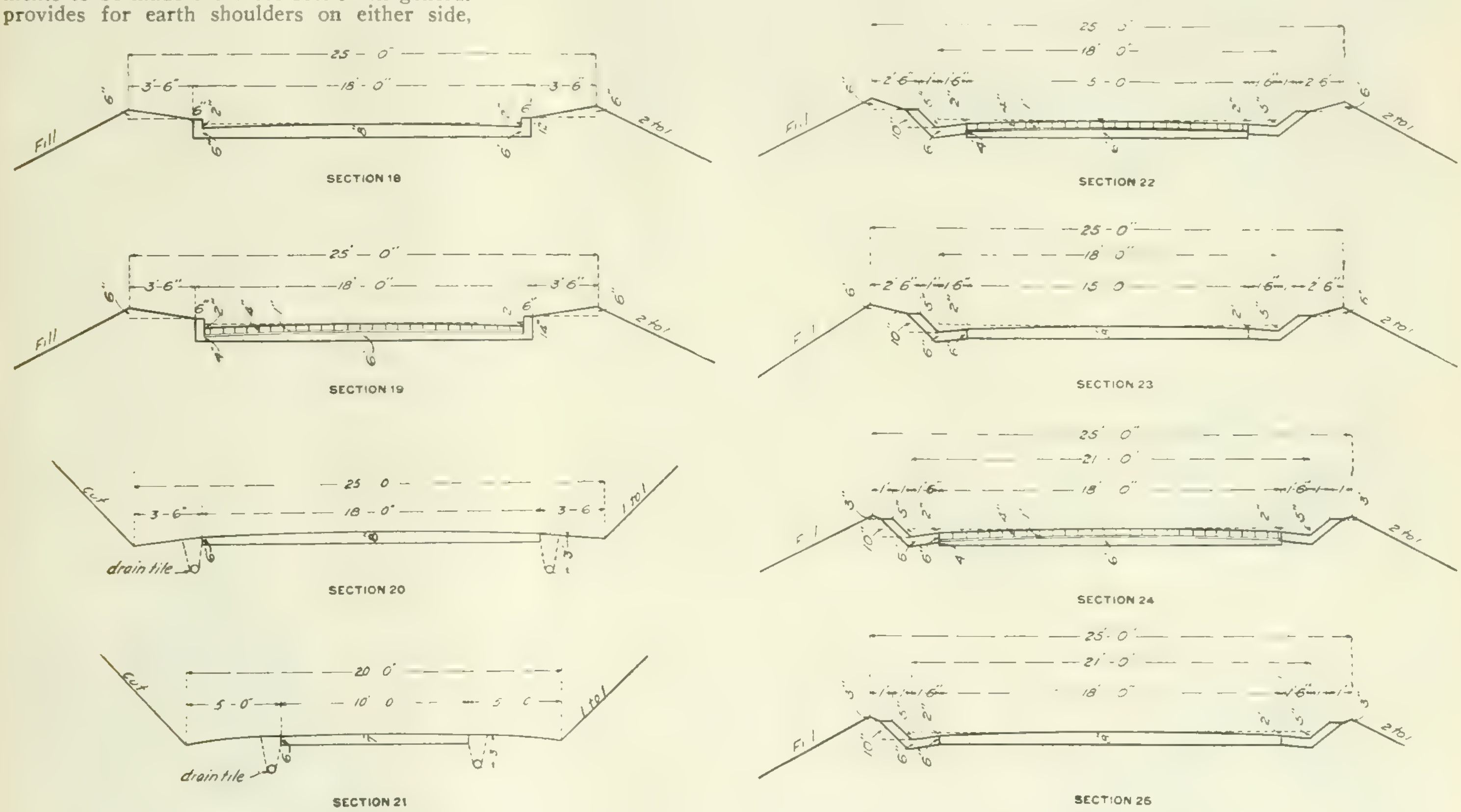


Fig. 3. Standard Fill Sections With Curbs and Gutters and Cut Sections Without Gutters.

Section 18—Eighteen-foot concrete roadway with concrete curb and raised earth shoulders. To be used on fills on double track roads. Section 19—Eighteen-foot brick roadway with concrete curb and raised earth shoulders. To be used on fills on double track roads. Section 20—Eighteen-foot concrete or brick roadways. To be used in deep cuts where gutters are not required. Section 21—Ten-foot concrete or brick roadways. To be used where gutters are not required. Section 22—Fifteen-foot brick roadway with grouted curb and gutter, and raised earth shoulders. Used on fills on single track roads. Section 23—Fifteen-foot concrete roadway with grouted curb and gutter and raised earth shoulders. Used on fills on single track roads. Section 24—Eighteen-foot brick roadway with grouted curb and gutter and raised shoulders. Used on fills on double track roads. Section 25—Eighteen-foot concrete roadway with grouted curb and gutter and raised earth shoulders. Used on fills on double track roads.

wide enough to accommodate vehicles. Where there are deep cuts or embankments it is evident that if they were made wide enough to provide for this side road traffic, the cost would be greatly increased; far beyond the cost which the use of the side roads would justify.

To meet these conditions, special sections have been devised to give a maximum safety with a minimum cost. On steep gradients, where a considerable amount of water must be carried in the gutters, it becomes necessary to provide a paved gutter. Occasionally it is necessary to carry the water from the gutters in a cut on to a fill, although usually the water may be diverted from the road where the gutter runs out of the cut.

To provide for the exceptional condition, where the water must be carried on, the fill sections 18, 19, 22, 23, 24, 25 and 30 are used. The gutters will be constructed of grouted macadam; that is, macadam which will be laid in place and then the crevices filled with cement grout.

Where the road surface is either brick or concrete, it is evident that the exceptional Wherever under-drains are required drains are placed as shown in sections 20 and 21.

In all clay soils, in order to prevent seepage of water under the edge of the pavement, and thereby softening that portion of the subgrade to a greater extent than under the center, there are provided longitudinal blind drains from which lateral blind drains branch to the gutters, the lateral drains being placed every 50 ft. Where the edge of the road is protected by a paved gutter, which will carry the water and prevent seepage under the edge of the pavement, these blind underdrains are not considered necessary.

It is evident that the proper selection of the cross section to use on a given stretch of road requires judgment based on a knowledge of the conditions and general experience in road construction.

The city council of Toronto has adopted a minimum wage scale of 25 cts. per hour for all workmen engaged by the city. The present minimum rate of 18 cts. was established many years ago, since which time the cost of living has greatly increased.

Hard Times and Hard Roads.

To THE EDITORS:—I note with surprise your editorial in this number (July 15) entitled "Hard Times and Hard Roads." The condition described, viz., the dearth of contractors in public road work does not apply to Mississippi, nor to this section of the south. The enclosed clipping gives a good idea of the competition in road work here. Every job, however small, has from "four to forty" bids from contractors and about half that many applications from engineers. A great deal of road work is being done in Mississippi and there are some large contractors on this work.

C. S. WOOD,
Engineer, Road District No. 3.

Columbus, Miss., July 16, 1914.

To THE EDITORS:—Your editorial "Hard Times and Hard Roads" has urged me to reply to your inquiry why no contractors bid on "Hard Roads" when so much work is offered.

The intention of county road superintendents is to get as good a road as possible for as little money as possible. Yet a perusal of many specifications reveals frequent opportunities for the inspector—if one is provided—to permit the contractor to leave work before it is completed; while still complying with the spirit of the specifications.

I suggest uniform specifications and standard width and thickness for all roads falling within a certain classification; Class A, Class B and Class C, for example. These classes to be determined by a traffic census.

The clause usually found in specifications, "Rock to be thoroughly wetted and to be of specified thickness after being thoroughly rolled with a 10-ton roller" should be modified. If I were inspecting a road I would not consider a "thoroughly wetted and rolled road" as being such as easily as I would if I were a contractor. And the question of doing a thing thoroughly might mean to one a certain degree, and to another, a different degree of thoroughness. All of which adds to or lessens the cost.

To be more specific, last week the road commissioners here let two contracts on two sections of road approximately three miles long consisting of: 9,500 cu. yds. earth excavation; 158 cu. yds. plain and reinforced concrete culverts; 14,000 sq. yds. of rock surface, 9 ins. thick; roadway 20 ft. width with 10-ft. rock surface and 5-ft. shoulders each side. Specifications provide for first course 5 ins. thick "after being thoroughly wetted and rolled." Shoulders to be brought to this elevation and thoroughly rolled. Then, second course 4 ins. thick, "after being thoroughly wetted and rolled"; shoulders being brought to the proper elevation to make the required sections.

From experience and study of other roads I find the extent of wetting and rolling required to be a question of personal opinion rather than a study of the actual conditions existing after being thoroughly wetted and rolled. And that if rolling is thoroughly done the voids will be from 5 to 10 per cent of the total space originally occupied by the stone.

In this particular case I based my bid upon rolling until space occupied should be 90 per cent of original space occupied by rock in the quarry—measured by specific gravity of rock and macadam. I then added 10 per cent profit to all labor and arrived at a bid. The following bids were received:

No. 1 (successful bid)	$14,400
No. 2 (my bid)	16,700
No. 3	16,950
Engineer's approximate estimate	14,100

Wondering why I should have made such an error I consulted a representative of a crushed stone company furnishing stone to this section—all stone used here being shipped in by rail. I found that the county superintendent had figures of 90 cents per ton f. o. b. nearest railroad point to work, from a quarry that could not fill a bill of material if ordered. The quarry representative I questioned verified other quarry prices of $1.12 per ton f. o. b., contingent on the non-settlement of the Eastern railroad rate case, before signing contract.

Probably the other high bidder saw conditions as I saw them, i. e., that if required to thoroughly wet and roll this work it would require from 10 to 15 per cent more rock than would be required if this provision was not enforced. The successful bidder on the work was a contractor of experience and good judgment and his bid was probably based on "usual" happenings and inspection.

Probably other contractors look at "Hard Roads" work as I. To do a thorough job requires material and labor and to prepare a bid time and expense—only to find the work let to someone willing to take chances on getting off with less work and material than actually required by strict compliance with loosely drawn specifications.

AN UNSUCCESSFUL BIDDER.

July 20, 1914.

[The editors have repeatedly called attention to the importance of supplying contractors with all unofficial information available as to the probable difficulties to be encountered in accomplishing the work proposed. There is no one factor which will do more to bring about close bidding. It requires time and expense to prepare a carefully estimated bid. The average contractor has no more instinctive ability to estimate correctly from incomplete data than an engineer. Yet he is called upon to estimate correctly the cost of work from data much less complete than that used by the engineer in designing. An engineer likes to have a satisfied contractor on the work he supervises. The contractor will bid lower if he knows his profit is certain.

There are three possible explanations for great variation in bids (omitting of course the plunger whose work is usually unsatisfactory): (1) loosely drawn specifications permitting a wide variation in interpretation; (2) lack of ability on the part of the contractor to estimate correctly; and (3) insufficient information on which to bid correctly and consequent plunging on items in regard to which data are lacking.

It is all very well to say that the contractor should obtain the information desired from other sources, and that the engineer has nothing to do with it, but the fact nevertheless remains that where a wide variation in bid prices exists it is a reflection on the ability of the engineer, either in estimating, or in the quality of the information furnished on the plans, for it is impossible to prepare plans without giving some information. Moreover, a reputable contractor who would frequently bid low on work in order to fill in between jobs, often does not consider it a good business proposition to expend time and money to secure information which is withheld, and consequently plunges.

This condition should not exist. That it does exist is due to the disposition of the average contractor to gamble and the disposition of many engineers to take advantage of this fact to escape slight additional work. It is a situation which can be materially improved by the use of a little common sense on the part of engineers.—Editors.]

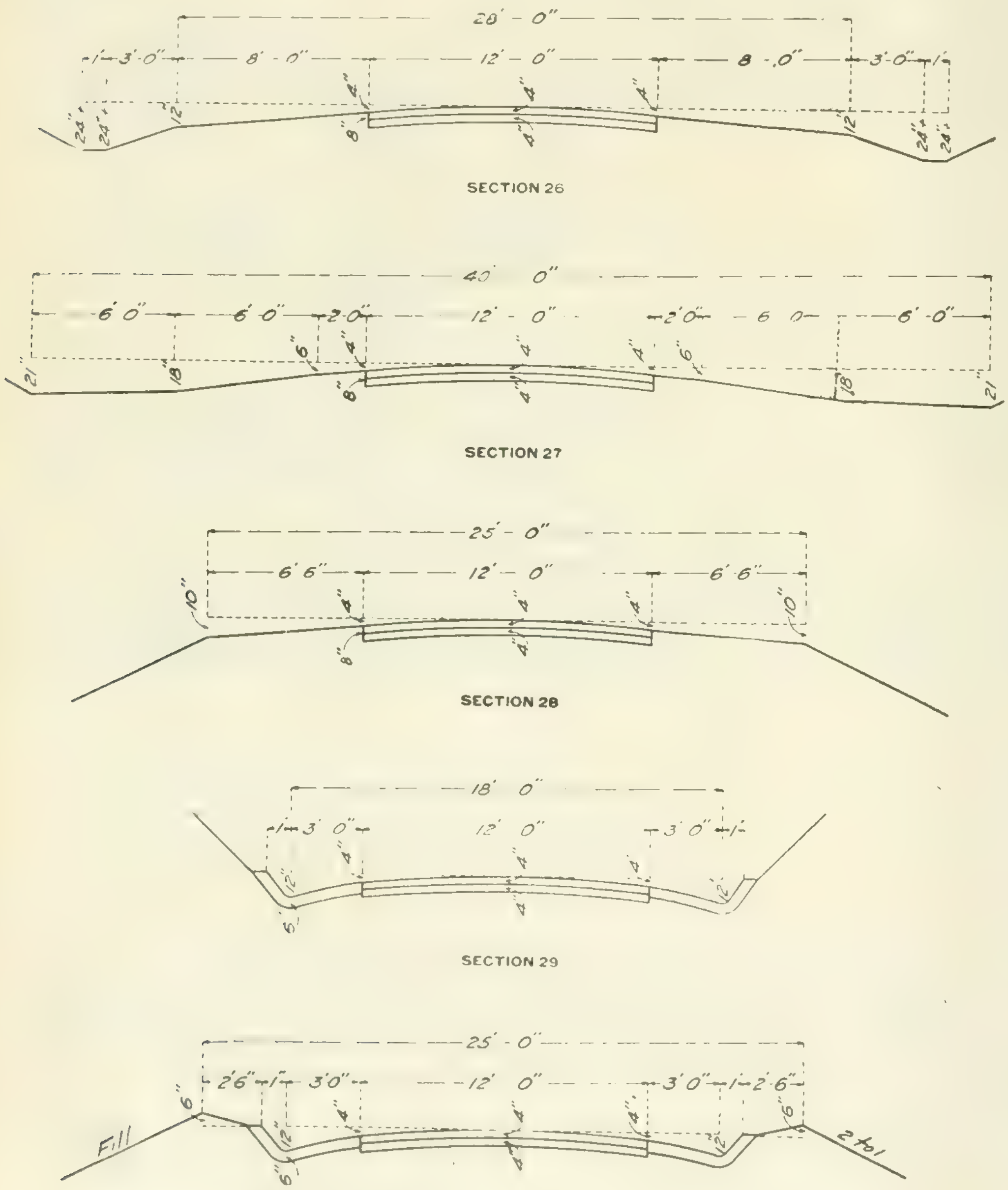


Fig. 4. Standard Macadam Sections for Illinois Roads.

Section 26—Twelve-foot water bound macadam roadway with earth shoulders. To be used in a level country. Section 27—Twelve-foot water bound macadam roadway with earth shoulders and broad side ditches available for traffic where width between fences permits. Especially adaptable for a low lying level country. Section 28—Twelve-foot water bound macadam roadway with earth shoulders. To be used on deep fills. For shallow fills use Section 1. Section 29—Twelve-foot water bound macadam roadway for single track roads in deep cuts on grades that require grouted gutters. Section 30—Twelve-foot water bound macadam roadway for single track roads on deep fills where grouted gutters are necessary.

Instructions to Road Superintendents for the Design of Bridges in Illinois.—The abutments and piers of all bridges shall be of concrete, reinforced concrete, or stone masonry. All footings of piers and abutments are to be placed not less than 4 ft. below stream bed unless rock is encountered. If there is likelihood that the stream bed will scour, piling should be provided or other equally efficient means adopted to protect the abutments and piers. Plans for bridges showing steel tubes or steel legs will not be approved.

All culverts and bridges of 60 ft. span or less should be of concrete, reinforced concrete or construction that will insure equal durability, and no plans for steel bridges of spans less than 60 ft. will be approved, except it is shown the bridge may be of a temporary character.

All bridges must be designed in accordance with the specifications of the State Highway Commission, copies of which will be furnished to all county superintendents.

A Vancouver firm was the successful bidder recently on an $800,000 dredging contract on the north arm of Fraser River. The improvement includes building a 4-mile jetty and dredging a ship channel 300 ft. in width.

SEWERAGE

Methods and Apparatus Used in the Measurement of Run-Off From Sewered Areas.

The Committee on Run-Off from Sewered Areas of the Boston Society of Civil Engineers was appointed on May 1, 1907, and presented its final report on March 4, 1914.

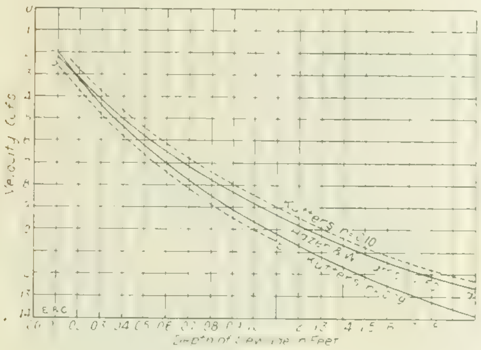


Fig. 1. Velocity Curves for the Ester Ave. Sewer, Pawtucket, R. I., as Obtained from Float Measurements, D = 52 ins.; S = 0.60 ft. per 100. Surface is Cement.

This report, which is published in the Journal of the Society for June, 1914, is divided into three parts, namely, methods and apparatus used in measuring precipitation, methods and apparatus used in measuring run-off showing the relation between precipitation and flow in sewers. The present article relates wholly to the second subdivision of the report. The descriptions of recording gages are quoted from the manuscript of a forthcoming "Handbook of American Sewerage Practice,"

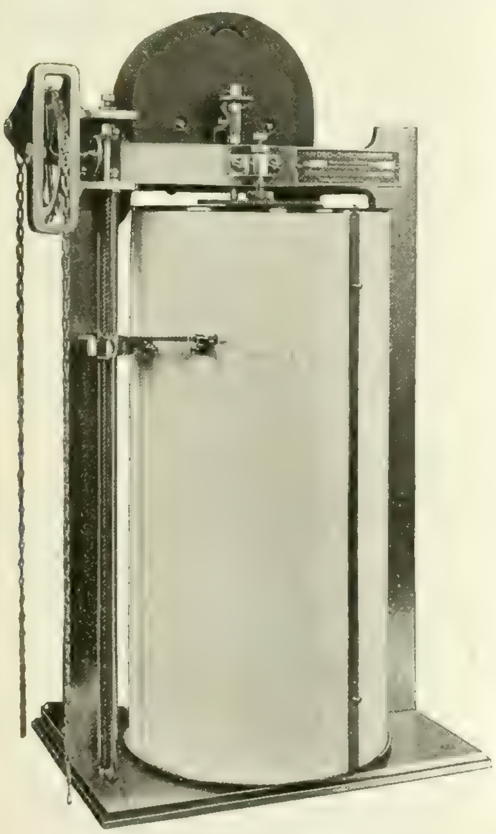


Fig. 2. View of the Hydro-Chronograph.

by Leonard Metcalf and Harrison P. Eddy. The third portion of the original report comprises, in tabular and narrative form, all available records of storm-water flow in sewers.

MEASUREMENT OF RUN-OFF.

A measurement of the actual volume of storm-water run-off in sewers is not usually practicable. Weirs installed in the sewers themselves are objectionable on the score of

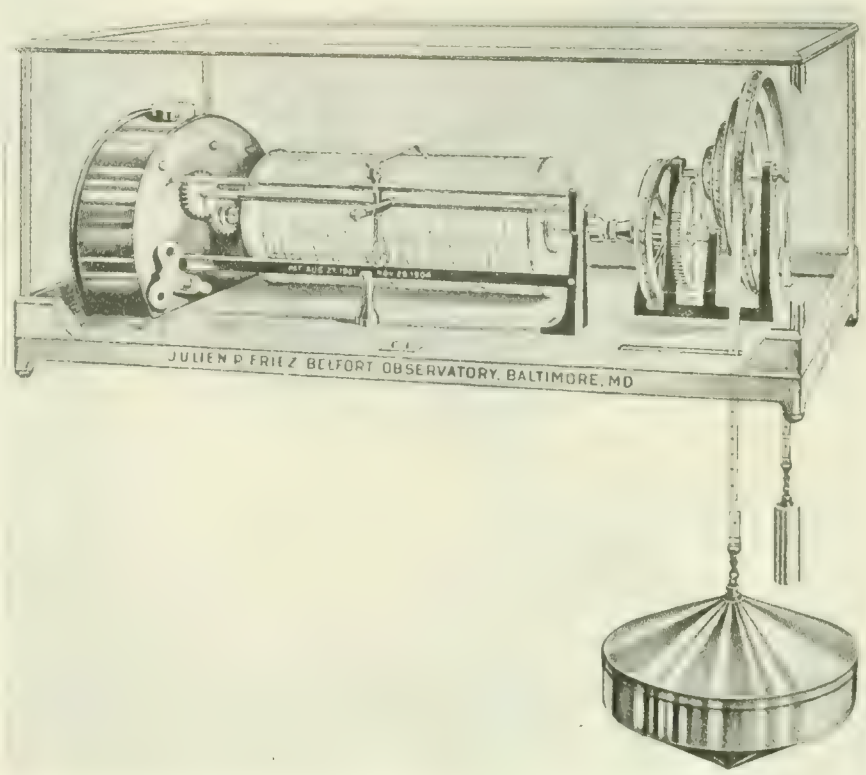


Fig. 3. View of the Friez Automatic Water Stage Register.

the head required and also because they cause a retardation of velocity and retention of sediment; it is also difficult to arrange weirs which give satisfactory results under wide variations of flow, and frequently with high velocities of approach. Venturi meters are expensive if furnished with recorders, which are indispensable in studies of storm flow; they have an insufficient range for measuring the wide fluctuations which are likely to occur; and as they must usually be set in inverted siphons in order to register properly, their installation in sewers already built involves some difficulties. Current meters for continuously recording the flow of sewage are not ordinarily practicable on account of the foreign material in the sewage, which is likely to clog the meter, or otherwise derange it. As a result, gagings, so-called, of storm-water flows in sewers, have almost invariably been made by recording the level of the sewage flowing and computing the quantity of flow, using Kutter's formula, usually with an assumed coefficient of roughness. In order to compute the flow in the sewer from observations of this kind, it is necessary to know the cross-section of the flowing stream, the slope, and the coefficient of roughness. The former can be readily computed from the known or measured cross-section of the sewer, having given the elevation of the surface of the sewage, which is easily obtained from a record of the water level or flow gage. In most observations of this character, the hydraulic slope has been assumed as parallel to the invert of the sewer, and a coefficient of roughness, *n* in Kutter's formula, has been assumed. In many cases these assumptions have probably been wide of the truth.

With regard to the hydraulic grade it has been noted that there are marked differences between the grade of the sewer and the water surface grade.

Many observers have noted this condition. It is, therefore, evident that the use of the grade of the sewer as representing the hydraulic grade may result in serious errors in computing the actual flow in the sewer. It is obvious that correctly to compute this slope two or more water level indicators are necessary, and these must be exactly synchronized so that the true hydraulic slope correspond-

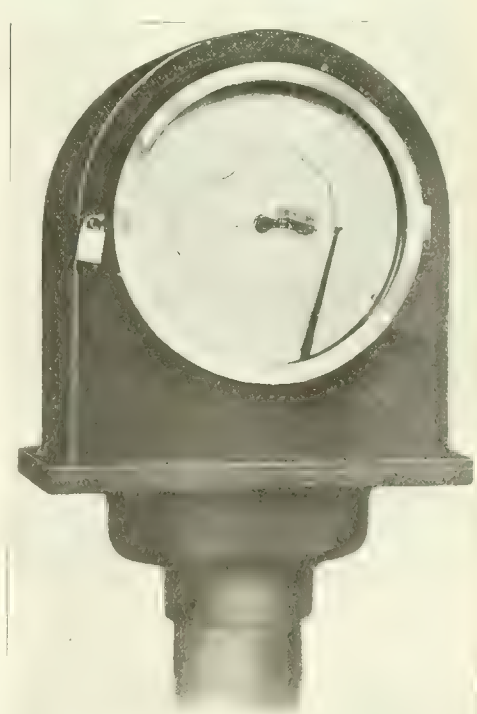


Fig. 4. View of Builders Iron Foundry Water Level Recorder.

ing to the depth at any time can be properly computed. The use of maximum flow gages, indicating merely the maximum height reached by the flood wave at any point, for the purpose of determining the hydraulic slope, is not to be commended, although such gages serve as a valuable check upon the readings of the automatic gages. The crest of the flood wave progressing down-stream

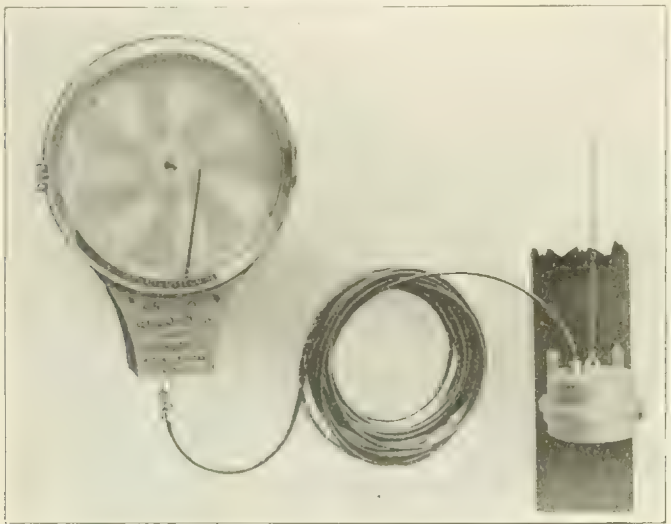


Fig. 5. View of the Bristol Recording Gage.

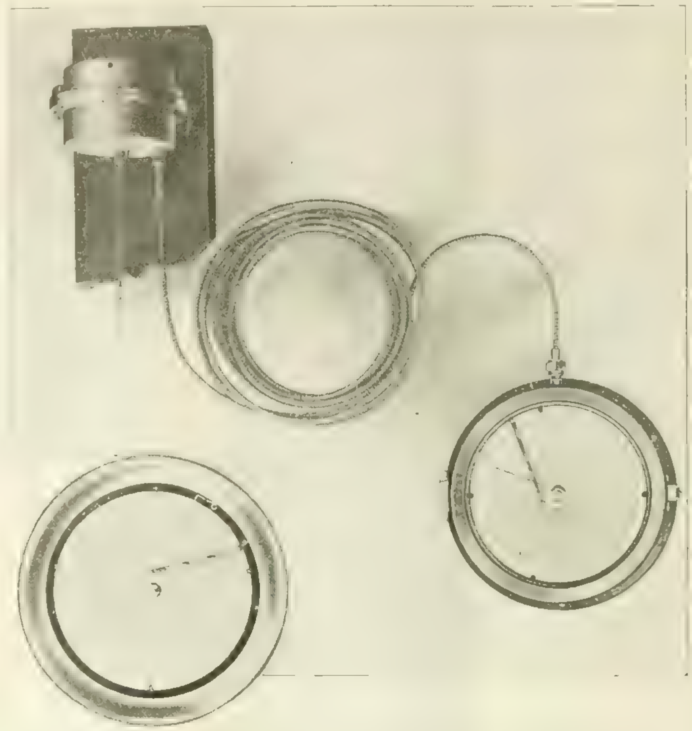


Fig. 6. View of the Foxboro Recording Gage.

leaves its record on each of these gages, but it is obvious that the hydraulic slope is not the slope between the highest point reached at one gage and the highest point reached at the succeeding gage, since the crest of the flood was not at both of these points at the same time.

Wide errors may also be introduced into the estimate of flow by incorrect assumption of the value of the coefficient. It is only necessary to call attention to the fact that the values of n in Kutter's formula for the classes of sewers ordinarily gaged may range from 0.009 to 0.017 in order to realize that assumptions of this coefficient may be far from the truth. These two values, as it happens, have been found by velocity measurements at Pawtucket and Philadelphia to apply to the particular cases referred to; but it is evident that the assumption of such coefficients without experimental determination may introduce serious errors, possibly as much as 50 per cent. Obviously, this method

Fig. 7. View of Sanborn Flow Recorder.

of estimating flow can only be correctly employed when the coefficient is experimentally determined for the sewer under consideration. Either the coefficient of roughness n in Kutter's formula, or the coefficient C in the Hazen-Williams formula, may be determined and employed in the estimation of flow. It is not thought best to use the Chezy formula directly, since the coefficient in this formula would not be constant for varying depths in the sewer and the resulting changes in the hydraulic radius.

In the study of run-off at Pawtucket the following investigations were made in an endeavor to find the value for n in the Kutter formula for the sewer under investigation. The diameter of the sewer was too small to permit of the use of current meters when measuring storm flows. It is seldom that a depth greater than 1 ft. is reached in this sewer, and most of the observations had to do with much lesser depths.

Because of these conditions, floats were tried between manholes 447 ft. apart. The surface slope of the discharge corresponded with the slope of the sewers, as nearly as it was practical to measure the depth of flow, and the slope of the sewer, 0.006, was therefore adopted for the value of s.

The floats used were pieces of wood 3 ins. in diameter and 2 ins. long, and the time taken for the passage of these between manholes was recorded by observers. About 170 observations were recorded, during storms occurring between February, 1905, and February, 1906, and from these data 91 velocities were figured for various depths of flow between 0.16 and 1.1 ft. These velocities have been plotted and a curve drawn which corresponds very closely to the curve of Kutter's formula when using a value for n of 0.0085; see Fig. 1.

As the velocity measured was the surface velocity and therefore, for the shallow depths observed, was very nearly a maximum, it is fair to assume a somewhat lower figure for the average velocity. Mr. Fteley, in his measurements of the flow in the Sudbury River Conduit, found the average velocity there to be about 88 per cent of the maximum velocity, and a velocity curve, cd of Fig. 1, representing 88 per cent of the observed velocity has been drawn.

This curve lies between the curves of the Kutter formula drawn with values, for n of 0.010 and 0.009, but very close to the former curve. It is identical with the velocity curve of the Hazen and Williams formula when giving a value of 150 to c in that formula, [illegible].

With respect to this latter formula, it may be said that c has a range of 145 to 152 when compared with the experiments of Darcy and Bazin in semi-circular conduits of 4.1 ft. diameter, with a surface of pure cement.

The following quotation is taken from W. G. Taylor's article upon a New Main Intercepting Sewer at Waterbury, Conn., which was published in the Engineering Record of April 4, 1908. "Observations upon the sewage flow in the main carrier, at depths up to the springing line, have shown that the value of n in Kutter's formula when applied to the sewer flow is not greater than 0.010." The sewer for which these values were obtained was of reinforced concrete of horseshoe shape, 5 ft. 6 ins. x 4 ft. 5 ins. Great care was used in churning the deposited concrete, and the interior and exterior surfaces are reported as being "very smooth."

TYPES OF RECORDING GAGES.

Leaving Venturi and current meters out of consideration, practically the only type of

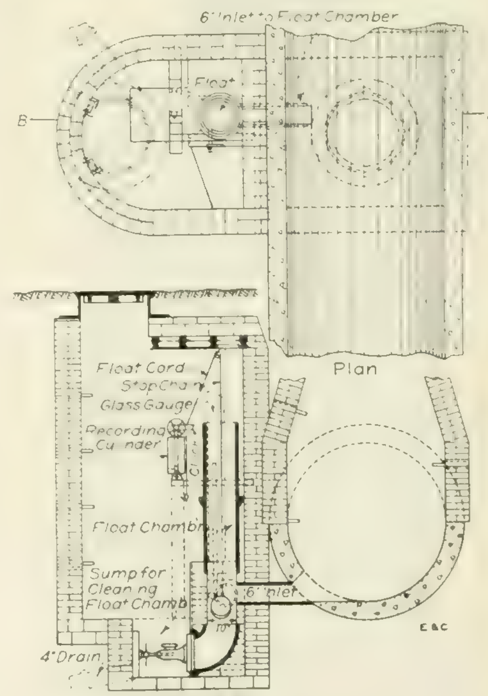


Fig. 8. Plan and Vertical Section of the Gaging Manhole on the Ester Ave. Sewer, Pawtucket, R. I.

automatic gage applicable to gaging storm water flows in sewers is a gage of the water level recorder type. All of the gages available for this purpose may be divided into two general classes,—float gages and pneumatic pressure gages. Either class is equally applicable to keeping a continuous record of the head of water over a weir in case it is practicable to use a weir for accurate measurements of flow.

In order to secure proper registration with any type of gage, it is practically essential to install the float or pressure chamber in a separate manhole connected with the sewer, rather than in the sewer itself. This adds

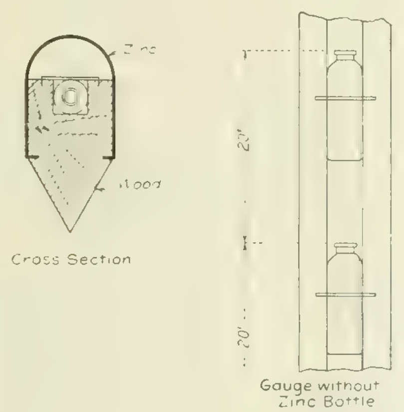


Fig. 9. Details of Gage of Battle Type for Indicating Highest Stage Reached by Flow in Ester Ave. Sewer, Pawtucket, R. I.

materially to the cost of installing the gage and keeping records of sewer flow, but it has not been found practicable to obtain trustworthy records by means of a gage installed directly in the sewer itself.

Float Gages.—In this type of gage a float contained within a pipe or other suitable guide is connected with a recording apparatus through the medium of a cord, chain, tape or by a solid rod or tube.

The Hydro-Chronograph, illustrated in Fig. 2, made by the Hydro Manufacturing Company of Philadelphia, consists of a float and a recorder. The float is connected by a chain with a sprocket wheel at the recorder. The motion of the float is thus transmitted, on a reduced scale, to a pen moving in front of a vertical recording drum, which is rotated monthly, weekly, or daily, as desired. The diameter of the cylinder is such that a time scale of about 1 in. per hour may be employed.

The cylinder is 8 ins. long and 12 ins. in circumference.

The clock can be arranged to drive the pen the length of the cylinder once a week or once a day. In the latter case, the time scale would be ⅓ in. to the hour, and this is the largest scale for which the instrument is regularly made. Sprocket wheels are provided for different ranges in height of water, as follows:

Range of water level.	Total width (circumference of chart).	Scale of heights.
1 ft.	1 ft.	1 ft. to 1 ft.
5 ft.	1 ft.	0.2 ft. to 1 ft.
10 ft.	1 ft.	0.1 ft. to 1 ft.
15 ft.	1 ft.	0.0667 ft. to 1 ft.
20 ft.	1 ft.	0.05 ft. to 1 ft.

List prices of these instruments range from $115 to $150.

The amount of reduction in vertical scale will depend upon the range of motion of the float. This company manufactures a weir gage in which the fluctuations of water level are recorded without reduction; but this can be employed only for a range of about 2 ft. For large sewers it is impracticable to use a drum long enough to cover the range of elevation, without reduction. The list prices of these instruments range from $100 to about $200.

Friez's Improved Automatic Water Stage Register.—This instrument is made by Julien P. Friez of Baltimore. As shown by the illustration, Fig. 3, motion of the float causes the drum to rotate, while the pen is caused to move parallel to the axis of the drum by means of clockwork.

Builders Iron Foundry Water Level Recorder.—In this gage, as illustrated in Fig. 4, the cord from the float moves an arm carrying a pen in front of a circular chart, which is rotated by clockwork. The pen accordingly moves in a circular arc, and the time scale varies with the position of the pen. The instrument is enclosed in a cast-iron box mounted upon a hollow standard through which the float cord passes. It is made in two sizes, having 8-in. and 12-in. dials, and the prices are $75 and $90, respectively; or they can be obtained with an iron outer door for $5 additional.

Obviously, with this gage, the scale of heights as recorded upon the chart will depend upon the range to be covered and the size of the chart. A rectangular chart is not necessary for records of this kind, and the only disadvantage of this form of record is that the time-scale is unduly small when the pen is in its lowest position.

Builders Iron Foundry has also in some cases constructed a modification of the recording instrument of the Venturi meter for use with a float, to indicate and record directly the rate of flow over a weir, and also to integrate these rates and show on a recorder the total quantity passed.

Pneumatic Pressure Gages.—In these gages a diaphragm box or pressure chamber is immersed in the liquid, and the changes in pressure resulting from rising or falling surface are transmitted through a small pneumatic tube to a recording apparatus located at any convenient point.

Diaphragm Gage.—The illustrations, Figs. 5 and 6, show this instrument, which is well known and widely used. It is made for either 8-in. or 12-in. charts, and the prices range from $55 to $80, including 25 ft. of connecting tubing. These instruments are made by the Bristol Company of Waterbury, Conn., and by the Industrial Instrument Co., of Foxboro, Mass. The Bristol recording instrument shown in Fig. 5 is with a seven-day chart. The dark sections of chart represent the night periods. The flexible connecting tube is here shown protected with flexible copper armor. The sensitive bulb may be screwed or bolted to a vertical plank.

Sanborn Flow Recorder.—For sewers, the recorder, illustrated in Fig. 7, may be placed in a manhole, at the sidewalk, or in a nearby building. One-fourth inch copper tubing connects from the recorder to the inlet at the sewer where is located a "compensator," which is a special form of diving-bell. It resembles a piece of tubing, 1½ ins. in diameter, varying in length from a few inches for small sewers to 3 ft. for 20-ft. sewers; it is placed slanting, on an angle of 45° with the vertical, in the direction of flow, and extends to within a few inches of the bottom of the sewer. This compensator is constructed smooth outside and inside so that sewage is not apt to collect. The inlet is at the very bottom. Claims made for this device are, that no float is required, no diaphragm at the inlet, the pressure medium is air and will not freeze, and the recorder may be placed in any convenient location.

The price complete with compensator and 25 ft. of tubing is $75. It is made by the American Steam Gauge & Valve Mfg. Co., Boston.

Installation of Automatic Sewer Gage.—A reliable automatic record of the depth of the storm flow in the sewer is of equal importance with the record of the rate of precipitation, but is even more difficult to obtain. So many difficulties beset the installation of an accessible recording device that it has been very hard to obtain the co-operation of municipal engineers in this work. In sewers less than 4 ft. in diameter and in any sewer where the normal dry-weather flow is of very shallow depth, the installation of a recording device in the sewer itself is apt to produce such an obstruction to the flow as will set up artificial conditions, which make a record of the correct depth of flow impossible.

It is therefore much better to construct an auxiliary manhole, independent of the sewer, for the special purpose of installing the recording mechanism. Such a manhole is shown in Fig. 8. In this manhole a float chamber can be constructed and connected with the main sewer by a small pipe, or pipes, and these need be the only connection with the sewer. Under such a construction it will be possible to visit and inspect the recording mechanism without the inconvenience attendant upon a descent into a regular manhole which is a part of the sewer itself. It will still have the disagreeable feature, however, of being below ground and accessible only through an opening in the street surface. A much better location for the recording device is at the edge of the curb and above the level of the sidewalk. This can be accomplished through a construction similar to a police signal box and the chart will thus be made readily accessible. The only criticism of such a method of installation lies in the necessity of providing some method of accurately checking the chart record with the depth of flow.

Particular care should be taken in connecting the gaging chamber or float chamber with the sewer, to see that the connecting pipe is normal to the direction of flow, and does not project into the sewer. If this precaution is not observed, the recorded heights will be in error—too high if the connecting pipe is directed upstream or against the current, and too low if in the reverse direction. The precautions taken should be the same as in installing a piezometer connection to a water pipe.

It is highly necessary that recording devices be regularly inspected in order that they may be sure to be in operation when most needed, and the more accessible and convenient it is possible to make their location, the more careful attention will they receive. Maximum rates of precipitation and the attendant depth of flow in the sewer are of infrequent occurrence, and it is very essential that the recording device be in operation whenever such a discharge takes place.

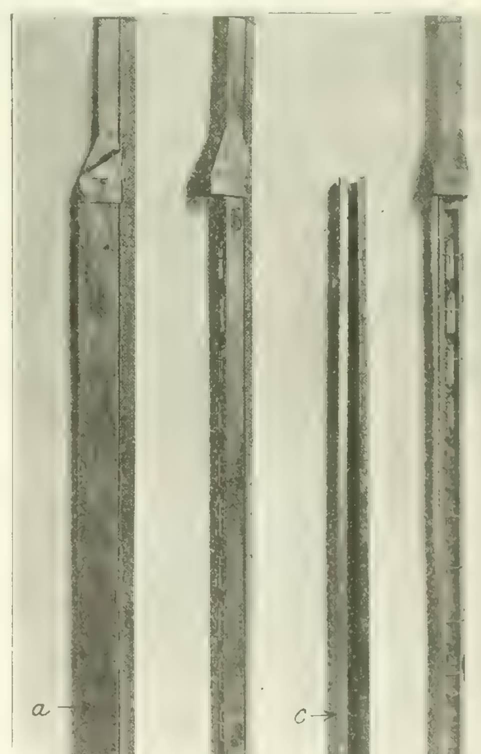


Fig. 10. View of Gage Stick for Measuring Maximum Height Reached by Flow in Sewers, Pawtucket, R. I.

Maximum Flow Gages.—Practically the only information to be obtained from a maximum flow gage is the greatest height reached by the flood wave at the point of observation since the last recorded measurement. Ordinarily the records thus obtained are of little value, but they may occasionally serve as a valuable check upon the records of an automatic gage which may be out of order and fail to indicate the highest point reached by the flood. It is, therefore, advisable to install such maximum flow gages at all points where automatic water level recorders are installed.

In the earlier observations of run-off in sewers, the maximum gages consisted merely of whitewashed laths set firmly in position in manholes, the expectation being that the highest point reached by the sewage would be clearly indicated on the whitewash. In some cases this simple type of gage has proven satisfactory, although in many cases the whitewash has peeled off above the height to which the sewage reached, and in other cases, for some unexplained reason, the maximum height could not be distinguished upon the gage. The most satisfactory type thus far devised consists of a rod to which are firmly fastened a number of small vials having their mouths set at uniform distances apart, usually one-tenth or two-tenths of a foot, the whole being properly protected from the flowing current by a shield or perforated tube. On examining this rod, it is evident that the sewage must have been as high as the highest vial which is found to be filled with water. Inverting the rod and emptying the vials is all that is necessary to prepare the gage for use This type of gage, as used at Pawtucket, R. I., is shown in Figs. 9 and 10. It is best located in a manhole, with the bottom of the gage slightly above the normal dry-weather flow.

In some cases, gages of this type have not proved satisfactory where high velocities have obtained,—such as 8 ft. per second.

CONSTRUCTION PLANT

MACHINES DEVICES MATERIALS

Large Humphrey Pumps for the Drainage of Lake Mareotis at Mex, Near Alexandria, Egypt.

(Contributed.)

An installation of Humphrey pumps has been ordered by the Ministry of Public Works of the Egyptian Government for the drainage of Lake Mareotis at Mex, near Alexandria. When completed this plant will be one of the largest pumping installations in the world, and will consist of 18 pumps each capable of delivering 100,000,000 gals. per day through a lift of 20 ft. The present order includes the first ten of these pumps together with the necessary gas producer plant, Venturi water meters, traveling cranes, locomotive weighbridge, regulating gates and screens, and a complete gas-driven electric light and power installation.

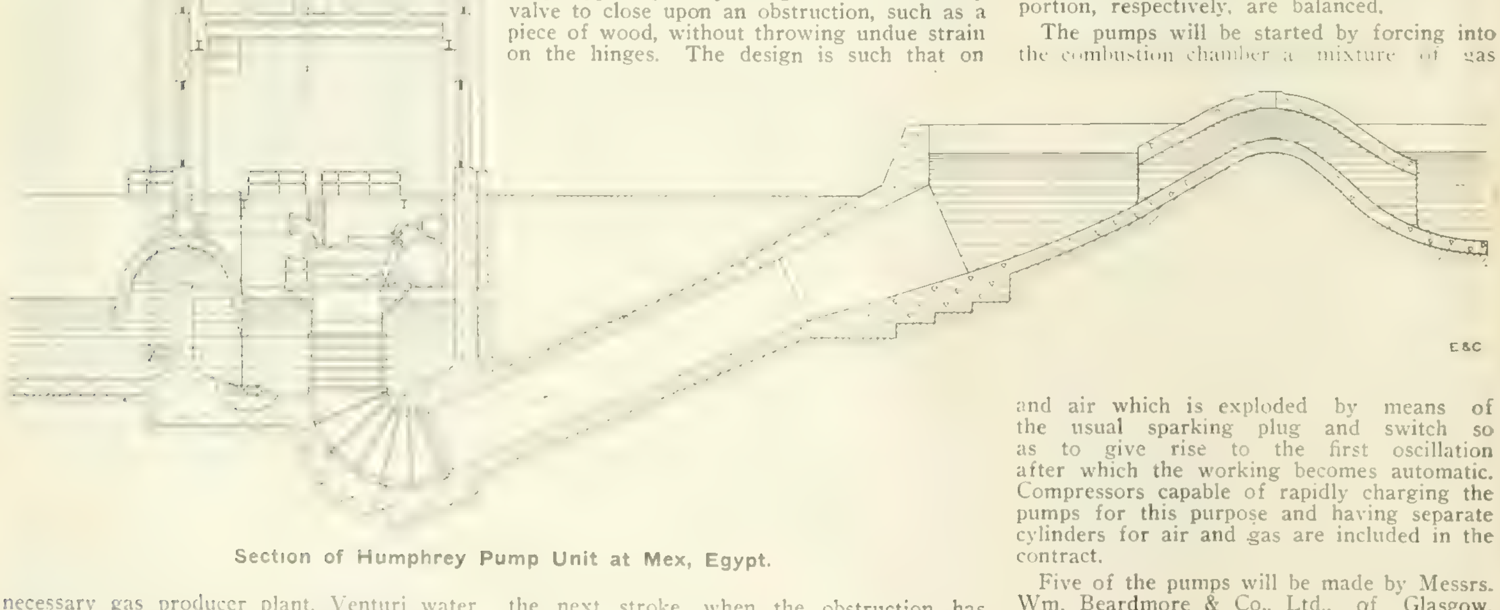
The great size of the pumps may be judged from the fact that each unit is to be capable of delivering between two and three times the amount delivered by each large Humphrey pump unit installed at the Metropolitan Water Board's Pumping Station at Chingford. It will be recalled that the large pumps at Chingford, which were started in operation in March of 1913, had an output of 40,000,000 gallons per day each. The pumps comprised in the present contract have been designed with the aid of the valuable experience obtained with the Chingford Station and include many notable improvements although operating on the same cycle which has proved so successful at Chingford. The following approximate dimensions of the pumps for Mex will be of interest.

General Section of Pumping Plant Arrangement at Mex, Egypt.

The maximum internal diameter of the combustion chamber will be 8 ft. 8 ins. and its height about 14 ft. The water valve box will be 8 ft. 8 ins. diameter and 7 ft. high, and will be fitted with 100 valves of the hinge type especially designed to enable any valve to close upon an obstruction, such as a piece of wood, without throwing undue strain on the hinges. The design is such that on the next stroke, when the obstruction has been removed by the rush of water, the valve will automatically re-adjust its position and close fairly upon its seat. This type of valve is a result of careful experiment and has proved itself thoroughly satisfactory under test

All the main castings will be of steel, and will be carried upon a cast steel bend which connects to the play-pipe. This bend will be in two parts, making up the 118.5° required to connect with the inclined play pipe.

At Chingford the bend had an angle of 90° and connected to a horizontal play pipe, which in turn connected through a second 90° bend to a vertical conical water tower. At Mex, however, there will be only one bend, and the play pipe is sloped upwards so as to deliver the water at the required elevation in the discharge basin. In order to reduce the velocity of the water at the exit, the play pipe ends with a long conical portion giving a 12-ft. diameter outlet. Not only does a sloping play pipe eliminate one of the bends and give rise to a simpler construction, but it has the additional advantage that the vertical components of the forces due to momentum of the moving masses of water in the combustion chamber portion and play pipe portion, respectively, are balanced.



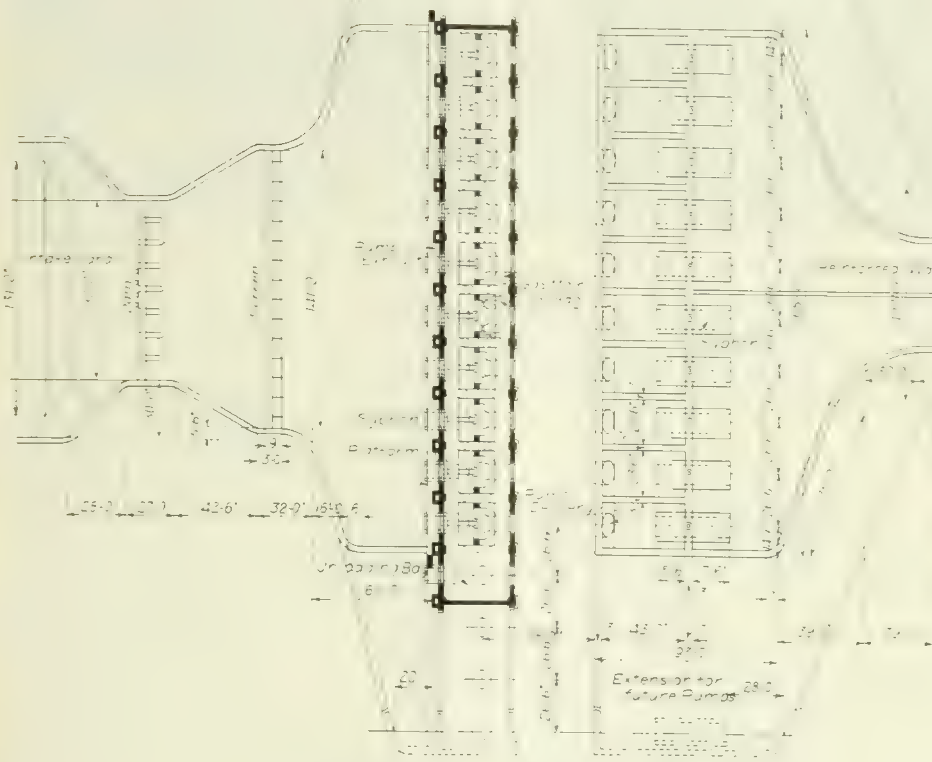
Section of Humphrey Pump Unit at Mex, Egypt.

The pumps will be started by forcing into the combustion chamber a mixture of gas and air which is exploded by means of the usual sparking plug and switch so as to give rise to the first oscillation after which the working becomes automatic. Compressors capable of rapidly charging the pumps for this purpose and having separate cylinders for air and gas are included in the contract.

Five of the pumps will be made by Messrs. Wm. Beardmore & Co., Ltd., of Glasgow, and five by Messrs. The Tecnomasio Italiano Brown Boveri at Milan.

The anthracite gas plant capable of gasifying 44 tons per day will be of the self-vaporizing type, and will be supplied by the Power Gas Corporation, Limited. There will be nine producers, one of which will be a spare.

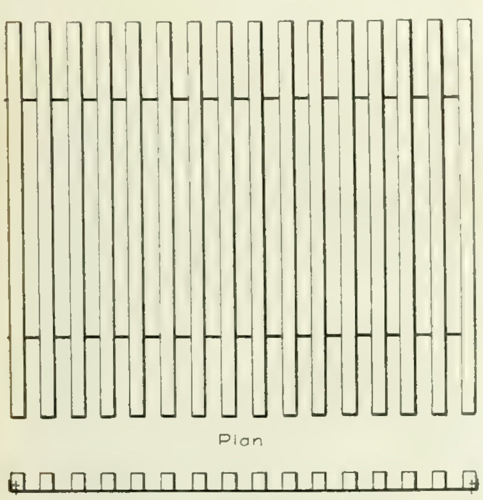
The producers will be of the luted type. The gas from the collecting main will pass through two large gas cooling towers, and then through dry scrubbers into a gas holder of 10,000 cu. ft. capacity which besides forming a mixing chamber for the gas will be used for accurately measuring the gas during the official trials of the plant, although for ordinary purposes the gas will be measured



Plan of Humphrey Pump Installation at Mex, Egypt.

through a rotary meter. The plant will be complete with all auxiliaries and special care has been taken that the plant shall be suitable for operation under the climatic conditions of Egypt.

The plant for electric power and lighting will comprise two gas engines each for 180



Mat Lagging for Floor Forms.

B. H. P. normal load, of the two-cylinder horizontal type, running at 190 r. p. m. and direct coupled to 120-k.w. generators. The current will be supplied at 220 to 230 volts, and the plant will be complete with generator, power and distribution switchboard panels. The lighting will be partly by arc lamps, partly by incandescent lamps, and the capacity of the plant is such that one unit will normally run the whole of the electrically-driven auxiliaries both at the producer plant and at the main pump house. As this plant has to start operation in order to supply current to the large overhead traveling crane, and for working the centrifugal drainage pumps during excavation, it will be supplied with a 200-HP. suction gas producer plant for temporary use, until the main plant is at work.

There will be two Venturi meters supplied by Messrs. George Kent, Ltd., each arranged to measure the water delivered from one or more pumps up to a total of five. These meters will be of square section and 12 x 12 ft. in the throat. Continuous recording apparatus and counters will be fitted to show the rate of pumping and the quantity of water delivered, and will be electrically connected to the recording instruments placed in the pump house. The meters are guaranteed to be accurate within 1¼ per cent. Except that the throat diameters will be lined with gunmetal to ensure smooth and accurate stream lines they will be mostly constructed of ¾-in. steel plates bolted to rolled steel joists and jointed with lead strips. The meters will be partly supported in the natural rock as excavated and partly by concrete walls.

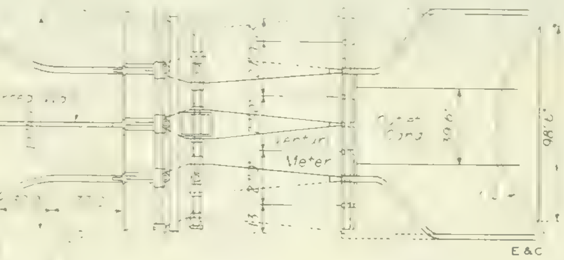
There will be a railway weighbridge placed at the entrance to the sidings for the main installation, having a capacity of 60,000 kilos and made sufficiently strong to take a locomotive up to 75 tons. It will be of the patent extension type manufactured by Henry Pooley & Sons, Ltd., and will carry a ticket printing steelyard. The weighbridge will be a combination of two self-contained machines, one weighing up to 40,000 kilos, and the other up to 20,000 kilos, and the over-all length will be 42 ft.

The electric crane in the pump house will be capable of lifting up to 30 tons and will have separate meters for the two lifting speeds and for longitudinal and transverse motions. The span will be about 35 ft. and the crane will be built and supplied by Messrs. Jos. Booth & Bros., Ltd., Rodley.

There are other items included in the installation which need not be mentioned in this preliminary notice, but it should be stated that the whole of the work will be carried out under the supervision of the well known firm of Messrs. Harper Bros. & Co., who are the Consulting Engineers to the Egyptian Government. They have prepared the specification and plans, and have designed the handsome buildings which will become a landmark in the neighborhood of Alexandria. The Humphrey pump is controlled by the Humphrey Gas Pump Co., Syracuse, N. Y.

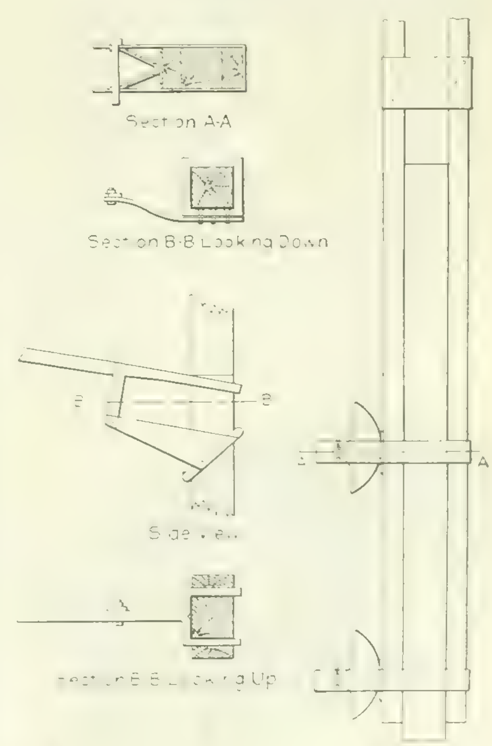
Adjustable Shore and Mat Lagging for Concrete Floor Forms.

The vertical shores supporting the joists and lagging for floor slab and beam centers usually require considerable cutting to fit and finally minor adjustment with wedges. An adjustable shore which has been designed to eliminate both of these tasks is shown by the



accompanying drawing. This shore consists of two 2x4-in. timbers with cap and cleats, as shown, between which telescopes a 4x4-in. timber. Two tightening clamps hold the telescoping posts rigid when they have been adjusted to the length of shore desired. For final adjustment of length the shore jack is used.

The mat lagging consists of a set of parallel bars connected by chains. These mats are of such size that when rolled up one man can



Adjustable Shore for Floor Forms.

carry and adjust one. They are laid on the centering joists and covered by a lagging of steel plates.

These shores and lagging mats are made by J. E. Hodges & Co., 405 Lincoln Inn Court, Cincinnati, Ohio.

Device for Loading Stone, Gravel and Sand from Stock Pile to Wagons.

(Contributed.)

The wagon loader illustrated here is built entirely of steel, and consists of a bucket elevator mounted on a steel truss frame. The truck rolls on two 54-in. diameter and two 28-in. diameter wheels. These large wheels permit the loader to be moved by two men on level ground.

Portable Elevator for Loading Wagons.

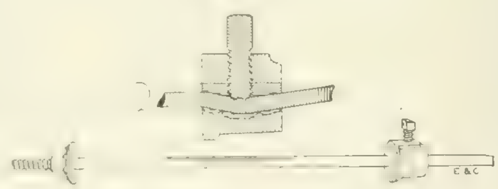
The motive power driving the elevator is either an electric motor or a gasoline engine, and either one of these units develops such an excess of power that if a bank of material should slide on the machine, burying the elevator, the machine will not stall but will dig itself clear. An important feature also is that the elevator can be pushed into the material a distance of about 30 ins. without moving the truck proper, simply by means of a hand lever working a pawl and ratchet. The elevator can also be tilted down so that the machine can be pushed into a bin that has an overhead tie-rod obstruction.

This machine will load broken stone, sand, or gravel, from the ground into wagons at a rate of 1 cu. yd. per minute, and at a cost of less than 1 ct. per cubic yard for electric current or gasoline, and one man can operate the machine. The machine can also be used in sand and gravel pits, to excavate the material, and it can be spouted into wagons, or onto small cars or conveyors. There is a wide field for their use among road contractors, especially when material is delivered by railroad. At such times, a deflecting board is rigged under a bottom dump car before the hopper in the car is open. In this way the material is diverted to the side of the track, where the wagon loader elevates it into wagons.

The over all dimensions of the machine when in an elevating position are 15 ft. long, 12 ft. 1 in. high and 5 ft. 8 ins. wide. The dimensions when the elevating medium is tilted down are 15 ft. 6 ins. long, 8 ft. high. The manufacturers are the George Haiss Manufacturing Co., 141st St. and Rider Ave., New York City.

An Adjustable Tie Rod Clamp.

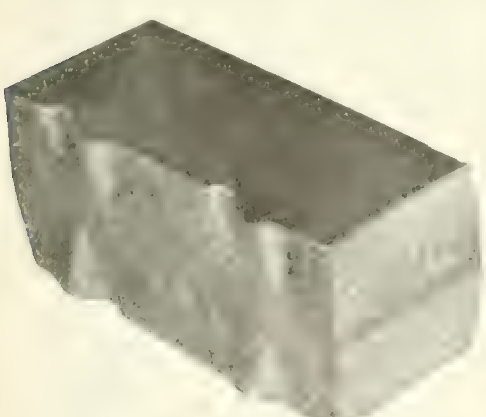
Tie rods for wall or column forms have usually to be cut to length and provided with head and nut or with a nut at each end. The clamp illustrated is designed to eliminate the head or nut at one end and all cutting to precise length. The tie rod is threaded and fitted with the usual nut at one end, then the rod is threaded through the forms and the clamp is slipped onto the projecting end and fastened; the rod is then tightened against the forms by turning the nut. To withdraw the rod the clamp is unfastened and slipped off and the nut end gives a hold with which to pull the rod free. This device is manufactured by the Universal Stamping Co., Ltd., Strasburg, Lancaster County, Pennsylvania.



Adjustable Tie Rod Clamp.

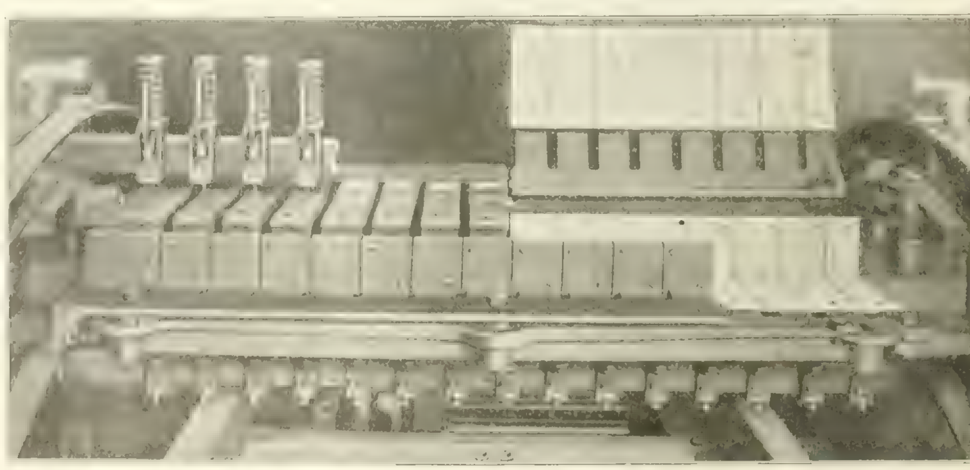
A New Form of Wire Cut Lug Paving Brick and the Machine for Making It.

An improved form of wire-cut-lug paving brick is shown by the accompanying illustration. In addition to the wire cut separating lugs on one side of the brick, one end is molded with a flat wedge shaped end serving to separate the end joints for grouting as the lugs proper separate the side joints. The height of the end wedge or bevel is 1/16 in., and the projection of the side lugs is 3/16 in. The brick itself is 3½x8½ ins. and 40 brick will lay 1 sq. yd. of pavement. The top and ends of the brick are molded smooth but the sides are rough as cut by the wires.



Improved Wire-Cut-Lug Paving Brick.

The machine illustrated here is one for making the new form of brick; as will be seen parts of the platen are broken away to show the shapes of the slots that guide the cutting wires. The column of clay, 9½ ins. wide and 4½ ins. high, passes through the platen to the left end when the cutter is automatically set in motion and the wire frame travels across and through the column, cutting it into 14 blocks. The slots in the platen have offsets, which deflect the wires, thus forming lugs and grooves. The cutter is mounted on a carriage and travels with the column of clay, which is a continuous moving column, while the cut is being made. When the wires clear the column the carriage returns toward the auger for another cut. The top platen is raised while the cutter is returning.



Machine for Manufacturing Wire-Cut-Lug Paving Brick.

The production of wire-cut-lug brick is controlled by the Dunn Wire-Cut-Lug Brick Co., Conneaut, Ohio, which has licensed numerous plants throughout the country to manufacture.

Attachment to Motor Truck to Spread Stone to Even Depth in Dumping.

The operation of a simple catch made by blacksmiths to cause broken stone or gravel to be spread in an even layer when dumped for road work is illustrated here. The catch permits the truck tail gate to swing open only a certain number of inches. It has three steps, one of which spreads the load, with the truck at first speed, to a depth of 6 ins., one to a depth of 8 ins., and one to a depth of 10 ins. This catch was devised for and first installed by a truck mechanic of the two Peerless motor trucks being employed in county road work of Bexar County, Texas. The illustration shows one of the trucks operating on this work. Peerless Motor Car Co., 93rd and Quincy Streets, Cleveland, O.

Attachment to Motor Truck to Spread Stone in Dumping.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLII. CHICAGO, ILL., AUGUST 5, 1914. Number 6.

The Maintenance of Bituminous Surfaced Roads in San Joaquin County, California.

Methods and costs of constructing bituminous road surfaces are well established. Methods and costs of maintaining such road surfaces are not so well established and few accurate data covering them have been published. Perhaps the oldest bituminous work in which a binder, high in bitumens, and undamaged by manipulation was used, is located on the Pacific coast. Here are found natural bitumens possessing important qualities without mechanical treatment. For this reason alone it is natural to expect that bituminous road construction should have been highly developed. Likewise, maintenance methods there should be highly developed for similar reasons. Moreover, traffic and climatic conditions are favorable to this type of construction.

In this issue of ENGINEERING AND CONTRACTING is given a rather complete discussion of the maintenance conditions and methods which exist in a California county. The system has been in operation two years—not long enough to draw definite conclusions. Yet the faults as well as the virtues of bituminous roads have been discussed with a frankness and close attention to detail which should be appreciated by engineers and road superintendents who are struggling with maintenance problems.

The article points out the fact that early maintenance consists largely of remedying imperfections in construction. The covering of excess oil constituted the major part of this maintenance and ⅛-in. to ½-in. stone free from dust was found more satisfactory for this purpose than sand. On an adobe, or clayey soil, it is believed advisable to construct a water bound gravel or macadam road and to allow traffic to compact it thoroughly before placing a bituminous surface. The width surfaced bears an important relation to the maintenance of earth shoulders, which maintenance cost is high; and where there is sufficient reason to contract the width of surface used it is economy to construct a stable shoulder of some firm material. The average cost of one-man patrol maintenance for the first two years of the life of the surface varies from $115 to $125, and the average cost of reconstructing the surface is from $0.145 to $0.195 per square yard. It is demonstrated that a motor patrol furnishes a flexible and economical means for repairs beyond the capacity of the patrolman and not sufficiently important to justify the use of a reconstruction gang. The discussion of these features is not theoretical but is supported by figures and illustrations.

We believe the motor-patrol system to be of great value in the maintenance of bituminous surfaces in rural sections. It possesses the good points of both the gang and one man patrol systems. The force is sufficiently large for all ordinary repairs, whatever their nature; it has a wide radius of operation obviating camping difficulties; and it is economical in operation. On the other hand, the roads traveled should be uniform in surface and the initial cost of equipment is somewhat high.

The importance of preserving data such as that collected by San Joaquin County is worthy of emphasis. The competition between different types of surfacing is so keen at the present time and for certain conditions the advantage and disadvantage are so nearly balanced that the acid test of maintenance cost must be applied. We have made an effort to secure accurate data on this subject and it is hoped that engineers will devote the attention deserved to the collection of other similar data.

Common Errors in Calculating the Present Worth, or Capitalized Value, of an Annual Gain.

The startling number of errors in solving problems of "capitalized value" serves to indicate poor courses in engineering economics at most of our engineering colleges. Recently one of the most striking of these errors was made by a professor of Worcester Polytechnic Institute in an address to the students of another technical school. In that address the attempt was made to demonstrate the value of the annual product of engineering schools. It was stated that the average "technically-trained graduate of our engineering colleges earns annually, on the average, at least $3,000." This was stated to be $2,200 in excess of the average earnings of a "trade-trained man."

The $2,200 annual gain was capitalized at 4 per cent, giving $55,000 as the "increased potential value to the community" resulting from the engineering training received by each engineering graduate.

There are several logical errors in this deduction, the first being that a man's gain to *himself* by virtue of an education has no necessary relation to the gain to *society* at large, for superior ability may result merely in securing for its possessor an unfair share of the products of society. Conversely, superior ability may, and often is, not rewarded in proportion to achievement. Hence personal gain is a rather poor criterion of social gain. This, however, is a matter that might be open to debate, but there is no room for debate on the question of capitalizing at 4 per cent the $2,200 annual gain in salary. That is demonstrably false economics.

Theoretically no annual gain can properly be capitalized at ordinary interest rates unless that gain will continue forever.

Since the duration of the average engineer's earning capacity is, say, 40 years, the present worth of an average $2,200 gain is not $2,200 $\div 0.04 = \$55,000$, but $\$2,200 \div (0.04 + 0.01) = \$44,000$. The 1 per cent is added to the 4 per cent because the 1 per cent gives the amount of the annuity required to amortize the value in 40 years.

Could it be proved that the annual gain of $2,200 is perpetual, so far as society is concerned, it would be proper to capitalize it at, say, 4 per cent. But this is neither subject to proof nor is its proof implied in the method used to derive the $2,200. Clearly, then, the deduction of the $55,000 present worth was erroneous, whether viewed from the standpoint of society or of the individual.

Capitalizing an annual gain at ordinary interest rates, without any consideration of the *functional* life of a plant, is an almost universal error committed by engineers. That we are not speaking too strongly may be easily shown by putting a problem like this to a number of engineers:

What is the capitalized value of the advantage that plant A has over plant B, under the following conditions: The first cost of A is $150,000 and of B is $100,000.

	Annual cost. A	Annual cost. B
Interest at 6 per cent	$9,000	$6,000
Functional depreciation at 4 per cent	6,000	4,000
Operating expenses, including repairs and natural renewals.	4,000	10,000
Total annual cost	$19,000	$20,000

"Functional depreciation" is the loss of value due to obsolescence and inadequacy, and is assumed to average 4 per cent.

In solving the capitalized value of the $1,000 gain of plant A over B, most engineers will divide the $1,000 by 6 per cent, giving $16,667. This is erroneous. The $1,000 gain should be capitalized at 10 per cent (6 per cent interest plus 4 per cent functional depreciation), giving $10,000 as the economic value of plant A over plant B. That this is a correct answer may be tested by adding the $10,000 to the $150,000 first cost of A, and then calculating the annual cost of the $160,000 plant A. The result will then be $20,000, or the same as for plant B, thus proving them to be on a parity when $10,000 (or the $1,000 gain capitalized at 10 per cent) is added to plant A.

By such a problem it is easy to make evident the fact that the functional depreciation rate must be added to the interest rate to get the percentage basis for capitalization. Yet how often has this been done? Wellington's admirable treatise, "The Economic Theory of Railway Location," nowhere shows the slightest recognition of the fact that functional depreciation plays a part in capitalizing annual gains or losses. He capitalizes the saving effected route A over route B at ordinary interest rates, though he himself is careful elsewhere to point out that no economically located road is built for all time. The writer is of the opinion that the average railroad in America has had a functional life of not more than 25 years—wholly regardless of its natural life. Furthermore, it seems probable that the functional life of existing roads will not exceed about 25 years, of such portent are the changes due to electric traction and the rise in terminal land values. If 25 years is an average functional life of railways, then a functional depreciation rate of about 2.4 per cent must be used in all calculations, since annuities based on that rate and compounded at 4 per cent will amortize the entire investment in 25 years.

It is not our purpose now to discuss the overlapping effects of natural and functional depreciation, which are themselves the cause of much confusion among engineers. We desire merely to emphasize the grave danger of capitalizing annual gains at ordinary interest rates.

A Comprehensive Chart for Designing Reinforced Concrete Beams.

A considerable number of diagrams has been platted to expedite the design of reinforced concrete beams, some of which are ingenious and effective as far as their application extends. In the buildings section of this issue we are publishing a chart for use in designing reinforced concrete beams, containing either single or double reinforcement. It is believed that this chart is the first comprehensive representation in graphical form of the formulas in common use underlying the design of reinforced concrete beams containing both single and double reinforcement. The chart can be used to solve the involved expressions which are encountered in the design of reinforced concrete beams, thus reducing the algebraic solutions to a minimum. It is sufficiently comprehensive to enable the designer to solve problems involving all the usual variations of stresses and given conditions and requirements. It should prove of particular value to students of reinforced concrete design, as it shows clearly the effect of changing various elements of the problem.

Another chart by the same author will

be published in the near future for use in designing simple reinforced concrete rectangular beams. This chart will enable the designer to find the required values of b, d and p for any combination of f_s, f_c and v, within the usual range of these values, without additional computations as soon as the maximum bending moments and shears have been determined.

ROADS AND STREETS

Methods and Cost of Road Maintenance in San Joaquin County, California.

Contributed by W. B. H[illegible], Assistant Engineer, Highway Maintenance Department, San Joaquin County, California.

In March, 1909, the citizens of San Joaquin County, California, voted a bond issue of $1,890,000 for the permanent improvement of 240 miles of county highways, shown in Fig. 1. The bond issue was voted under a statute of political code known as the "Savage Act," which provided that any county could bond itself to an amount not to exceed 5 per cent of the assessed valuation of the property, after deducting all other bonded indebtedness. It further provided for a highway commission, consisting of three members with power, through the County Board of Supervisors, to advertise for bids and let contracts for the construction of highways and bridges.

In October, 1909, the first contract was let for 20.18 miles of asphalt macadam road. Additional contracts were let and completed until in January, 1912, approximately 120 miles of road were finished and in need of maintenance.

ORGANIZATION FOR MAINTENANCE.

The organization for maintenance is perhaps unique in that it is probably the first attempt by an individual county to do away with the system whereby the road money of the county is set aside into various road district funds, and its expenditure left to the supervisor or commissioner of the district with little attempt to systematize the work or to keep cost data thereon.

In January, 1912, the Board of Supervisors adopted a resolution providing for the systematic maintenance of all highways completed and to be completed under the bond issue. The resolution specifically stated that the entire system should be maintained as a unit, and funds expended without regard to mileage or assessed valuation of any road district. It further specified that direction of the work be placed under a competent chief, who should have charge of the supervision of the work and accounting of the funds. As the county surveyor was the logical official to take charge of such work, he was empowered to organize the department and purchase all equipment necessary for the prosecution of the work; the system to be so devised and operated that at any time the amount expended on any road could be ascertained.

In February, 1909, the county surveyor proceeded to organize the Highway Maintenance Department of San Joaquin County, the permanent organization of which is as follows: The county surveyor is the official head. Next in charge and directly responsible to him is the assistant engineer who has charge of a second assistant engineer and an accountant. All patrolmen are responsible to the second assistant engineer.

The equipment purchased consisted of heavy machinery and minor equipment necessary to carry on the work. The following is an inventory list of the equipment on hand Jan. 1, 1914.

Item.	Cost.
Bl[illegible]	[illegible]
Heat[illegible]	[illegible]
Road dr[illegible]	[illegible]
Office f[illegible]	[illegible]
Oil [illegible]	[illegible]
Road grade[illegible]	[illegible]
Motor v[illegible]	[illegible]
Oil pots	[illegible]
Oiling machines	2,099.67
Street sweeper	[illegible]
Material wagons	1,037.77
Oil wagons	[illegible]
Water wagons	1,251.73
Equipment under construction January, 1, 1914	106.05
Miscellaneous equipment, including brooms, picks, pouring pots, plows, rakes, shovels, scrapers, wrenches, etc.	557.27
Total cost of equipment for maintenance purposes	$24,126.70

The methods of maintenance instituted were as follows: One-man patrol, gang system, motor patrol, prison labor.

ONE-MAN PATROL SYSTEM.

A patrolman is placed on a monthly salary of $90, and is required to devote his entire time to the roads under his patrol. Each patrolman is required to furnish a horse and a one-horse wagon of low running gear, Fig. 2, capable of carrying at least 2,000 lbs. He is given charge of from 10 to 15 miles of road. The following is a list of equipment necessary for his work as patrolman:

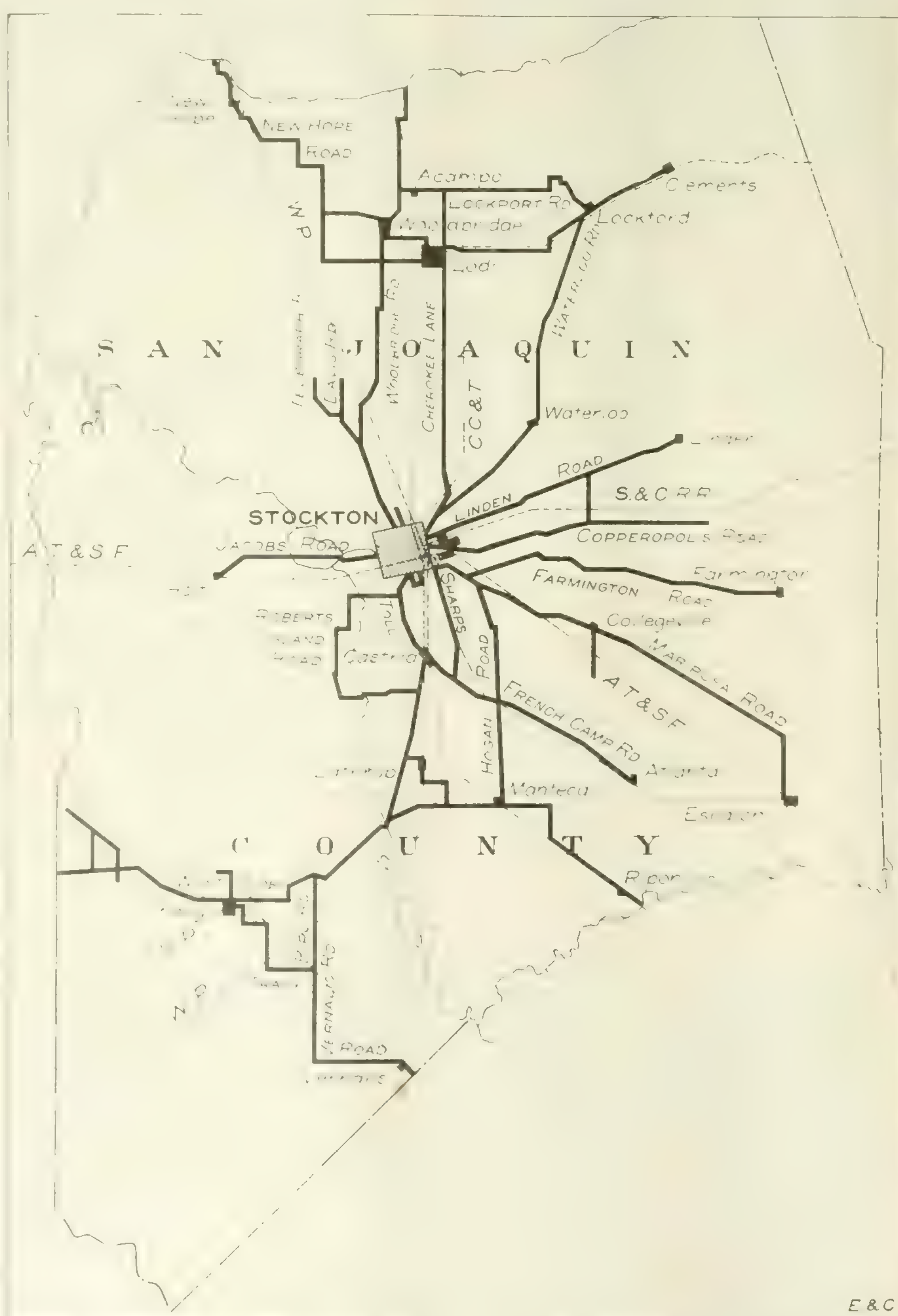


Fig. 1. Map of Improved Highways in San Joaquin County, California.

For storing during the winter months and making necessary repairs on equipment not in use, a corporation yard was provided. For housing purposes a 50x100-ft. corrugated sheet-iron warehouse and a 16x140-ft. open corrugated sheet iron shed were constructed.

Item.	Cost.
1 one-horse wagon (capacity 2,000 lbs., furnished by patrolman)	$40.00
1 25-gallon oil heating furnace	25.00
1 pick	1.00
1 mattock	1.00
1 rake	.85
1 round point shovel	1.00
1 square point shovel	1.00
1 8x8-in. cast iron tamper	1.25
1 8-in. Tarco pouring pot	5.00
Total cost of equipment for one patrolman	$76.10

Fig. 2. Patrolman's Equipment and Method of Patching.

The heavy asphaltic road oil which the patrolman uses in his work of patching, is kept in underground wooden tanks of capacity varying from 150 to 370 bbls., located along railway sidings at convenient points.

Where railway sidings are not convenient to roads under charge of patrolman, small underground pits or tanks having a capacity of 24 bbls. are constructed. The oil for the patrolman's use is delivered to these pits in tank wagons. Stone is placed in piles of three or four tons each, 400 or 500 ft. apart on the edge of the road out of the way of traffic.

The patrolman's chief work in the warm

PATROLMAN'S DAILY REPORT TO HIGHWAY MAINTENANCE DEPARTMENT OF SAN JOAQUIN COUNTY.

ORIGINAL Date..............191..

Name or Number of Road	Mile	Time	Oil

EXTRA LABOR, STOCK AND EQUIPMENT

	No.	Time	Amt.
Laborers			
Horses			
Wagons			

Remarks:

(Patrolman states nature of work he is doing under heading "Remarks")

Signed:

Patrolman

Fig. 3. Daily Report Form Used by Patrolmen.

(Size 4½ ins. by 7½ ins.)

weather of spring, summer and fall consists of keeping the sealing coat of oil macadam highways intact. No attempt is made to use heavy asphaltic oil or do any patching work whatsoever in which oil is required during cold, damp, rainy weather. His procedure in work of this class is as follows:

When he notices a spot or place upon the surface of the roadway where the sealing coat has worn through leaving the rocks exposed, he first cleans the spot thoroughly of all dust and dirt by means of a stable broom, places a thin coat of hot, heavy asphaltic oil over the surface of the exposed rock, and covers the same with rock screenings or fine pea gravel in sufficient quantity to absorb the oil. The heavy asphaltic oil sticks to the surface of the exposed rock and unites along the edges with the old sealing coat where it has not been worn through, forming a patch which seals the surface.

In places where the sealing coat has not been worn through, but depressions have developed, the depression is first cleaned by broom of all dust or fine materials and then given a very light coat of heavy asphaltic oil, after which rock is placed in the depression in sufficient quantity to bring the surface nearly to a level of the surrounding pavement. The size of the rock to be used varies from ¾ to 1½ ins. in size, according to the depth of the depression. The surface of the rock is then given an additional coat of oil and covered with fine material to absorb the same. It is thoroughly tamped, which operation completes the process.

On the waterbound macadam roads he keeps all hollows or depressions filled with cementing road gravel, fine rock or rock screenings. Patrolmen have various other duties, such as keeping the earth shoulders along the edges of the oil macadam highways well drained so as not to allow any water to penetrate under the edge of the pavement to the subgrade, cutting all obnoxious weeds, keeping the roadway clear of rubbish, the waterways open for drainage, etc.

Cost.—The cost per mile per year of maintenance by the one-man patrol system may almost be considered as a fixed sum. For instance, a patrolman has charge of 12 miles of roadway, and receives $1,080 per year; dividing this amount by the number of miles in charge of the patrolman gives a labor cost of $90 per mile per year. From experience we have found the cost of material used will be from $25 to $35 per mile per year. Adding the above unit figures shows us that the total maintenance cost per mile per year varies from $115 to $125.

Reports.—All patrolmen are required to render daily reports, Fig. 3, to the office showing the road, or roads, upon which they have been working, nature of the work performed, and the amount of oil used. A patrolman is not expected to maintain the mileage of roadway given him, exclusive of other assistance. When the condition of the section of roadway is such that it would be impracticable or impossible for him to maintain the same, a larger outfit is employed for the work.

MAINTENANCE BY GANG SYSTEM.

The gang outfit consists of from 12 to 16 men in charge of a superintendent, 14 to 20 horses or mules, and the following equipment:

Items.	Cost.
1 steam road roller	$ 3,250.00
1 portable steam boiler	873.80
1 Tomer (steam pressure) oiler	1,600.00
3 road oil wagons	655.20
3 fuel oil wagons	719.60
4 dump wagons	617.00
2 spreader wagons	370.00
2 stick wagons	260.00
1 rotary street broom	450.00
2 600-gal. water wagons	830.75
2 Road King graders	847.00
1 Austin scarifier	450.00
1 portable centrifugal pumping plant	218.57
18 shovels	18.00
6 stable brooms	6.00
Minor equipment (small tools)	25.00
Total cost of gang equipment	$11,191.10

The gang outfit handles work in which the entire sealing coat of the roadway is in need of replacement or where the surface has become so rough or uneven that it is necessary to reconstruct it. In replacing the sealing coat, heavy asphaltic oil is applied in a fine spray under pressure, over the entire surface of the pavement, and covered with screenings, stone chips or pea gravel in sufficient quantity to absorb the same. The cost of replacing a sealing coat on an oiled macadam road 14 ft. in width, using ¼ gal. of oil per square yard and .026 tons of material to absorb the oil varies from $0.0547

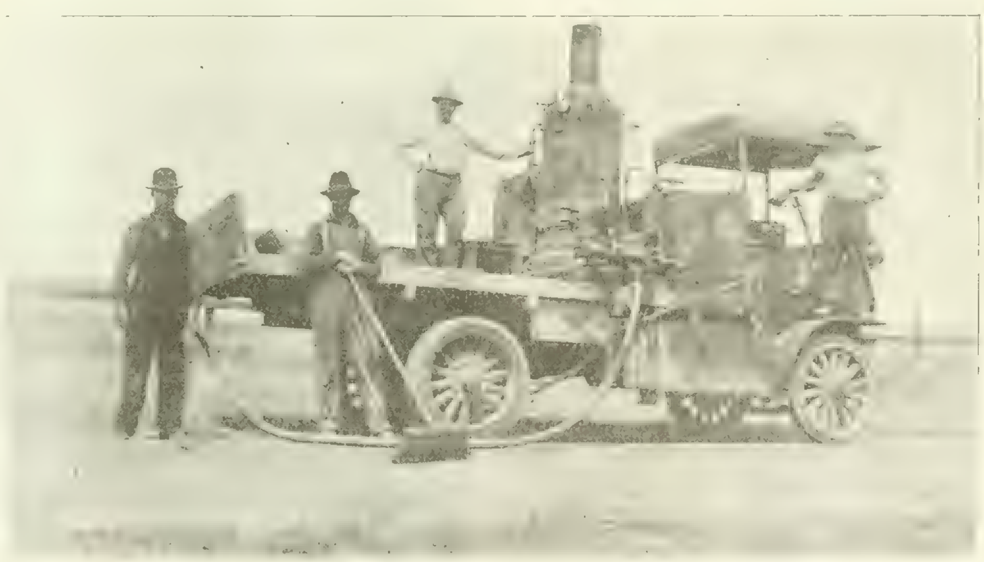


Fig. 4. Motor Patrol Outfit Showing Equipment and Gang.

to $0.073 per square yard, or from $450 to $600 per mile, the chief governing factor being the f. o. b. prices of material and the length of haul.

In places where the roadway has become rough and uneven in order to repair the same the entire surface of the roadway is scarified to a depth of 3 or 4 ins., shaped with a grader and rolled lightly into place, after which the entire surface of the pavement is given a single layer of ¾ or 1½-in. stone. This layer of stone is then thoroughly rolled and forms a surface upon which an additional coat of oil can be successfully applied.

Oil is generally applied in two applications of ½ and ¼ gals. per square yard, respectively, covered with fine material to absorb the same, and thoroughly rolled. By this process depressions and hollows are obviated, and the road after completion presents a smooth and uniform surface.

The cost of reconstructing an oiled macadam road 14 ft. in width in the above manner, using ¾ in. stone for a surface layer, and oiling with ¾ gal. of oil per square yard, varies from $1,200 to $1,600 per mile, or from $0.145 to $0.195 per square yard. Using 1½ in. stone for a surface layer, the cost will vary from $1,800 to $2,400 per mile, or from $0.219 to $0.292 per square yard, the chief governing factors, as before stated, being the f. o. b. prices of material and the length of haul.

It has not been found advisable to place a coat of oil upon the surface of an oiled macadam road which has been scarified without first adding a thin layer of rock, for the reason that the surface of the pavement, after scarifying, has more or less oil incorporated in the fine material which was used to absorb oil at the time of construction, and an additional coat of oil without the addition of a coarser rock material would tend to give too great an amount of oil and fine material over the surface of the upper or wearing course of rock, thereby making it soft and spongy. The sealing or wearing coat should be kept as thin as possible so that tractive resistance, and the tendency to roll or creep, be reduced to a minimum.

The gang outfit has disadvantages as well as advantages. It is a large outfit and involves the expenditure of from $55 to $75 per day. In order that it be worked to the best advantage it must be kept constantly at work, with as few delays as possible, which means that great care must be taken to have material on hand when needed, work planned in advance, and distances of moving made as short as possible.

TABLE I.—ORIGINAL COST, TRAFFIC, AND MAINTENANCE OF IMPROVED ROADS IN SAN JOAQUIN COUNTY, CALIFORNIA.

								Traffic record.—Aug. 25, 1913, to Aug. 31, 1913, inc., 7 a. m. to 7 p. m.					Maintenance costs.		
									Horse drawn.		Motor drawn.				
Number.	Road	Total mileage	Type	Mileage, each type	Width.	Square yards	Original cost	Number of stations.	Total, 7 days	Daily average per sta.	Total, 7 days	Daily average per sta.	Total for year.	Per mile per annum.	per sq. yd. per annum.
*1	Lower Sacramento.	20.18	Asphalt macadam	....	10 and 14	128,700	$213,967.31	5	5,349	153.8	10,079	287.9	5,708.32	$282.87	$0.0443
*2	Cherokee Lane	14.82	Asphalt macadam Oiled macadam	00.65 14.17	14 and 24	125,540	118,159.76	3	3,117	148.4	3,054	145.5	2,789.53	188.22	.0222
3	Waterloo	19.36	Oiled macadam	.	10-12 and 15	134,258	146,368.83	3	2,771	132.0	1,987	94.5	2,666.75	151.31	.0198
4	Linden	13.39	Oiled macadam	.	12 and 15	86,400	134,758.30	2	1,598	114.1	1.063	75.9	1,776,38	115.08	.0205
*5	Copperopolis	8.99	Oiled macadam Asphalt macadam	7.99 1.00	12-16 and 20	93,600	74,726.93	1	955	136.5	653	93.3	753.87	83.85	.0081
6	Farmington	13.81	Oiled macadam Bitucrete macadam	13.19 00.62	12 and 14	102,936	108,396.56	2	660	47.1	869	62.0	2,261.90	163.78	.0219
7	Mariposa	17.75	Oiled macadam	....	12 and 14	152,000	124,944.18	2	728	52.0	722	51.5	2,817.03	158.71	.0185
8	Hogan	9.64	Oiled macadam Gravel	4.31 5.33	10 and 12	71,560	70,856.95	3	1,171	55.7	1,908	90.8	2,230.30	269.05	.0312
9	French Camp	14.08	Oiled macadam Asphalt	13.19 00.89	14 and 20	118,842	120,089.08	3	2,364	112.9	3,753	178.7	2,429.80	161.75	.0204
*10	Roberts Island	8.96	Oiled macadam	....	14 and 16	51,575	53,944.46	1	393	56.1	140	20.0	2,520.63	281.31	.0489
11	Jacobs	6.80	Oiled macadam Gravel	4.15 2.65	14 and 16	34,400	47,078.65	1	367	52.4	207	29.5	996.50	103.51	.0289
12	North Stockton	0.93	Asphalt macadam Asphalt	00.50 00.43	16	87,033	17,752.98	..	.....	.....	.....	.....	43.62	56.99	.0005
13	Homestead	1.70	Oiled macadam	...	16	15,957	19,209.64	..	.....	.....	.....	.....	553.12	326.83	.0346
14	New Hope	14.33	Oiled macadam	...	14	117,967	72,819.18	2	1,071	76.5	721	51.5	2,381.82	166.21	.0202
15	Acampo-Lockeford	8.71	Oiled macadam	..	12 and 15	72,573	54,639.69	3	1,965	93.5	1,474	70.2	1,010.10	115.97	.0139
16	Lodi-Lockeford	6.69	Oiled macadam	....	14 and 16	59,100	30,762.70	2	1,802	128.7	1,070	76.4	1,880.94	281.15	.0318
17	Lodi-Woodbridge	1.67	Oiled macadam	.....	15	14,696	11,586.26	1	1,423	203.3	1,219	174.1	434.20	260.00	.0295
*18	Lodi-Lafayette	6.08	Oiled macadam	.....	10-12 and 14	42,153	38,474.89	1	988	141.1	952	136.0	755.56	124.43	.0179
19	West Side	21.88	Oiled macadam	...	14	179,707	213,243.29	3	758	36.1	4,629	220.4	6,674.77	301.38	.0371
20	Lathrop	3.99	Oiled macadam	...	10-14 and 16	26,133	20,799.44	1	139	19.8	251	35.9	536.53	134.46	.0205
21	Mossdale Br.-Manteca	4.80	Oiled macadam	.....	14-15 and 16	40,893	26,854.45	2	353	25.2	1,113	79.5	1,487.48	309.89	.0363
*22	Manteca-Ripon	8.28	Oiled macadam	..	14 and 15	69,144	48,989.44	1	621	88.7	1,066	152.2	2,685.50	324.33	.0389
23	Tracy-Vernalis	14.49	Oiled macadam Gravel	1.00 13.49	14	119,011	57,661.73	1	157	22.4	57	8.1	1,008.81	69.62	.0084

Note. Entire mileage on Roads Nos. 3, 4, 8, 9, 12 and 19 not under maintenance for entire year.

		Mileage under maintenance, Jan. 1, 1913, to July 1, 1913.	Mileage under maintenance, July 1, 1913, to Jan. 1, 1914.
3	Waterloo	10.83	19.36
4	Linden	11.58	13.39
8	Hogan	4.31	9.64
12	N. Stockton.	0.50	0.93
19	West Side	23.13	21.88

*Described in detail in the text.

MAINTENANCE BY MOTOR PATROL SYSTEM.

The motor patrol outfit consists of a three-ton gasoline truck, Fig. 4, having a bed 14 ft. long, 6 ft. wide and 14 ins. deep. There is mounted on a platform 5½ ft. square and directly in rear of the driver's seat, a 4-H.P. steam boiler, a 250-gal. oil tank, a 100-gal. water tank and a 5-ft. x 2-ft. x 3-ft. steam pump. Behind the platform upon which the boiler, tanks and pump are mounted, there is left 8½ ft. of the bed of the truck in which to carry material. The truck is also equipped for towing a 1,000-gal. tank of oil. The cost

TRUCK DRIVER'S DAILY REPORT TO HIGHWAY MAINTENANCE DEPT. OF SAN JOAQUIN COUNTY.

Date..........191..

Speedometer Reading on Leaving ......

Mileage Traveled

No. Gallons Gasoline or Distillate.. Gal.

Remarks....

Driver Sign Here.

Fig. 5. Daily Report Form Used by Motor Patrol.

Fig. 6. Work Truck Used in Transporting Prison Labor.

of this outfit with minor tools necessary for the work, is as follows:

Item.	Cost.
1 3-ton gasoline truck	$3,922.75
1 patching machine (mounted on truck)	455.06
1 1,000-gal. oil wagon	260.08
4 square-point shovels	4.00
2 stable brooms	2.00
2 cast iron tampers	2.50
1 pick	1.90
Total cost of automobile patrol equipment	$4,387.31

In places where the condition of the roadway is such that it is impracticable for patrolmen to repair, and at the same time would hardly justify repairing with the gang outfit, the auto patrol is used. It requires but four men to operate this outfit, namely: truck driver, oil man and two laborers. The truck driver's duty is to attend to the moving of the truck back and forth along the road when engaged in the application of oil. The oilman attends to the steam boiler and spraying the oil upon the surface of the road. One laborer works upon the truck covering the oil with fine material as soon as it is applied by the oilman. The second laborer works in advance of the truck, sweeping the dust and dirt from all spots to be oiled.

In the use of this outfit it is necessary that all material used in the work be placed along the road in piles in advance of the work. This, however, may be done at seasons of the year when it is impracticable to apply oil.

The method of replacing the sealing coat is essentially the same as that used by patrolmen, with the exception that the oil is applied in a thin spray under pressure. This outfit has proven to be very handy, and the expenditure per day for labor is small, being as follows:

Item.	Cost per day.
Foreman (oil man)	$ 4.00
3 laborers	7.50
Truck expense (including oil, gasoline, tires and repairs, average of 345 days)	2.70
Daily cost of operating motor patrol	$14.20

On light surface work this outfit will patch approximately two miles of road per day. On heavy surface patching one mile per day is a good average.

The motor patrol is able to move rapidly when not engaged in actual work of applying oil, and can carry a sufficient supply of oil in a trailer wagon to last three or four days, without returning to headquarters. No large expense is involved in moving it from one part of the county to another, as in the case of a gang outfit. Figure 5 shows the form of report used by this outfit.

MAINTENANCE BY PRISON LABOR SYSTEM.

Two forces, consisting of from 12 to 18 county prisoners, are employed constantly in maintenance work. Each of these forces is in charge of two deputy sheriffs, one of which directs the work upon the roadway, while the other acts as guard. The following equipment is provided for the two forces:

Item.	Cost.
1 1½-ton gasoline auto truck	$3,401.35
1 2-ton gasoline auto truck	2,840.00
24 square-point shovels	24.00
6 round-point shovels	6.00
18 14-in. rakes	15.30
24 mattocks	24.00
24 picks	24.00
24 stable brooms	24.00
Total cost of equipment for 2 prison labor gangs	$6,358.65

One truck, Fig. 6, is light and fast, capable of traveling from 25 to 30 miles per hour, and works at as great a distance as 30 miles from headquarters. The main work performed by these forces consists of the spreading of screenings, stone chips or pea gravel upon the surface of the oil macadam highways where excess oil has appeared upon the surface. They also work at times in conjunction with the gang outfit in spreading stone and covering oil.

In the winter seasons, when no difficulty is experienced with oil coming to the surface, the forces are kept at work filling depressions in the waterbound macadam roads, opening up waterways of culverts, painting bridges, clearing grass from wings of bridge abutments, unloading material from cars, and various other odd jobs.

One great advantage in these forces is that they are always available on demand. If they were not available, it would be necessary during the summer to employ hired labor for the work which they perform.

MAINTENANCE COSTS FROM JAN. 1, 1913, TO JAN. 1, 1914.

At the beginning of the year 1913 there was in charge of the maintenance department 222.33 miles of completed highways. During the following six months, 19 additional miles were turned over to the department, making a total of 241.33 miles, or the entire system shown in Fig. 1. Table I gives briefly data concerning the various roads of the system and the cost of maintenance for the year 1913.

In order to study and understand this tabulation of cost data, it is necessary to know something of the type or construction of the roads involved, and the character of the maintenance work done upon them. As the type of construction in a great many cases is the same it will not be necessary to describe the method of construction and the maintenance cost of each road. A few concrete examples will be given.

LOWER SACRAMENTO ROAD.

Asphalt macadam ....20.18 miles

In constructing this road the cross section of which is shown in Fig. 7a, the following features were given consideration and methods used:

The road is one of the main arteries leading out of the county. It carries the heaviest motor travel of any road, and the type of construction is higher and more durable than is used on the other roads. The pavement is asphalt macadam, varying in width from 10 to 14 ft. The natural soil, over which a portion of the road passes, is of a heavy adobe nature. On this class of soil, before the placing of the asphalt macadam, a layer of cementing road gravel 5 ins. thick was placed and thoroughly compacted. The soil over which the remaining portion of the road passes is of a sandy loam nature. Upon this section the asphalt macadam was placed directly upon the natural earth subgrade. The thickness of the pavement on sections of the road which pass over heavy soils is 4 ins.; of a sandy loam nature, 3 ins.

The greater portion of the money expended on this road for the year was in the construction of graveled shoulders 3 ft. wide and 4 ins. thick, along the first four miles.

(a) Lower Sacramento Road.
(b) Cherokee Lane Road.
(c) Copperopolis Road.
(d) Roberts Island Road.
(e) Lodi-Lafayette Road.
(f) Manteca-Ripon Road.

Fig. 7. Cross Sections Used on Various Roads in San Joaquin County, California.

(Note: The thickness and width of surfacing varied to fit soil, traffic and topographic conditions.)

Maintaining earth shoulders flush with the edges of the pavement constituted the entire remaining expense.

The highway is one of the through roads over which there is a heavy automobile travel. The pavement being too narrow for vehicles to pass they are required to turn off of the pavement. As a result of this the earth shoulder next to the pavement is worn away, leaving a very noticeable depression, as shown in Fig. 8.

Fig. 8. Surface Failure on a Wet Adobe Foundation.

In the fall and winter this depression is brought to grade at the edge of the pavement with a road grader. This method of maintaining the earth shoulder flush with the edges of the pavement is but a temporary relief, for during the summer months, owing to the lack of rain, the earth shoulders cut into dust, leaving the depression mentioned.

During the year an oiled stone shoulder 2½ miles long was constructed along the edge of the pavement. This work was in the nature of an experiment to ascertain a feasible method of keeping the shoulders flush. The ¾-in. stone was placed along the edges in a layer about 1 in. deep and 2 ft. wide, given a coat of heavy asphaltic oil applied at the rate of approximately 1 gal. per square yard, and covered with fine material in sufficient quantity to absorb the oil. The oil was allowed to lap about 6 ins. upon the edge of the asphalt macadam pavement. The results from this experiment have been very satisfactory. The shoulders are remaining flush with the edge of the pavement, and the oil has made a union with the asphalt so that it is difficult to detect the line of division between the shoulder and the pavement.

Fig. 9. Peeled Surface Due to Poor Penetration.

As shown in Table I, the maintenance cost per mile on this road has been $282.87. This entire amount has been spent on the shoulders in an effort to keep them flush with the edges of the pavement. The figures seem high, but taking into account the fact that the pavement is narrow and the motor traffic excessive, the reasons for the expenditure are apparent.

CHEROKEE LANE ROAD.

Asphalt macadam	.65 miles
Oiled macadam	14.17 miles

This road is one of the main highways connecting the two largest cities in the county. It is not the heaviest traveled road, but carries more than the average amount of traffic, ranking third in horse-drawn traffic, and sixth in motor-drawn traffic. It is one of the best oiled macadam highways in the system. The cross section is shown in Fig. 7b.

A portion of this road 0.65 miles long, adjoining the city of Stockton, was improved with asphalt macadam 24 ft. in width at a cost of $12,972.75. The asphalt macadam was placed 4 ins. in thickness upon a base of cementing road gravel 5 ins. in thickness. The road is wide enough to allow vehicles to pass without turning off the pavement onto the shoulders, and no difficulty has been experienced in the maintaining of the earth shoulders.

The pavement required no actual maintenance work during the year, and at present is in excellent condition.

Adjoining the asphalt macadam a section of oiled macadam, 6.07 miles in length, was constructed over a heavy adobe soil, at a cost of $49,428.58, in the following manner:

After the grade had been prepared and the subgrade excavated, a coat of 80 per cent asphaltic road oil was applied at the rate of ½ gal. per square yard over a width of 14 ft., and covered with the natural earth from the sides of the road. It was then lightly rolled and given an additional coat of heavy asphaltic oil, applied at the rate of ½ gal. per square yard.

This coat was likewise covered with earth from the sides of the road, the object of the oiled subgrade being to prevent moisture from penetrating the pavement proper from the subgrade. Upon this oiled subgrade was placed a layer of 1½ in. stone 10 ft. in width and 4 ins. in thickness, feathering out on each side for a distance of 2 ft. This course of stone was thoroughly rolled and filled with screenings in sufficient quantity to fill all voids, and thoroughly bond it. Upon this course was placed a wearing surface of 1½ in. stone 2 ins. in thickness and 14 ft. in width. This course was thoroughly rolled, and the voids filled, leaving the uppermost surface of the rock exposed to receive oil. Two applications of heavy asphaltic oil were applied at the rate of ¾ and ½ gal. each per square yard, each coat being covered with screenings and thoroughly rolled.

The maintenance work upon this section of the roadway has consisted almost entirely in the covering with fine material, the excess oil which has appeared upon the surface. Some maintenance work has been done by the patrolmen in charge of this section.

The remaining section of this road, 8.10 miles in length, was constructed upon the natural earth subgrade at a cost of $49,277.40.

The rock was placed 4 ins. in thickness and 14 ft. in width, thoroughly rolled and the voids filled with rock screenings, leaving but the uppermost surface of the rock exposed for oil. It was then given two applications of heavy asphaltic oil, applied at the rate of ¾ and ½ gals. per square yard, each coat being covered with rock screenings in sufficient quantity to absorb the oil.

The maintenance work on this section of the road has consisted entirely of that done by the patrolmen.

Fig. 10. Earth Shoulder Along Asphalt Macadam Pavement Worn Away by Traffic.

It is impossible to draw any definite conclusion on the cost of maintenance with respect to traffic on their road for the reason that the maintenance work for the entire year has been more in the nature of remedying imperfections of construction, such as covering excess oil and patching places which were given an insufficient amount of oil at the time of construction, rather than due to the wear and tear of traffic.

COPPEROPOLIS ROAD.

Asphalt macadam .... 1.00 mile
Oil macadam .... 7.99 miles

This road, Fig. 7c, is one of the main local traveled roads, and the best oiled macadam road in the system, notwithstanding the fact that the greater portion of the road is constructed over a very heavy adobe soil.

The first mile of this road was constructed of asphalt macadam at a cost of $12,058.42. The first half mile was 20 ft. in width and the second half mile 16 ft. in width. Asphalt macadam was placed 4 ins. in thickness upon the stone macadam base. The remaining portion of the road, 7.99 miles in length, was constructed at a total cost of $62,668.51, in the following manner:

In 1910, that portion of the roadway six miles in length passing over heavy adobe soil was graded, and a layer of cementing road gravel 6 ins. in thickness and 12 ft. in width per day. The effects of this traffic have apparently been beneficial, due to the kneading effect of the tires on the bitumen surface. The maintenance cost of $83.85 per mile, as shown in Table I, has consisted entirely on the overhead expense chargeable to the road, and in covering excess oil with screenings.

ROBERTS ISLAND ROAD.

Oiled macadam .... 8.96 miles

The greater portion of this road, Fig. 7d, passes over reclaimed lands. The natural soils of an adobe nature overlying a peat formation. The road was constructed in the year 1912 at a cost of $53,944.46. It was constructed in the following manner:

A layer of cementing road gravel 4 ins. thick, and varying in width from 10 to 12 ft., was placed upon the subgrade, watered, rolled and thoroughly compacted, after which a second course of 1½-in. rock 2 ins. in thickness, varying in width from 14 to 16 ft., was placed.

This course of rock was then given two applications of 80 per cent asphaltic road oil ing to the surface. The road was constructed in 1911 at a total cost of $38,474.89. The substance of the specifications under which it was constructed is as follows:

A layer of 1½-in. stone 4½ ins. in thickness, and varying in width from 10 to 14 ft., was placed upon the subgrade. This course of rock was thoroughly rolled and the voids filled with cementing earth from the sides of the road, leaving but the uppermost surface of the rock exposed to receive oil. Two applications of 80 per cent asphaltic oil were then made at the rates of ¾ and ½ gal. per square yard, each coat being covered with fine material in sufficient quantity to absorb the oil.

The maintenance work upon this road has consisted entirely of that done by patrolmen. The average cost per mile of maintenance on this road, as shown in Table I, has been $124.43, which amount includes the overhead charges for the year, amounting to $42.51 per mile. The pavement at present is in excellent condition.

SAN JOAQUIN COUNTY HIGHWAY MAINTENANCE DEPARTMENT

TRAFFIC CENSUS RECORD

Road............

Date............ 191 .

Location of Station...

Kind of Vehicle		Tally	Totals
Single Horse	Light Vehicle		
Two-Horse	Light Vehicle		
Two-Horse	Heavy Vehicle		
Four or More Horses	Heavy Vehicle		
Automobile	Run-about		
Automobile	Touring Car		
Motorcycles			
Weather Conditions		Total	

Signature of Observer

Observer will not write in this space

INSTRUCTIONS TO OBSERVERS.

1. Examine carefully the card marked "Sample," sent to you with the cards upon which you are to keep your records, and be sure that you understand the method of keeping the tally.

2. Your duties begin promptly at........o'clock in the morning and end at........o'clock in the evening for the seven days.

3. In recording vehicles which pass your station, make a mark on the proper line for every vehicle of the kinds called for on the cards. Make an entry for every vehicle, no matter in which direction it is going or whether it has passed you previously. Entries may be made with a lead pencil.

4. **"Light Vehicle"** means a buggy, cart, buckboard, carryall, spring wagon or any vehicle other than motor vehicles, which are usually for pleasure or light business purposes.

"Heavy Vehicle" means farm wagon, milk wagon, hay and grain wagons, truck wagons, drays, grocery wagons, motor trucks, provision wagons or any other vehicle, except motor vehicles, which are used for carrying heavy loads.

5. After you have recorded all the vehicles during the period from........o'clock to........o'clock, add your tally marks and place the totals at the right of the card and at the bottom.

6. When you are sure the totals are added correctly, enclose the card for the day, after you have signed it, in one of the addressed envelopes furnished to you, and mail at once.

7. Do not fail to record every vehicle, called for by the card, which passes your station. Do not record bicycles or pedestrians.

8. If there is anything which you do not understand about the cards or these instructions, write at once to the county surveyor, to whom you are to send the cards.

Fig. 11. Traffic Census Daily Tally Card.

(Size 6⅛ ins. by 9½ ins. On cardboard with instructions on the reverse side.)

was placed upon the grade. This water-bonded gravel pavement was allowed to pass through two winters, after which a contract was let for placing an oil macadam top on the same, and finish the remaining portion of the road with oil macadam. In placing the oil macadam top over the gravel base the gravel was first thoroughly scarified, shaped with a grader and widened to 16 ft., after which it was thoroughly rolled, watered and compacted, and a layer of 1½-in. rock 16 ft. in width was placed.

The 1½-in. stone was then given three applications of heavy asphaltic road oil applied at the rate of ¾, ½ and ⅛ gals., respectively, per square yard, each application being covered with coarse sand in sufficient quantity to absorb the oil.

The effect of traffic on this road with respect to maintenance costs is not apparent. The road is fairly heavy traveled, ranking fifth in horse-drawn traffic, with a daily average of 136.5 vehicles, and eighth in motor-drawn traffic, with an average of 93.3 vehicles applied at the rate of ¾ and ½ gal. per square yard, each application being covered with sufficient fine material to absorb the same, and thoroughly rolled.

A great deal of trouble was experienced from the excess oil appearing upon the surface, it being necessary to cover the excess oil with fine material several times. Owing to the remoteness of railway sidings the material haul was long, the average being 4½ miles. The maintenance cost has necessarily been high. It is impossible to come to any definite conclusions on the cost of maintenance on this road with respect to traffic, for the reason that the traffic for the year was exceptionally light. The entire maintenance cost of $281.31 per mile has been due to excess oil appearing upon the surface.

LODI-LAFAYETTE ROAD.

Oiled macadam .... 6.08 miles

This road, Fig. 7e, is an expensive type of oiled macadam, and well adapted to the sandy loam soil upon which it is placed. Little or no trouble has been caused by oil com-

MANTECA-RIPON ROAD.

Oiled macadam .... 8.28 miles

This road, Fig. 7f, is one of the main roads leading out of the county, and is subjected to a heavy travel. The road is one of the first oiled macadam roads built under the bond issue. The first two miles of the roadway were constructed in 1910, the remaining 6.28 miles being constructed in the following year. Both sections were constructed in precisely the same manner, which is as follows:

After the roadway has been graded and the subgrade prepared, a layer of 1½-in. stone 4 ins. thick and varying in width from 14 to 15 ft., was placed upon the subgrade. The rock was thoroughly rolled, and the voids filled to within approximately 1 in. of the surface, after which it was given two coats of 80 per cent asphaltic road oil applied at the rate of ¾ and ½ gal. per square yard, each application being covered with screenings in sufficient quantity to absorb the oil, and rolled.

The maintenance work for the year 1913

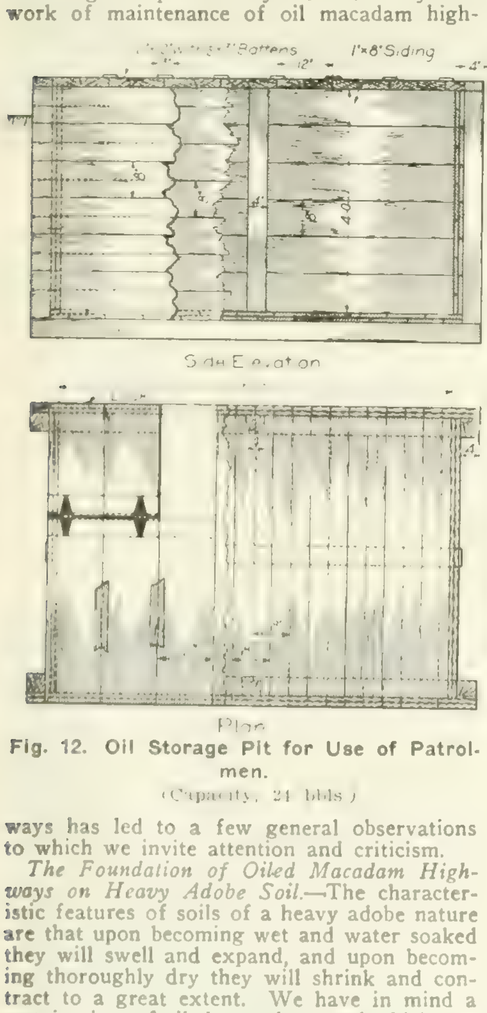
consisted of constructing an oiled earth shoulder 3 ft. wide on each side of the pavement for a distance of 3½ miles, and on one edge of the pavement for a distance of approximately 2½ miles. This work was done in the following manner at an approximate cost of $1,300. The earth was graded away from the edges of the pavement for a width of 3 ft. and to a depth of 1½ ins., after which a coat of 80 per cent asphaltic road oil was applied at the rate of 1 gal. per square yard, and covered with the earth which had been graded to the side. Owing to the loose sandy nature of the soil over which this road passes, considerable trouble had been experienced with the edges of the pavement raveling. The above-noted treatment has been a very satisfactory remedy for this trouble.

Other maintenance work has consisted entirely of that done by the patrolmen. The road, at present, is in excellent condition, and in need of no extra maintenance.

The effect of travel upon bitumen wearing surface of this road is not noticeable. The chief maintenance work has been upon the shoulders next to the oiled macadam pavement, in maintaining them flush with the edges of the pavement.

GENERAL OBSERVATIONS.

During the past two years the study and work of maintenance of oil macadam high-



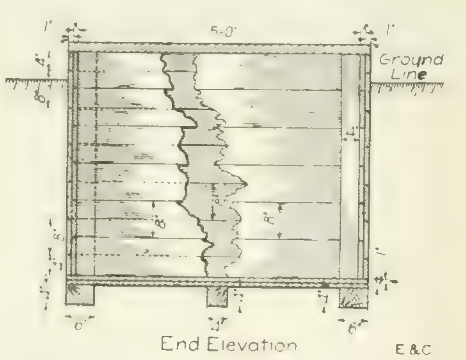


Fig. 12. Oil Storage Pit for Use of Patrolmen.
(Capacity, 24 bbls.)

ways has led to a few general observations to which we invite attention and criticism.

The Foundation of Oiled Macadam Highways on Heavy Adobe Soil.—The characteristic features of soils of a heavy adobe nature are that upon becoming wet and water soaked they will swell and expand, and upon becoming thoroughly dry they will shrink and contract to a great extent. We have in mind a certain piece of oiled macadam road which was built in this county on heavy adobe soil. A summary of the specifications under which it was constructed is as follows:

The roadway was first graded, the subgrade prepared, watered, rolled and thoroughly compacted, after which the foundation course of rock was placed. The size of the rock and foundation course ranged in size from 1½ to 3 ins.

This course of rock was thoroughly rolled, and the voids filled, after which it was rolled and watered again. A wearing course was then placed 16 ft. in width and 2 ins. in thickness. The size of the rock in this course ranged from ½ in. to 1½ ins. in size. This course was thoroughly rolled, and the voids filled to the surface. One gallon of crude oil containing not less than 60 per cent asphaltum was then applied in three applications with a gravity machine, each coat being covered with screenings in sufficient quantity to absorb the oil.

This road was constructed in the year 1908. The winter of 1909, being very wet, the pavement was flooded. The water, however, rose quickly and descended as rapidly, the pavement being under water but a very short time.

In the summer of 1909 the pavement began to show signs of failure, and upon examination by digging into the center of the pavement, the subgrade was found to be very wet. The pavement continued to get worse until in 1913, the surface was very badly rutted as shown in Fig. 8. Subsequent investigations by digging into the surface of the pavement, show that the heavy adobe earth in the subgrade in four years time had never dried out, and had worked its way through the two courses of stone, to the surface. This failure has led us to believe that where oil macadam highways are to be constructed on soil of this nature, it is advisable first to construct a water bonded, gravel, macadam pavement, and open it to traffic for two or three years, and then construct the oiled macadam pavement upon the gravel foundation. The gravel foundation does not seem to allow the adobe in the subgrade to work to the surface. It seems to act more as a concrete foundation would.

Some writers on road construction have recommended that in the construction of macadam highways on heavy soils, that the fine material be placed on the bottom and the coarser stone on top, the idea being to prevent the earth of the subgrade from working up in the rock, which is our idea.

Causes for excess oil appearing upon the surface of oiled macadam pavements.—We attribute the appearance of the excess oil upon the surface of the oiled macadam pavement to the following causes: (a). The over and uneven filling of voids in the wearing surface before the application of oil, and (b) weather conditons.

In some instances the voids in the wearing surface were entirely filled, leaving but the very uppermost surface of the rocks exposed to receive oil, and in other instances there was practically no filler in the top 2 ins. of the wearing surface. The filling of the voids in the wearing surface has varied greatly between these two extremes. As a result, after the completion of the pavements, spots have developed which indicate an over application of oil, but which, in reality, are mainly due to over filling of the voids. In some instances, in the same pavement within a few feet of each other, spots have developed which show an excess of oil, and others which show an insufficient amount of oil. As the amount of oil used in both cases has been the same, this shows that the correct amount of oil to be used is a function of the percentage of voids, as is generally understood. Figure 9 illustrates their condition.

In some instances oil was applied during warm summer months, and in other cases during cold, frosty weather of spring, winter and fall. As a result of the application of oil during cold weather, we have found that the oil does not absorb the same amount of screenings as it would were it applied during warm weather, and any excess screenings which may have been placed upon the surface of the pavement for absorbing the oil have worked to the sides of the road, and in a great many instances have been lost, due to wind and traffic. In the warm summer weather, capillarity and the kneading effect of traffic have caused the oil, which is apparently lying dormant, to work to the surface, and a covering of fine material is necessary to absorb it.

A large percentage of the cost of maintenance of a greater portion of the oiled macadam highways, for the first two years after their construction, has been in the covering of the excess oil which has appeared upon the surface.

The proper maintenance and care of excess oil appearing upon the surface.—Experience has taught us that where the excess oil appears upon the surface of the oil macadam pavement, it is not advisable to use fine sand, screenings or rock dust. If such are used, a mat or carpet of oil and fine material is formed over the surface, the thickness depending upon the amount of oil which comes to the surface. This mat or carpet has insufficient coarse material incorporated in it to give it the proper solidity and density. It tends to squeeze out and roll under the wheels of heavily loaded wagons, and forms depressions in the wheel tracks. It also has a tendency to roll and creep into knobs and depressions. Owing to the crown of the pavement, there is also a tendency for the oil to gravitate to the edges of the pavement, and there to collect in excess, forming wrinkles on the edges. Wherever oil appears in excess upon the surface, it is not advisable to use any material to absorb it which contains an excess of 15 to 20 per cent of material under ¼ in. in size. In our judgment, the material best suited for this purpose is a crushed rock ranging in size from ⅛ to ½ in. with dust taken out. The use of material of this class gives the road a hard surface, making it easier for traction.

Oiled Earth Shoulders.—The commission, due to the provisions of the law under which the work was accomplished, built a number of roads from 12 to 15 ft. in width, with oiled shoulders. The oiled shoulders were constructed by applying 1½ gals of heavy asphaltic oil upon the natural soil along the edges of the pavement, and covering with sand or screenings. This had apparently the effect of widening the pavement, for the oiled shoulders presented a bitumenized appearance and resembled the main surface of the pavement. These oiled earth shoulders have not held up under traffic. When heavy vehicles attempt to pass upon the narrow portion of the pavement proper, the wheels come upon the shoulders and they have given way under the load. As a result ruts, Fig. 10, have formed along the edges of the pavement. These ruts allow the water in winter to collect in pools and work under the edge of the pavement, thereby softening the foundation and causing it to fail. This feature is especially true where travel is heavily congested in the vicinity of Stockton and Lodi. However, there are certain classes of soil upon which we have constructed oiled earth shoulders which have proven very satisfactory. We have found, upon a coarse, sandy soil, where horse drawn traffic is not exceedingly heavy, that it is possible to construct oiled earth shoulders which will give satisfaction. We have found, upon soils of a loam or adobe nature, that the oiled earth shoulder built in this way is a failure.

Where the earth shoulders have failed, they must be remedied by the construction of an oiled gravel or stone shoulders, 3 or 4 ins. thick, and from 2 to 4 ft. in width, along both sides of the pavement. This will give the pavement a sufficient width to allow vehicles to pass without coming upon the earth shoulders. The cost of this class of work will necessarily be high, and should be considered as a betterment of conditions, and not an ordinary maintenance charge.

CONCLUSIONS AS TO TRAFFIC.

It is impossible for us at this time to draw any definite conclusions between traffic conditions and cost of maintenance, for the following reasons:

1. We have had charge of the entire system of highways for a period of only two years, both of which were unfavorable years, owing to scarcity of rainfall. This naturally caused a shortage of crops, and traffic conditions for 1912 and 1913 were far below normal.

2. The major part of our maintenance work for the past two years has been more in the nature of remedying imperfections in construction, rather than repairing defects caused by traffic.

The covering of excess oil with rock material to absorb it, has been our greatest expense. The work we have done in the way of patching the surface of oil macadam highways has been necessitated, not by the wear of traffic, but by the imperfections on construction.

We are not, at this time, able to note any wear due to traffic on the bituminous surfaces of the oiled macadam highways.

TRAFFIC CENSUS.

In Fig. 11 is shown a daily traffic census record card. This card is 6½ by 9½ ins. in size and instructions for its use are printed on the back.

To ascertain the amount and class of travel, and to study the maintenance cost with respect to this feature, and as an aid to future improvements a yearly traffic census has been taken. The traffic census for the year 1913 was taken over a period of seven days of twelve hours each, from August 25, 1913, to August 31, 1913, which dates cover the corresponding week of the 1912 traffic census. Below is a comparative statement of the 1912 and 1913 traffic census:

Item.	1912.	1913.
Total number of observers	21	33
Total number of stations	29	49
Total number of vehicles of all kinds	50,155	67,734
Per cent of motor traffic to total traffic	45.8	55.2
Per cent of horse-drawn traffic to total traffic	54.2	44.8
Motor traffic—		
Per cent of auto runabouts to total motor traffic	17.5	17.1
Per cent of touring cars to total motor traffic	60.0	59.6
Per cent of motorcycles to total motor traffic	21.1	19.7
Per cent of motor trucks to total motor traffic	1.4	3.6
Horse-drawn traffic—		
Per cent of single-horse light vehicle to total horse-drawn traffic	57.1	63.8
Per cent of two-horse light vehicle to total horse-drawn traffic	9.4	8.0
Per cent of two-horse heavy vehicle to total horse-drawn traffic	23.5	22.7
Per cent of four or more horse heavy vehicle to total horse-drawn traffic	10.0	5.5

SPECIFICATIONS FOR ROAD OIL.

The following is an abstract from the specifications for road oil used by this county:

1. Oil.—The oil shall be natural oil, with an asphaltic base, which may be treated to remove the water and other foreign matter, or residuum of such oil from which a portion of the volatile materials have been removed by distillation. It must not have been injured by overheating, and it must not have been manufactured by adding solid asphalt or refinery products to natural oil or distillate.

2. Volatility.—The oil shall not loose in excess of 1½ per cent volatile matter when heated from a temperature of 77° F. to 220° F. and maintained at 220° F. for 15 minutes. The time required for heating from 77° to 220° F. shall be 1 hour.

3. Water and Sediment.—Deduction will be made for water and sediment in exact proportion to the percentage of water and sediment found therein in excess of 1 per cent, and the oil shall not contain over 2 per cent of such water and sediment.

4. Organic Impurities—The oil shall not contain in excess of 0.2 of 1 per cent of organic matter insoluble in carbon tetrachloride at ordinary temperature.

5. Gravity.—Gravity of oil shall be between 10° and 11° Baume.

6. Asphalt.—After being freed from water, sediment and other foreign matter, the oil shall contain not less than 80 per cent asphalt, having at a temperature of 77° F. a penetration of 80, District of Columbia Standard. The percentage of residue of said oil or residuum is determined by heating in a crystallizing dish 2¼ ins. in diameter and 1½ ins. high in an evaporating oven at a temperature of 400° F., maintained until it has reached the proper consistency, when the weight of the residue shall be determined and the per cent calculated.

7. Measurement.—In determining the quantity of oil delivered the correction for expansion by heat shall be as follows: From the measured volume of oil received at any temperature above 60° F., an amount equivalent to 0.4 of one per cent for every 10° F. shall be subtracted as the correction for expansion by heat. For the purpose of measuring the oil, a temperature of 60° F. shall be deemed normal temperature.

OIL STORAGE.

Patrolmen's Oil Pits.—The patrolmen's oil pits, not located adjacent to railway sidings, to which oil is delivered in tank wagons, are constructed in accordance with the plans shown in Fig. 12. The inside dimensions of these pits are as follows: Length, 7 ft.; width, 5 ft.; depth, 4 ft. They are placed approximately 3½ ft. under ground, the upper 8 or 10 ins. of the pit being allowed to extend above the ground. To prevent the oil from seeping out, a double thickness of 1x8-in. lumber is used, the outer siding being made to lap over the cracks of the inner as shown in the drawing.

All pits are provided with small hinged doors and a means of locking the same to prevent rubbish from being thrown into the pit. No provision is made for heating the oil. The patrolmen dip the oil in heavy dippers, during the warm part of the day when the oil is warm and soft from the heat of the sun.

These pits have a capacity of 24 bbls., and the average cost of constructing six of them was $26.67; the highest cost being $30.66, and the lowest, $20.78.

Siding Pits.—At central points adjacent to railway sidings where large amounts of oil are to be used large under ground oil pits are constructed. A pit having a capacity of 370 bbls. has the following inside dimensions: Length, 19 ft. 8 ins.; width, 17 ft. 8 ins.; depth, 6 ft.

They are placed approximately 5½ ft. under ground, the upper 6 or 8 ins. extending above the ground. At the time of construction of pits of this type they were piped with ¾-in. perforated pipe and live steam was forced directly into the oil for heating. This method of heating has proven unsatisfactory, owing to the large amount of water which is deposited in the oil by the condensation of steam. As soon as emptied they will be piped with coils for heating purposes.

A heating box 4 ft. by 4 ft. in dimensions was constructed in one corner of the pit and similarly piped. The idea was that when only a small amount of oil was required, it would not be necessary to heat the entire pit. Oil in this small box or pit could be heated in a very short time. The heating box was left open at the bottom to allow oil to flow in from the sides when oil was being pumped from the heating box. A small hinged door with a lock on it is provided for the use of patrolmen.

All oil pits are covered with 2x12-in. planks with battens, or tar paper, placed over the cracks in order to prevent water from seeping into the pits and have a substantial rail around the sides. The present pits are not floored and the cracks on the ends and sides are not covered with battens. This allows water to seep into the oil from the sides and bottom. Upon being emptied they will be floored with 2x12-in. lumber and all cracks will be caulked or covered with battens. The cost of pits of this type is approximately $175.

WATER WORKS

Construction of Water Works Tunnels in the Metropolitan Water District of Massachusetts.

II.

Method and Cost of Constructing, by Day Labor, a Brick Lined Tunnel for 36-in. Water Main Under Chelsea Creek, Boston, Mass.

Contributed by William E. Foss, Assistant to the Chief Engineer, Metropolitan Water and Sewerage Board, Boston, Mass.

The East Boston low-service district includes a thickly settled residential district, which had a population of 56,230 in 1910, and much valuable wharf property along the water front. For 40 years this district was dependent for its water supply upon two mains laid under the bed of Chelsea Creek, a tidal arm of the ocean separating East Boston from the mainland of Chelsea. The creek, over these pipes, is about 1,500 ft. wide at high water, while the low water channel is about 300 ft. wide. At the channel the pipes are buried several feet below the bed of the creek, but on the flats on either side the pipes are supported on piles just above the surface of the mud, and are exposed twice each day at low tide.

For the purpose of guarding against the interruption of the supply to the district by the breaking of these mains, a new 36-in. main was laid in a different location. This new main connects with existing 20-in. and 24-in. mains in Chelsea and extends along the northerly shore of the creek to a point near the Chelsea Street bridge leading to East Boston, where it turns easterly and crosses under the channel through a tunnel 504 ft. long to the East Boston shore. It is to the methods and cost of constructing this brick-lined tunnel, for carrying the 36-in. cast-iron water main under the creek, that the present article relates. This work was by day labor and was performed from July, 1910, to January, 1911.

The tunnel is located about 25 ft. down stream from the westerly side line of the Chelsea Street bridge, and includes a vertical shaft at each side of the creek, 9 ft. 4 ins. outside diameter, with top at elevation 14 on the Chelsea shore and at elevation 10 on the East Boston shore. The horizontal section of the tunnel, joining the shafts, is 400 ft. in length, 8 ft. 2 ins. outside diameter with the top 36 ft. below mean low water at the Chelsea end. A longitudinal section through shafts and tunnel are shown in Fig. 1. The shafts were constructed with 12-in. and the horizontal portion of the tunnel with 8-in. brick walls, as shown in Fig. 2.

The Chelsea shaft rises about 12 ft. above the bed of the creek and is protected by a steel casing which extends about 13 ft. into the silt bottom. The East Boston shaft was sunk through the earth filling, back of the masonry sea wall, and is protected by a steel casing for a distance of 8 ft. below the top.

The axis of the horizontal section of the 36-in. pipe, which was laid in the tunnel, is at elevation -40. The pipes were laid with ½-in. opening between the end of the spigot and the bottom of the socket, and the joints were run solid with lead and were calked both inside and outside after the pipe was laid. A detail of the pipe joint is shown in Fig. 3. The space between the pipe and the brick wall was filled with Portland cement concrete. A view showing the laying of the pipe and the concrete filling is shown in Fig. 4.

Special 36-in. branches were used at the junction of the horizontal and vertical portions of the pipe line. Thirty-six-inch ¼ curves with manholes were set at the top of the shafts. The pipes used were 1.61 ins. thick.

The steam plant for operating the air compressors, hoists and electric lighting plant was set up on the Chelsea shore of the creek during the latter part of July, and the work of sinking the shaft was begun during the week ending August 13. After August 21, when the air lock was in place, the work was carried on continuously during 24 hours per day, with three shifts. While excavating the mud and silt just below the bed of the creek some inconvenience was experienced on account of gas, which entered the shaft and affected the eyes of the workmen.

The work of excavating and lining the shaft was completed about September 1. An air lock was then built at the entrance to the horizontal portion of the tunnel, and the small lock which had been used for sinking the shaft was removed. A view of the air lock construction is shown in Fig. 5. The excavation and lining of the horizontal portion of the tunnel progressed at the rate of about 5 ft. per day. The air pressure maintained varied from 14 to 23 lbs. per square inch, according to the stage of the tide in the creek above.

On October 13 a blow-out occurred about 150 ft. from the Chelsea shaft at a point where a pile had been removed. As a result, the tunnel was flooded with water to a depth of about 4 ft. After the hole was stopped the water was pumped out, and the work proceeded without further mishap.

On the East Boston side of the creek the material excavated was hardpan containing boulders, which required some blasting, so that the rate of progress was less than it had been in the sand and clay on the Chelsea side of the creek. A 2½-in. steel pipe was driven, during the week ending November 12, on the center line of the tunnel near the East Boston end, from the surface of the ground to the center of the tunnel, for use in supplying compressed air for sinking the East Boston shaft.

Work in the tunnel was discontinued on November 17, when steel sections of the East Boston shaft and the hoisting engine were set up on the East Boston side of the creek. On November 18 the work of excavating the East Boston shaft was begun, and on November 24 air pressure was applied.

An opening was made from the bottom of the shaft into the tunnel on December 3. All excavation and the brick lining for the tunnel were completed on December 6, and the air pressure was removed on the morning of December 9. A total of 104 lin. ft. of shaft and 400 ft. of tunnel were built. The tunnel was cleaned out and plastered and, after calking a few small leaks, was substantially watertight.

The force employed on this work while working continuously under air pressure averaged about 15 men for each of the three shifts. After the air pressure was removed the work was carried on with three shifts working six days per week, the force employed averaging about 16 men per shift. The following wages were paid:

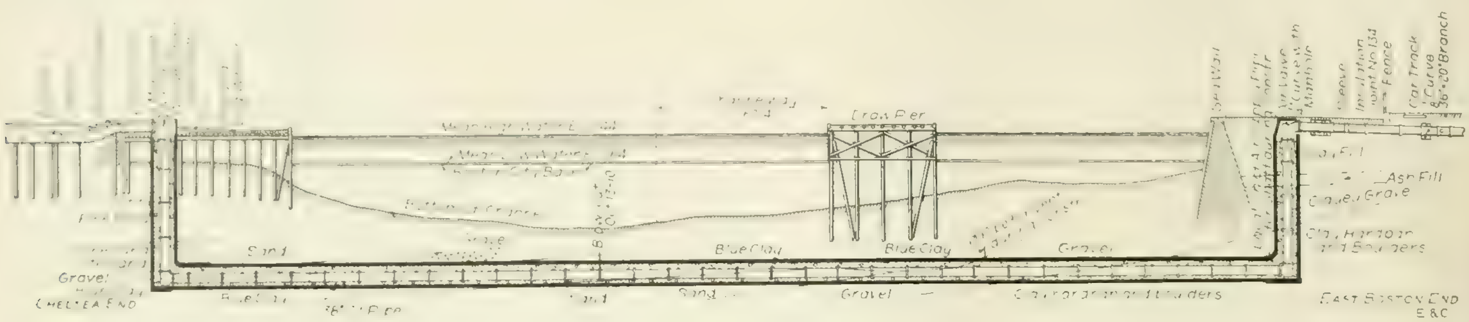


Fig. 1. Longitudinal Section Through Water Tunnel Under Chelsea Creek, at Chelsea Street Bridge, Boston, Mass.

PRICES PAID FOR LABOR.

	For 8-hour day.
Engineer in charge of plant	$5.00
Engineer	4.00
Fireman	2.40
Head miner and foreman	4.00
Miner	2.80
Tunnel laborer	2.40
Laborer	2.00
Calker	3.50
Head mason	7.20
Mason	6.40
Boy	1.60
Double team with driver	5.00

PRICES PAID FOR MATERIALS.

The following prices were paid for materials:

Brick, delivered on work, per M	$9.70
Portland cement, delivered on work in bags (credit for empty bags returned, 6½ cts. each), per bbl	1.68
Coal, delivered on work (for 14,700 B. T. U. per pound of dry coal), per gross ton	4.10
Ipswich sand, delivered on wharf, for brick-work, per ton	0.75
Plum Island sand, delivered on wharf, for concrete, per ton	0.75
Bank sand, delivered on cars at bank, per cu. yd.	0.35
Freight, additional, per cu. yd	0.35
Teaming, additional, per cu. yd	0.40
Screened gravel for concrete, delivered on cars at bank, per cu. yd	0.75
Freight, additional, per cu. yd	0.35
Teaming, additional, per cu. yd	0.43
Broken stone for concrete, delivered on work, per ton	1.50
Lead, per 100 lbs	4.675

GENERAL EXPENSES.

Following is an itemized list of general and itemized expenses:

Superintendence and rental of plant		$ 7,700.00
Installing plant:		
Labor	$ 412.90	
Teaming	130.00	
Supplies	121.19	
		664.09
Housing plant:		
Labor	525.85	
Lumber, $743.77; $205.36 received for old lumber	538.41	
Miscellaneous supplies	39.78	
		1,104.04
Operating plant:		
Labor	1,234.40	
Coal	1,947.65	
Water for boilers and compressors	225.35	
Miscellaneous supplies	538.49	
		6,945.89
Removing plant:		
Labor	230.80	
Teaming	106.00	
		336.80
Rental of land		195.00
Miscellaneous expenses		88.35
Total general expenses		$17,034.17

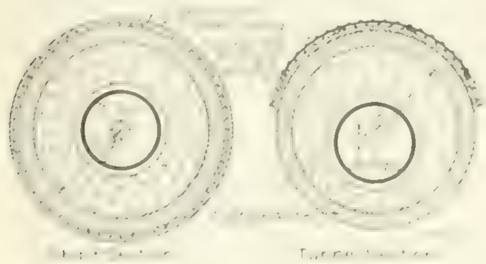


Fig. 2. Cross Sections of Shafts and Tunnel of Chelsea Creek Tunnel.

The plant was operated from 7 a. m. Aug. 11, 1910, to 4 p. m. Jan. 11, 1911, a total of 4,017 hours. The cost of general expenses per hour was therefore $4.24.

EARTH EXCAVATION.

About 1,040 cu. yds. of earth were excavated under air pressure of 14 to 23 lbs. per square inch. Work was continuous for three shifts per 24 hours. The mud and silt was excavated just below the bed of the creek at the shaft on Chelsea shore. Fine sand and gravel were excavated for 10 ft. below the silt, and the bottom 5 ft. of excavation was in clay.

The shaft on the East Boston shore was excavated through filling of clay and ashes for a depth of 25 ft., coarse gravel for a depth of 15 ft., and below this line the excavation was in hard pan. The horizontal portion of the tunnel was excavated through stratified sand, clay and gravel, the strata dipping about 6° towards the Chelsea shore, so that for the first portion of the work the floor was in clay and the arch in sand. As the work progressed the floor was in gravel and the arch in clay, and at the East Boston end the entire excavation was in hard pan, with some boulders which required blasting. The cost of earth excavation was as follows:

	Cost.	Per cent of total.
General expenses	$10,190.40	50.0
Steel casings for shaft	833.00	4.1
Roof plates for tunnel	1,387.31	6.8
Lumber	98.96	0.5
Tools	112.36	0.5
Labor	7,778.70	38.1
Total	$20,400.73	
Cost per lin. ft. of tunnel	40.68	
Cost per cu. yd	19.62	

BRICK LINING.

Approximately 320 cu. yds. of brick masonry were built under air pressure of 14 to 23 lbs. per square inch. Work was continuous for three shifts per 24 hours.

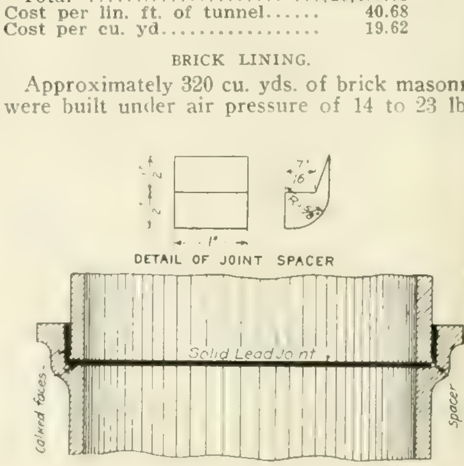


Fig. 3. Detail of Pipe Joint and Joint Spacer, Chelsea Creek Water Tunnel.

	Cost.	Per cent of total.
General expenses	$ 3,078.64	29.2
Brick	1,971.52	18.7
Cement	1,129.00	10.7
Sand	331.39	3.1
Mason	1,210.80	11.5
Labor	2,786.25	26.5
Miscellaneous supplies	36.76	0.3
Total	$10,544.36	
Cost per lin. ft. of tunnel	21.03	
Cost per cu. yd. of tunnel	32.95	

LAYING 36-IN. PIPE.

Five hundred and three feet of 36-in. pipe were laid. Work was continuous for six days per week with three shifts per 24 hours.

	Cost.	Per cent of total.
General expenses	$1,708.96	42.2
Lead	382.41	9.4
Calking	252.00	6.2
Labor	1,661.10	41.0
Tools and miscellaneous	50.25	1.2
Total	$4,054.72	
Cost per lin. ft. of tunnel	8.08	

PORTLAND CEMENT CONCRETE PROTECTION FOR PIPE.

Four hundred and seventy-eight yards of concrete were placed. Frost was removed from the sand and gravel for concreting, by steam. The materials were mixed in the following proportions: 380 lbs. of cement, 10 cu. ft. of sand, 18 cu. ft. of gravel. Work was continuous for six days per week with three shifts per 24 hours.

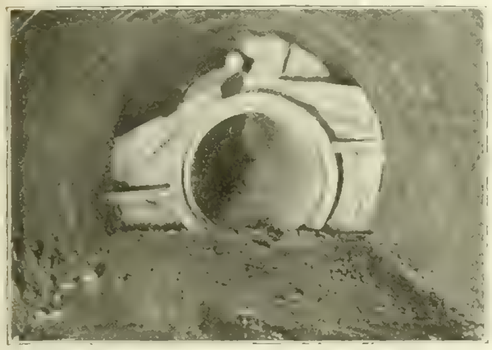


Fig. 4. View in Chelsea Creek Water Tunnel Showing Laying of Cast Iron Pipe and Concrete Filling.

	Cost.	Per cent of total.
General expenses	$2,056.17	35.8
Cement	917.11	16.0
Sand and gravel	744.15	13.0
Labor	1,994.70	34.8
Miscellaneous expenses	25.00	0.4
Total	$5,737.13	
Cost per lin. ft. of tunnel	11.44	
Cost per cu. yd. of tunnel	12.00	

COST OF CAST IRON PIPE AND SPECIALS.

The cost of the cast iron pipe and special castings was as follows:

36-in. cast iron pipe, class H:	
157.348 tons straight pipe, delivered on wharf, at $24.40 per ton	$3,839.29
Cost per lin. ft. of tunnel	7.65
36-in. cast iron specials:	
11.1605 tons, delivered on wharf, at $49	546.86
Cost per lin. ft. of tunnel	1.09

SUMMARY OF COSTS.

	Total cost.	Cost per lin. ft. of tunnel.
Earth excavation	$20,400.73	$40.68
Brick lining	10,544.36	21.03
Laying 36-in. pipe	4,054.72	8.08
Portland cement concrete protection for pipe	5,737.13	11.44
	$40,736.94	$81.23
36-in. cast iron pipe	3,839.29	7.65
36-in. special castings	546.86	1.09
Total*	$45,123.09	$89.97

*Exclusive of land damages and engineering and testing and miscellaneous. These items amounted to a sum of $5,687.53 additional, making the total cost $50,810.62.

CONSTRUCTION PLANT.

Following is a list of the construction plant units employed and their estimated value when new:

	Est. value when new.
1 air compressor, Ingersoll No. 119, 24x24-¼x30 ins.	$ 2,700
1 air compressor, Ingersoll No. 82, 18x18-½x24 ins.	2,200
2 75 hp. vertical Manning boilers, 60x13 ins.	2,400
1 60 hp. horizontal Economical boiler, 50 ins x 10 ft.	700
2 air receivers, 4x12 ft.	200
1 shaft lock, 6x6 ft.	700
1 tunnel lock, 6x12 ft.	225
1 4x3-in. Worthington duplex pump	200
1 4x3-in. Knowles duplex pump	200
1 head house and conveyor	200
1 12 hp. Paine dynamo engine	500
1 Crocker-Wheeler generator, 50 amperes, 110 volts, 12000 R. P. M.	130
1 two-drum, 20 hp. double cylinder, Floyd Mfg. Co. hoisting engine and boiler	1,200
1 20 hp. double cylinder Kendal & Roberts hoisting engine, without boiler	600
5 tons steel rails	200
3 car trucks, with wheels, axles and boxes	105
Centers, ribs and lagging	100
Pipe and fittings	500
Total	$13,240
6 skips, 5x2½x2½ ft.	180

Regulations Governing Preparation of Reports on Water Supply Systems and Extensions in Saskatchewan.

In ENGINEERING AND CONTRACTING of Sept. 24, 1913, the rules and regulations of the New Jersey State Board of Health relative to reports on water supply and water purification systems were published. The need of set rules and regulations to govern the submission of water and sewer plans to State Boards of Health was set forth in our issue of Oct. 29, 1913, and need not be repeated. Following the lead of New Jersey in this particular the province of Saskatchewan has recently issued, through the provincial Bureau of Public Health, a set of regulations governing the preparation of plans for the installation, in that province, of water works, sewerage and sewage disposal systems. Saskatchewan is the first of the Canadian provinces to take this action. These regulations differ sufficiently from those issued by the New Jersey Board to warrant their publication here. The Saskatchewan regulations as to water works systems and extensions which follow contain many suggestions of value to water supply engineers generally.

ENGINEER'S REPORT.

A report shall be submitted prepared by the engineer acting for the municipality giving information under the following heads, in so far as such are applicable to the proposed works.

1. Population.—The present population and the rate of increase during each of the last five years; available data justifying a future increase of population and probable increase in next ten years.
2. Maximum population which system will provide for if fully developed.
3. Population provided for by proposed system.
4. Area and topography of municipality.
5. Estimated water consumption per capita per 24 hours.
6. Alternative sources of supply (if any) and if investigated.
7. Source of supply recommended.
8. Present available water and how determined.
9. Estimated supply of water under full development.
10. Quality of water.
11. Any present sources of pollution of source of supply.
12. Measures recommended to prevent future pollution of source of supply.
13. Nature and construction of intake or collection works.
14. Wells.—The number, depth, size and construction of wells; the nature of the ground through which they are sunk; the construction of collecting galleries.
15. Watercourses.—Minimum dry weather flow, approximate watershed area. If continuous winter flow, reduction in available supply during winter.
16. Catchment Areas.—Approximate area of watershed, character of watershed surface with reference to probable run-off, population, arable and stock farming and all available data having reference to rainfall.
17. Natural Lakes.—Approximate area of watershed, average depth of lake, area of lake, if overflow is continuous, approximate annual overflow, nature of watershed relative to population, arable and stock farming, and all available data having reference to rainfall. Population on shore winter and summer. If surface is used for traffic in winter.
18. The capacities and character of pumps; duplication of machinery and energy.
19. Sedimentation.—The size and construction of basins.
20. Filtration.—A full description of the proposed plant, including the type of filter, the nature of filtering media, the maximum rate of operation of each unit of the plant; the method of cleansing filtering media; the nature and amount of coagulant or disinfectant estimated to be necessary.
21. Provision for future extensions.
22. Provision for measuring and recording amount of water supplied to municipality.
23. Provision made for storage and capacity of reservoir, stand pipes, etc.
24. Size of force or gravity mains.
25. Distribution system, especially with reference to cut off valves, sluice valves, hydrants and through circulation; minimum size of pipes and minimum depth of trench; character of subsoil.
26. Pressure available for domestic and fire purposes.
27. Particulars as to any provision made for turning water other than the domestic supply into mains for fire protection purposes.
28. Provision to be made for inspection of construction.
29. Control and operation.

In the case of extensions to an existing system, the report shall, in addition to describing the nature of the proposed extensions, deal with as many of the above matters as are thereby affected.

ESTIMATES OF COST.

A statement prepared by the engineer shall be submitted showing the estimated cost of the proposed system or extensions. The estimated cost of all pipe lines shall be shown in schedule form as follows:

Location.	Between.	Size of pipe.	Character of pipe—whether steel, cast iron, etc.	No. of existing buildings requiring water.	Average depth of trench	Length in feet.	Cost of pipe laid complete per lin. foot.	Total cost.

The estimated cost of all other sections of the work (e. g., land, intake, headworks, pumps, reservoir, hydrants, valves, etc.) shall be shown separately at the end of the schedule. The estimated operating costs per annum shall also be given.

GENERAL PLANS.

The following plans shall be submitted:

Plan of Municipality—A general plan of the entire municipality to a scale of not greater than 200 and not less than 500 ft. to an inch. This plan shall show:

Fig. 5. View of Air Lock, Chelsea Creek Water Tunnel, Boston, Mass.

1. The municipal boundary.
2. All streets existing or proposed.
3. The approximate location of all habitable buildings not served with water at the date of application.

4. The surface elevations at all existing or proposed street intersections.

5. The elevation of the highest point in the municipality.

6. The location and size of all existing and proposed water mains.

7. The location of stand pipes, valves, hydrants and all appurtenances.

Location Plan.—A plan to a suitable scale, showing the site of the municipality, any outside territory affected, and the following:

1. The source of supply, intakes, wells, filters, pumps, reservoir and any special features.

2. The route of gravity or force mains.

3. The following elevations shall be shown:

(a) The low, mean and high water levels of the watercourse or lake at the intake; (b) The highest known flood elevation of the watercourse or lake; (c) Surface and water elevations at standpipes, reservoirs and principal points in the system; (d) Surface elevations showing any irregularities of route with reference to the hydraulic gradient.

This plan is not required in the case of extensions to a distributing system.

Analysis of Proposed Source of Supply.—A sample of the water from the proposed source of supply shall be taken not more than six weeks previous to the date on which application is made to the Commissioner of Public Health for his certificate of approval. The sample shall be subjected to chemical analysis and bacteriological examination. A copy of the result of such analysis shall be submitted to the Commissioner of Public Health. The commissioner may require any number of samples, extending over a stated period of time, if necessary.

The above shall also apply to each application for approval of extensions to an existing system, the sample being taken from the existing mains, should there be no proposed change in the source of supply.

Detail Drawings.—Detail drawings shall be submitted to a scale which shall clearly indicate the design of all sections of the proposed system or extensions, including all wells, collecting galleries, intakes, filters, settling basins, reservoirs, conduits, standpipes, siphons, blowoffs, pumps, machinery buildings, and other appurtenances.

Profile.—In the case of gravity supply, a profile of the route of the pipe line shall be submitted, showing surface elevations, the hydraulic gradient of pipe line, and air and sluice valves.

Specifications.—Specifications shall be submitted covering all work to be undertaken in the proposed system of extensions.

In Saskatchewan certain affidavits are required in connection with the foregoing. Thus an affidavit must accompany any report of water analysis, stating that the report of the analysis and examination submitted is a true copy of the report supplied by the analyst, and

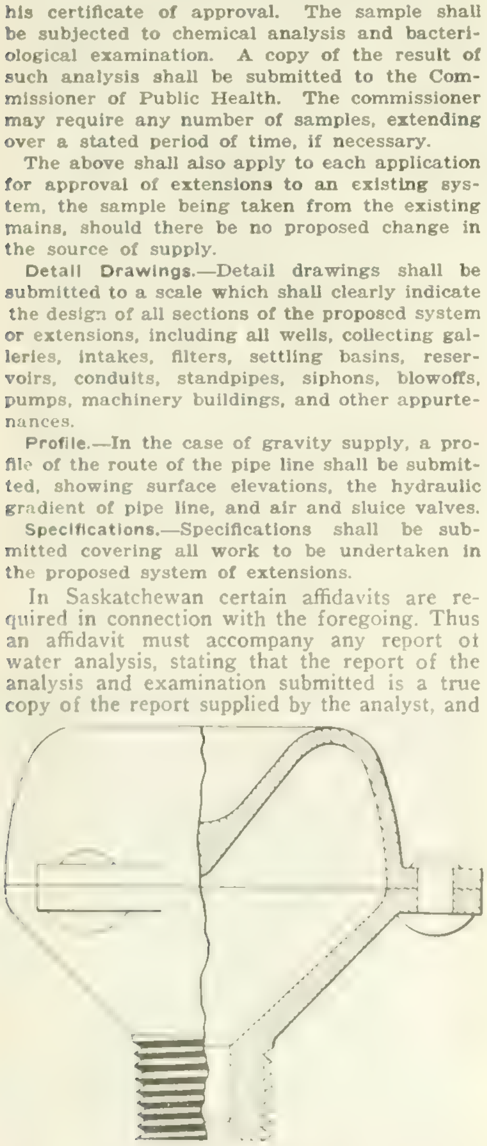


Fig. 1. Cross Section of Strainer, New Canaan Filters.

that the water analyzed and examined was taken from the proposed source of supply. The date on which any sample was taken must also be stated in the affidavit. Also all detail drawings and specifications must be accompanied by affidavits stating that the drawings and specifications are those to be used in the construction of the proposed system or extensions.

Design Features and Cost of Operating Water Filters of Pressure Type at New Canaan, Conn.

New Canaan, Conn., is supplied with water from a reservoir situated about three miles north of the town. The reservoir has a drainage area of 1 1/10 square miles and a storage capacity at the present time of 70,000,000 gals. About 20 per cent of the drainage area consists of swamps. These swamps act as large tea kettles, in which the leaves and vegetation are steeped, giving the water a high color, especially when a heavy rain flushes the swamps out.

In 1912 Mr. A. B. Hill of New Haven was called in to suggest some means of remedying the situation. After a careful investigation he recommended a series of marginal ditches around the swampy area, to take the water directly from the hillsides to the reservoir and prevent it from steeping in the swamps. In the summer and fall of 1912 these ditches were laid out and partly completed, work being stopped by the winter. The entire scheme of ditching has been only about two-thirds completed. The ditches that were completed served to reduce the color from the ditched areas considerably. It is interesting to note, in this connection, that since the New Haven Water Co. has done some of the same kind of ditching around one of their reservoirs, that the color of the water in the reservoir has been reduced from around 60 to from 20 to 30.

In the early spring of 1913, the Civic League of New Canaan petitioned the Public Utilities Commission to have the New Canaan Water Co. filter the water, and shortly afterwards the petition was granted, with permission for the water company to raise its rates to compensate for the expense of the filters.

The water company authorized its engineer to install a suitable filter plant. As slow sand filters have not been markedly successful in the removal of color and as there was no spare head, a pressure filter plant with the use of a coagulant was decided upon. The design features and the cost of operating this filtration plant are here described and recorded from information contained in the paper presented before the latest annual meeting of the Connecticut Society of Civil Engineers by Mr. Kenneth W. Leighton.

Excavation for the foundation of the building was started May 13, 1913, and water was turned through the filters on July 1, 1913. The filter plant was located so that the line of the old 12-in. supply main came about 2 ft. inside of the east wall of the building. About 80 ft. of this old supply main had to be taken up in order to insert the Venturi-meter and branches for the filters. As this 12-in. main forms the only supply for the town, a temporary 6-in. pipe was tapped in below the filter plant and run to a notch cut in the concrete spillway of the dam.

The filter plant proper consists of four filters, each capable of filtering 250,000 gals. in 24 hours. This amount is based on 2 gals. per minute per square foot of horizontal filtering area. This arrangement provides sufficient filtered water for washing one filter while the other three are in use, and also allows for future growth in consumption. The consumption for short periods has run as high as 600,000 gals. per 24 hours.

The filters are of the Continental Jewell type and consist of steel tanks 10 ft. in diameter and about 7 ft. high with convex tops and bottoms. Just above the convex portion of the bottom are placed a series of bronze strainers, about 200 in number. The slits in these strainers are so small that the sand or gravel cannot get through them. The strainer unit is shown in Fig. 1. The bottom of the tank is concreted in up to the strainers. Above the strainers is a 9-in. layer of gravel ⅛ to ¾ in. in size. Above the gravel is a 30-in. layer of sand. From the top of the sand to the top of the tank there is just room for a man to get in and move around uncom-

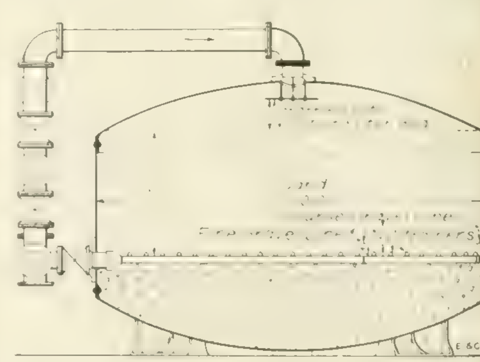


Fig. 3. Section of Filter Unit, Pressure Filters, New Canaan, Conn.

fortably. A section of one unit is shown in Fig. 2.

Before the water reaches the filters some of it is forced by a back pressure valve through either one of two tanks, each containing about 100 lbs. of alum. A certain amount of the alum is dissolved and carried back into the supply main. The alum unites with a portion of the alkali in the water, forming a flaky precipitate of aluminum hydroxide which

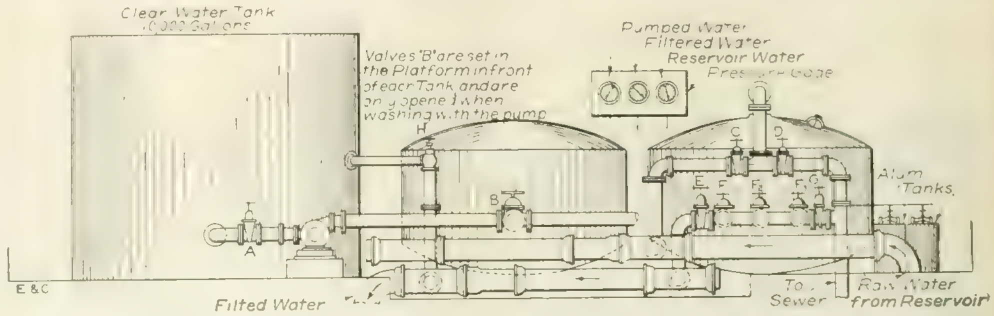


Fig. 2. Elevation of Two Pressure Filter Units and Clear Water Tank, New Canaan, Conn., Showing Piping for One Unit Only.

To Wash Filters With Reservoir Pressure:

Close valves C, G, F_3 and F_2. Open valve D and leave F_1 and E open. This will wash one section. As soon as the water in the sight glass runs fairly clear, close F_1 and open F_2 and so on until all three sections have been washed. Then close D and E and open C, F_1, F_2, F_3 and G. This will start filtering the raw water and run it to waste. As soon as it clears up (two or three minutes) close G and open E. This whole operation ought to take about 15 to 20 minutes. The clear water tank is filled by opening valve H. The filters should be washed every day and oftener if the pressure loss through the filter exceeds 4 lbs.

To Wash Filters With Clear Water Tank and Pump:

Close valves E, C, F_2, F_3 and leave B and G closed. Open A, F_1 and D. See that motor bearings are oiled and air out of pump. Start motor. This will raise the pressure to 25 or 30 lbs. Open valve B until the pressure drops to 18 or 19 lbs. When the wash water shows clear in the sight glass open F_2 and shut F_1. When again clear open F_3 and shut F_2. When finished washing this last section shut off the motor, close B, D and E and open F_1, F_2, F_3, C and G. This lets the filtered water run to waste. The water should become clear in about two minutes, and as soon as it does close G and open E. Do not open valve B before starting pump, as it might be opened too wide at first and so overload the motor.

serves to entangle small particles and coloring matter and to make a coating on top of the sand. One grain of alum per gallon will use up about eight parts of alkalinity per million, so that if the water is deficient in alkalinity some alkali, such as sodium carbonate, would have to be added.

In seven months there has been used 4⅓ tons of alum, and 47,596,000 gals. of water have been filtered, making 1 4/10 grains of alum per gallon of water. It is very likely that this can be reduced somewhat before long.

The water, on entering at the top of the filter unit, is deflected by a baffle plate, thus spreading it evenly over the top of the bed. In going through the filters, there is a loss in pressure of from 2 to 4 lbs. When it reaches the latter figure it is time to wash. The filters are washed at least once a day, even if the loss in pressure does not reach 4 lbs. The filters are of the sectional wash type and only one-third of a unit is washed at a time. This gives greater pressure and tends to keep the water from burrowing through the filtering material. The washing is done by reversing the flow of water. The washing of a unit takes about 15 minutes, unless the bed is exceptionally dirty. The scheme of washing a typical unit is shown in Fig. 3.

When the reservoir is low the filters are washed by using the 10,000-gal. clear water tank and the pump. The pump is a 5-in. suction, 5-in. discharge, Kingsford centrifugal pump coupled to a 10-h.p. Westinghouse induction motor. The clear water tank holds enough to wash approximately two filter units. The water used for washing is about 5 per cent of the total water consumed during 24 hours. After passing through the filters, the water is measured by a Venturi meter, which records the rate of flow every 10 minutes and the total amount in gallons.

The building is heated by an ordinary round station stove with about a 20-in. fire pot, and will burn about 5 tons of coal during the winter.

The efficiency of the plant is indicated by the comparative analyses of the raw water in past years and of the filtered water at the present. An examination of the raw water analyses made at monthly intervals from December, 1911, to June, 1913, shows that the color ranged from 28 to 96 parts per million, the turbidity from 1 p. p. m. to 60 p. p m. and the alkalinity from 7 p. p. m. to 25 p. p. m. During the same period the odor has been characterized as grassy, faint, faint peaty, or distinct peaty, and the color of the sediment has been termed slight brown, slight gray or dark brown. The last analysis of the filtered water by the State Board of Health is as follows: Color, 0; turbidity, 0; nitrates, 0; free ammonia, 0; odor, 0; sediment, 0; chlorine, only 0.1 above normal; hardness, 32.68 (less than 60 is considered soft water); bacteria, only 175 per c. c. and no suspicious ones.

Construction and Operation Cost Data.

The following tabulation gives the approximate cost of the filter plant, the estimated cost of operating for a month, and the cost per 1,000,000 gals. of water:

Cost of filter plant:	
Building	$ 6,000.00
Filters	12,500.00
Venturi-meter	600.00
Miscellaneous	1,900.00
	$21,000.00
Cost of operating per month:	
Attendants' salary	$ 65.00
Power (minimum charge)	10.00
Light	.50
Telephone	4.50
Compensation insurance	1.00
Coal	3.30
Alum	27.00
Miscellaneous	2.00
Depreciation:	
Machinery, 10 per cent	125.00
Building, 2 per cent	10.00
Interest on investment at 6 per cent	105.00
	$ 353.30

Amount of water filtered per month equals 7,000,000 gals.

Cost per 1,000,000 gals. of water filtered, $50.43.

It should be noted that more water could be filtered, if the occasion demanded, without increasing the present cost materially.

GENERAL

A Discussion of Relative Values in Sanitation.

To maintain a clean and safe environment costs money, often a good deal of money. How shall one determine whether a city's money expended for this purpose is well spent? The cost of municipal sanitary measures can usually be ascertained with reasonable exactness and hence their relative cost can be computed. But how may one know that the value to the community of these measures is commensurate with their cost and that a proper allotment of funds is being made? It may be said at the outset that there is no definite means of ascertaining this. The question cannot be answered categorically. The best that can be done is to call attention to a few guiding principles, but if due consideration is given to them much may be gained over present practice. These guiding principles were discussed by George C. Whipple, consulting engineer, New York City, and professor of sanitary engineering at Harvard University, in a paper presented before the latest annual meeting of the Connecticut Society of Civil Engineers. The following matter is from that paper as published in the annual journal of the society.

All sanitary measures are not directed to a common end. Some are chiefly and avowedly for the protection of the people against disease, as for example, vaccination, disinfection processes, and the sterilization by the use of chemicals of public water supplies. Some measures are to protect people against accident. On the other hand, many of our most conspicuous and expensive sanitary operations, such as refuse disposal, street cleaning, ventilation of buildings and cars, promote comfort and convenience more than health. Again we have the element of decency which, though intangible, is nevertheless an important factor in the situation, as it merges imperceptibly into morals. The proper disposal of the excretions of the human body is influenced by this motive. Between health, comfort and decency we often find it hard to discriminate and in attempting to evaluate them we encounter at the outset a great variety of philosophical conceptions.

Where no common denominator exists it is hard to make comparisons. As a matter of fact life, health, comfort and decency are prime factors and we cannot reduce them to lower terms. And so in deciding between measures which perform one or the other service we can do no more than exercise our best judgment, taking into account the sum total of the advantages to be derived from the measures recommended and our ability to pay for them.

But in order to judge wisely in regard to these matters it is necessary to be correctly informed as to what can be accomplished by sanitation and what need not be expected. This demands a careful study of the facts by persons competent to draw conclusions from them.

Popular Misconceptions.

The popularization of sanitary science has been both beneficial and harmful. So rapid have been the advances in certain directions, so marvelous, for example, have been the results of bacteriological discoveries, that a spirit of over-confidence in new ideas has been developed. A flabby gullibility is taking the place of robust criticism. This is due in great measure to the manner in which scientific studies are exploited or "featured" by the press. False and misleading headlines are read by the many, while the articles themselves, often of a very different tenor from the headlines, are read by but few; hence the quick reading public may get one opinion, while the more studious reader may get an entirely different opinion from the same page. When, as in one instance, public opinion is so influenced by diluted science as to cause the legislature of one of our states to propose a law requiring the sewage of all cities to be purified in a septic tank, the matter becomes more serious.

Hence one of the most important things to be done in considering relative values in sanitation is to educate the public as to what can be accomplished and what cannot be accomplished by the various sanitary measures that are continually being proposed.

A Few Comparisons.

It may be interesting to run over some of the sanitary measures in common use with reference to the nature of their usefulness, bearing in mind that the statements made are very general in character and subject to modification under exceptional conditions.

The purification of public water supplies has a very important influence on public health. But apart from this influence clean water, as contrasted with water which is turbid, discolored, or ill smelling, has a value of its own.

Sewerage, that is the removal of water closet wastes and other wastes from human habitations, is an important health measure. Sewerage has also a value reckoned in terms of comfort and decency.

Sewage treatment, sometimes miscalled sewage purification, except in special cases, has but a small influence on the public health. Nevertheless it may be very important when judged by standards of comfort and decency.

Street cleaning has an influence on public health, but its value lies chiefly in comfort and convenience.

Refuse collection and disposal has but little direct influence on the public health. Its value lies chiefly in comfort and convenience.

The ventilation of buildings and other enclosed spaces is only to a slight extent a health measure. It relates chiefly to personal comfort.

The congestion of population in the crowded districts of cities considered by itself and apart from the habits of the people, probably has less influence on the transmission of diseases than is commonly supposed, yet there is reason to believe that it has an important influence on what may be termed human vitality. Room congestion when associated with ignorance and poverty, is an important factor in public health.

The elaborate devices employed in house plumbing to prevent the entrance of sewer air and drain air into the rooms have but a slight influence on the public health. They do, however, promote comfort and may perhaps enhance human vitality.

The control of the mosquito by the elimination of bodies of standing water and the use of oil is in many places an important health measure. It is also an important measure considered from the standpoint of personal comfort.

The elimination of the fly by the control of its breeding place, chief of which is manure, is, in the absence of proper care of fecal matter and other body excretions, a measure which affects the public health. Likewise it is a matter of personal comfort.

It must not be understood that street cleaning, refuse disposal and sewage treatment, which are usually controlled by municipal authorities, have no influence on public health. Of course they do have an influence. But the minor cleansing operations performed by the people themselves in their homes and schools and places of business, the practice of personal cleanliness and the exercise of such personal habits as will prevent the transmission of living pathogenic organisms from person to person through such agencies as the common use of papers, pencils, toys, drinking cups, and the like, have a greater influence. This results from the general principle that the most dangerous infection is recent infection and that in general the life of pathogenic bacteria outside of the human body is not long.

Municipal cleansing operations began before the world knew much about the transmission of disease by means of living organisms. The relation between filth and disease was suspected, however, and it is probably fair to say that most of the cleansing measures that have been in practice for many years were first undertaken chiefly from the motive of protecting the people against disease. While science has shown that their influence in this direction is less than was once thought, yet we may be justified in continuing them from the standpoint of comfort, convenience and decency. It is evident from the general tenor of current discussions and from municipal appropriations, that the general public still regards them as measures that relate primarily to health.

The safeguarding of food against infection and putrefaction is a health measure, but food sophistication is chiefly an economic question.

In general it may be said that mankind needs clean air, clean water, and clean food, and needs them, too, in sufficient amounts. The quantity of food as well as its quality is important, but this question runs away from sanitation into economics.

COST OF SANITATION AND HEALTH CONSERVATION.

In the recent special report of the United States Bureau of Census on the financial statistics of cities for 1912, figures are given which show the relative cost of those measures which tend to promote cleanliness, and which are classed under the general term sanitation, and other measures which more directly affect the health of the people. It is interesting to notice that more than four times as much money is usually spent for measures which affect the public health indirectly as for those which are directly influential in controlling disease. If we consider all of the cities of the country which have populations higher than 30,000, we find that the cost of sanitation amounts on an average to $1.42 per capita per year, while the expenses for health conservation amount to only $0.35 per capita per year. The figures in Table I show that the per capita expenses of both sanitation and health conservation increase considerably as cities become larger, but that the percentages which these expenses are of the total municipal expenses remain practically constant for all cities whatever their size, averaging 8 per cent for sanitation and 2 per cent for health conservation.

TABLE I.—COST OF SANITATION AND HEALTH CONSERVATION IN AMERICAN CITIES.

Group.	Number of cities in group.	Population.	Cost per capita. Sanitation or the promotion of cleanliness.	Cost per capita. Conservation of of health.	Per cent of total municipal costs. Sanitation.	Per cent of total municipal costs. Conservation of health.
I	9	500,000 and over	$1.75	$0.44	8.2	2.1
II	9	300,000 to 500,000	1.51	0.41	7.5	2.1
III	28	100,000 to 300,000	1.25	0.29	8.8	2.0
IV	57	50,000 to 100,000	1.02	0.24	8.4	2.0
V	82	30,000 to 50,000	0.84	0.19	7.2	1.6
All cities	195	500,000 to 30,000	1.42	0.35	8.2	2.0

If the saving of lives is to be placed above comfort and decency it would seem logical for many cities to increase the appropriation for the work of their health departments.

Very often the cleansing operations are considered as a part of the health expenditures. In the light of modern bacteriology this is unwarranted.

THE [illegible]

Let us now consider some of the standards that may be applied to the various sanitary measures. First we have what may be called the Dollar for Dollar Standard. If we in America are to be true to our caricatures we should put this at the head. We should do the things that bring dividends and keep clean and well because it pays in dollars and cents, and because it costs money to be sick and to die and to be buried. Althou[illegible] ment has moral shortcomings it is sometimes used, and it must be admitted that the dollar, as a commensurable unit, has advantages.

The money standard may be applied in two ways, first as a unit measure of cost and, second, as a unit of value applied to the product. To the first use one can hardly take exception; the second is less exact, but even so it may be usefully applied. We say that pure water saves lives; that lives have a value; and that this value is subject to calculation. In this way we compute the effect of a water filtration plant in dollars and cents and compare this value with its cost to determine whether the filter is paying a dividend on the investment. Figured in this way we usually find that the filtration of a polluted water does pay a very good profit. For example, the filter in Albany, N. Y., which was installed in 1899 reduced the typhoid fever death-rate in that city from 104 to 26 per 100,000, and also decreased the death rate from other diarrheal diseases. Taking current assumptions for the value of the lives that would have been lost had the filter not been built, and assuming a daily water consumption of 100 gals. per capita, we arrive at the result that the value of lives saved was about $76 for every 1,000,000,000 gals. of water that was supplied; and inasmuch as the cost of the process did not much exceed $10 per 1,000,000 gals., the filter may have been said to have yielded in its early days a sevenfold profit.

In the same way we may study the various sanitary and hygienic measures that relate to the saving of lives of young children, such as motherhood training by district nurses, the pasteurization of milk and the like, and here also we find that the value of the lives saved, estimated in dollars, is larger than the cost of the protective work and we can compute the profit. And so we might go through the list and find whether the money spent in the eradication of smallpox is a paying proposition and that for the suppression of tuberculosis, and whether hospitals pay and Red Cross nurses and the lighthouse service of the United States Government.

In doing this we should come across some unexpected results. For example, does the treatment of sewage pay as a life-saving agency? Most people at first thought would say "Yes," but on study the proof of this fails to materialize.

In considering sanitary expenses on the dollar for dollar basis we find that money spent for some forms of sanitation reduces other costs. Cleansing of the atmosphere from smoke reduces the laundry bills, lessens the need of air washing, and furthers the growth of vegetation. Money spent for sewage treatment may mean that streams and harbors have added value as parks and playgrounds. Money spent for water softening means less money spent for soap and the care of boilers. In various ways one expense may be substituted for another with the enhancement of personal comfort as an additional item.

Value of Human Life.—In making these calculations we are in the habit of attributing a certain value to a human life. We say, perhaps, that a man of a certain age earns so much above his cost of support, that he has a certain expectation of life and therefore his prospective value to society is so much. Economically this is doubtless true. The man's dependents recognize it, but does society recognize it? Chapin asks this pertinent question. "When a man dies do the people who are left believe that their own wages would have been higher if the man had lived, by reason of his worth to society? If they think about it at all are they not more likely to argue to the contrary and say that, in accordance with the law of supply and demand, limitation of wage-earners tends to increase wages?"

Thus, however much we may compute and argue on economic grounds, the dollar for dollar standard does not appeal to the mass of people. Bond issues for sanitary reforms will not be voted in order to conserve human life reckoned as economic wealth. The saving of life for its own sake is a far more powerful argument for the establishing of life-saving agencies than any appeal to the pocketbook. Humanity, after all, is what counts.

Again we often hear this—I quote from a prominent sanitarian: "No expenditure can be too great if a single human life is saved. Life is priceless." From an individual standpoint this may be accepted, but the world knows that in the aggregate this is not true. If people actually believed that life was priceless wars would cease, and people would stop taking chances of accident in automobiles and aeroplanes. The theory of the priceless life carried to extremes would be ridiculous. What we call life-saving is in itself something of a misnomer. Actually it is only life lengthening. This is recognized in the new phrase that we are beginning to hear—"life extension."

THE LIFE STANDARD.

There is, however, a very proper and very useful way to employ the money standard in sanitation, and that is to find in what direction the expenditure of a given sum of money will save the largest number of lives. This is one of the greatest questions that a sanitarian can consider. It is today the most important of all hygienic problems because it comprehends all others.

We spend our money for life extension partly by the inertia of old habits and partly by the spur of each new discovery in science. We take up the new ideas one by one and ofttimes push them to extremes, regardless of their relative worth. Their newness obscures their true value.

If the life-saving agencies are to yield their best results we must place them upon a quantitative basis. That is to be the keynote of the new era upon which we are entering. Some might call it "Efficiency in the conservation of life."

THE VITALITY STANDARD.

Another standard which has never yet been put into actual use relates to the effect of municipal environment on the physique and vitality. Do cities yield as strong and healthy men as the country? Does good ventilation add to one's strength and stature? Does factory sanitation lessen the discomfort and the burden of toil as well as increase the efficiency of the laborers? As yet no means have been devised for applying this standard. Some weight should be given to it in comparing the relative value of sanitary measures. Health has its positive as well as its negative side. It would not be out of place to consider expenditures for parks, for fresh air excursions and for public baths as health measures.

THE STANDARD OF COMFORT.

Money and length of life are not the only things to be considered. It is what lies between one's birth and death that really counts. Of what avail to add a few years to the length of life if personal comfort and happiness are not also enhanced! What, after all, is the chief end of man, to live long upon the earth, or to obtain the best enjoyment from the best environment? Whatever may be our philosophy we must admit that our five senses deserve consideration. Judged from this standpoint, some of the sanitary arts which do not yield a dollar for dollar profit begin to loom large.

Clean streets, clean houses, clean cars, clean rivers and clean harbors have a value because they tend to make life more comfortable, more decent and more moral. Sewage treatment, while it does not pay large vital dividends, may yet yield a profit measured in terms of comfort and decency. To what extent can physical comfort and happiness be secured in a tenement house where people live crowded together 1,000 to the acre, 43 sq. ft. of land for each person, a square 7 ft. on a side, not much larger than a respectable lot in a cemetery? The greatest evil of congestion is that true home life is rendered impossible. The greatest evil of machine labor is that the laborer gets no mental pleasure from his work. The lack of home comforts and comfortable working conditions is bound to influence both vitality and health.

SANITARY SURVEYS.

In applying these general ideas to any particular case it is necessary first of all to obtain a general view of all of the local conditions. This demands a sanitary survey.

The sanitary survey is not a new idea. Such surveys were made in many cities of Europe and America 50 and even 100 years ago. The histories of the surveys made in Boston, New York and elsewhere are of great interest and deserve careful study by modern sanitarians. In our new surveys we find history repeating itself—but with one difference. We now have a knowledge of the germ theory of disease and this has resulted in a changed point of view.

The planning of a sanitary survey should be placed in the hands of some one sufficiently familiar with all phases of the problem to give each its true value. Usually it is preferable to have it conducted with the co-operation of the municipal authorities, for they alone have access to the most important statistical material. For example, the survey which is being conducted in Cambridge, Mass., is in charge of a commission composed of the city engineer, the superintendent of streets, the health officer and two university professors. The results of such surveys are likely to show unbalanced allotment of the funds devoted to sanitary work. A slight readjustment might result in the saving of many lives. When the practice of sanitary surveying has progressed somewhat further than it has, it may be desirable to standardize the methods of expressing the results. Possibly a system of grading the sanitary conditions of cities may be devised, similar to the scoring system now applied to milk farms.

VITAL STATISTICS.

An important part of any sanitary survey is the study of the vital statistics of the community under investigation. This must not be confined to a mere study of crude death rates. It is necessary to take into account births as well as deaths and to consider the composition of the population as to age, sex, nationality and occupation, and social and economic conditions.

Perhaps no branch of science has had more misuse than that of vital statistics. In this respect we are behind, and not ahead, of the statisticians of former days. We are too much inclined to deal with crude death rates and general averages, quite forgetting that the individual observations are of more importance than the average. We do not analyze our statistical material with that degree of care which is necessary to a full understanding of the facts. We are likewise inclined to make statistical comparisons of data between which there is no causal relation. During the last generation there has been a marked decrease in the crude death rate in cities the world over. We have been too quick to assume that this decrease has been due to improved sanitation and the rise of the science of bacteriology. To be sure this has been an important influence, but we forget that a large part of the decrease has been due to falling birth rates and immigration of people of middle age and that there are close relations between death rates, birth rates, marriage rates and economic conditions. We also see it stated that the sanitary condition of a city is measured by its infant mortality rate, a fact which does not appear to be borne out by statistical evidence. The death rate from children's diseases has been steadily decreasing for a generation and this may be fairly attributed to improved environmental conditions; but it is only within a comparatively short time that infant mortality rates have declined and this decline appears to be associated more closely with motherhood training than with sanitation. In many ways it may be shown that the whole subject of vital statistics needs to be placed upon a higher plane.

Finally what is most needed is a careful correlation between vital statistics and the sanitary conditions of our cities. This demands a minute analysis of the statistical material. At present our health department reports are utterly inadequate to enable the necessary correlations to be made. In order to determine relative values in sanitation it will therefore be necessary to develop anew the science of demography, a science destined to profoundly influence the health, lives and morals of the race.

Comparative Drilling Speeds As Reported at Twenty-four Tunnels.

The rate of drilling as reported at 24 tunnels is recorded in the following tabulation from Bureau of Mines, Bulletin 57, by D. W. Brunton and J. A. Davis:

Name of tunnel.	Type of drill.	Character of rock penetrated.	Drilling speeds per machine per hour. Feet.	Remarks.
Carter	Hammer	Granite	10	Approximate.
Catskill Aqueduct:				
Rondout	Piston	Shale	8	A fair average.
Wallkill	Piston	Shale	10.5	Normal conditions.
Central	Piston	Gneiss	8-16	
Fort William (water)	Piston	Trap	2	Phenomenally hard rock.
Gold Links	Hammer	Gneiss	8-10	Approximate.
Joker	Hammer	Breccia	12-15	
Laramie-Poudre	Hammer	Granite	15	Ordinary conditions.
Los Angeles Aqueduct:				
Little Lake	Hammer	Hard granite	15.84	Average of 15 accurately timed shifts.
Grape Vine	Hammer	Granite	10	Estimated average.
Lucania	Hammer	Granite	13	Average of 3 drills.
Marshall-Russell	Hammer	Granite	10-20	
Mission	Hammer	†	30	Medium ground.
Newhouse	Hammer	Gneiss	10	
Nisqually:				
Headworks end	Piston	Rhyolite	8-10	
Discharge end	Hammer	Rhyolite	10	Rock is much harder than at other end.
Ophelia	Piston	Granite	8-10	
Rawley	Hammer	Andesite	15-20	Av'ge of 4 accurately timed drill shifts, 19.7 feet.
Raymond	Hammer	Granite	8-12	
Siwatch	Hammer	Granite	10-15	
Snake Creek	Piston	Diabase	*8-12	
Stilwell	Piston	Andesite	5-10	
Strawberry	Piston	Shale	13	Test run.
Utah Metals	Piston	Quartzite	6- 8	

Includes time used in setting up and tearing down column or bar, in shifting machine to new holes, and in changing steel, but does not include time used in mucking for set-up or in loading, blasting and clearing smoke.

*During a competition test in which both drills were mounted on the same bar in order to obtain identical conditions of rock, etc., the piston machine drilled 21 feet per hour, whereas a hammer drill made only 20 feet. The conditions were unusual, however, because water under pressure was encountered in practically every hole drilled, and doubtless influenced the results greatly.

†Shale and sandstone.

BUILDINGS

A Comprehensive Chart for Designing Reinforced Concrete Beams.*

Contributed by Ralph R. Leffler, Chicago, Ill.

The object of this paper is to ease and expedite the design of a reinforced concrete beam for moment and, at the same time, to give proper consideration to economy of design. Shearing area is considered but the details of shear reinforcement are not dealt with. The writer believes this to be the first comprehensive representation in chart form of the formulas in common use underlying the design of reinforced concrete beams containing both single and double reinforcement.

By the aid of the chart given in Fig. 2, and by the use of the methods hereinafter set forth, the design of beams reinforced with both tensile steel and compressive steel is made easier than the design of beams reinforced with tensile steel only, and the design of beams reinforced with tensile steel only is made much easier than by the methods in common use. The obscure region that exists between beams reinforced with tensile steel only and beams reinforced with both tensile and compressive steel is charted so that the designer may know his whereabouts; so also is the region in which the concrete is not stressed to its allowable working value and the tensile steel is; and further, the region is charted in which the tensile steel is not stressed to its allowable working value and the concrete is. In short the whole field of designing rectangular reinforced concrete beams is covered.

As $p' = 0$ the formulas used for beams reinforced with both tensile and compressive steel are transformed into the formulas used for beams reinforced with tensile steel only. It follows that the designing of the one type of beams properly merges into the designing of the other type.

With steel costing 2¾ cts. per pound in place and concrete costing $9.00 per cubic yard in place, the cost of a beam in cents per foot of length equals $(p + p' + 0.0247)\ 9.35\, bd + 0.2314\, bd''$. At the above rates the cross-sectional area of the concrete can be increased economically up to 40.5 sq. ins. for every square inch that the steel area is decreased.

With the commonly used values of f_s, f_c and n, the steel used as compressive reinforcement is never stressed above something less than n times f_c, which is considerably less than f_s. Therefore, since the compressive steel is, under the most favorable condition, working at only about 75 per cent of its allowable working value and since steel costs per unit volume 40.5 times as much as concrete, it follows that the use of steel compressive reinforcement is costly and should be avoided if possible.

SCOPE AND ADVANTAGES OF THE CHART.

The purpose and scope of the chart shown in Fig. 2 can most easily be grasped from a study of the chart drawn to a natural scale and shown in Fig. 1. To secure greater accuracy in the results, the most used portion of the chart has been plotted to a logarithmic scale, as shown in Fig. 2. This latter chart is comprehensive and is recommended for general use. Where reference is made in the

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text to "the chart," the one shown in Fig. 2 is meant.

The chart solves graphically the involved expressions that are encountered in the design of reinforced concrete, thus reducing the algebraic part of the designing to the solution of a few simple equations, in which a slide rule may be used. It is as useful and all sufficient for a design in which the allowable f_s is 14,000 and the allowable f_c is 500 as for one in which the allowable values of f_s and f_c are 20,000 and 800, respectively. It covers completely all of the usual variations of given conditions and requirements.

USE OF THE CHART.

The interpolation of p and p' curves often will be found necessary, but this can be done easily and the results obtained will be found amply accurate.

As has been noted, the most used portion of the chart for $\frac{d'}{d} = 0.1$ has been plotted to a logarithmic scale. Results obtained by its aid will be found of great accuracy.

The numbers shown on the chart immediately under the curve $p' = 0$ assist, through the relationship $\frac{L}{K} = \frac{f_s}{f_c}$, in finding the value of p which stresses simultaneously the tensile steel and the concrete to any desired working unit value. They also assist in finding the actual f_c in a beam reinforced with tensile steel only by simply dividing the actual f_s by the number under the $p' = 0$ curve at the curve of the p used.

To facilitate the use of the chart the following data, which will be of service when the stated values are to be used, are given:

The allowable unit values recommended by the American Railway Engineering Association are $f_s = 17{,}000$, $f_c = 750$, $v = 35$, and $n = 15$, which require for a simultaneous full stressing of both the tensile steel and the concrete that $p = 0.008784$, for which value of p, $L = 0.17268$, $K = 0.007618$, $b = 0.10572 \frac{V^2}{M}$ and $d = 0.27025 \frac{M}{V}$.

The allowable unit values recommended by the Joint Committee are $f_s = 16{,}000$, $f_c = 650$, $v = 35$, and $n = 15$, which require for a simultaneous full stressing of both the tensile steel and the concrete that $p = 0.007691$, for which value of p, $L = 0.16542$, $K = 0.00672$, $b = 0.08777 \frac{V^2}{M}$, and $d = 0.3255 \frac{M}{V}$.

The Design of Beams in Which Economy is the Governing Factor.—Given M, the allowable f_s, the allowable f_c and $n = 15$. Ascertain the maximum total shear on any section of the beam and call it V. Let v (the allowable unit shear) be 35 or 60 or 105 depending on the nature of the web reinforcement that is to be used.

Then $V = 35bd$ or $bd = \frac{V}{35}$. Adjust b (making it as small as is properly possible) and d (making it as large as is properly possible) so that with $bd = \frac{V}{35}$ the beam will be a practical one consistent in its dimensions with good designing.

Then solve equation I for K and equation II (see Fig. 2) for L, and plot on the chart the point whose co-ordinates are the values of L and K thus obtained.

If the point comes below or on the curve $p' = 0$, the concrete will be stressed below or up to its allowable working unit value. The value of p to use will be that of the p curve which intersects $p' = 0$ at the point whose abscissa is K.

If the point falls above $p' = 0$, either one of three procedures may be followed.

The first solution is to put in compressive and tensile steel reinforcement, as indicated by the p' and p curves on which the point falls.

The second procedure is to run horizontally from the point until the $p' = 0$ curve is intersected. The value of p, taken from the p curve passing through this point of intersection, is the value to use. This method avoids the use of compressive steel reinforcement, but actual cost figures are necessary to determine whether it is more costly than the first method outlined.

The third solution is to increase b and d until the point comes down onto the curve on the curve $p' = 0$, no compressive reinforcement is needed, and the value of p to use is the one whose curve intersects the curve $p' = 0$ at that point.

If the point comes below the curve $p' = 0$, then the concrete is not stressed up to its allowable working value. To find the actual f_c, run vertically up from the point to a point on the curve $p' = 0$. Read off the co-ordinates of this point. Its co-ordinates L and K are to each other as f_s is to f_c (see equation III,

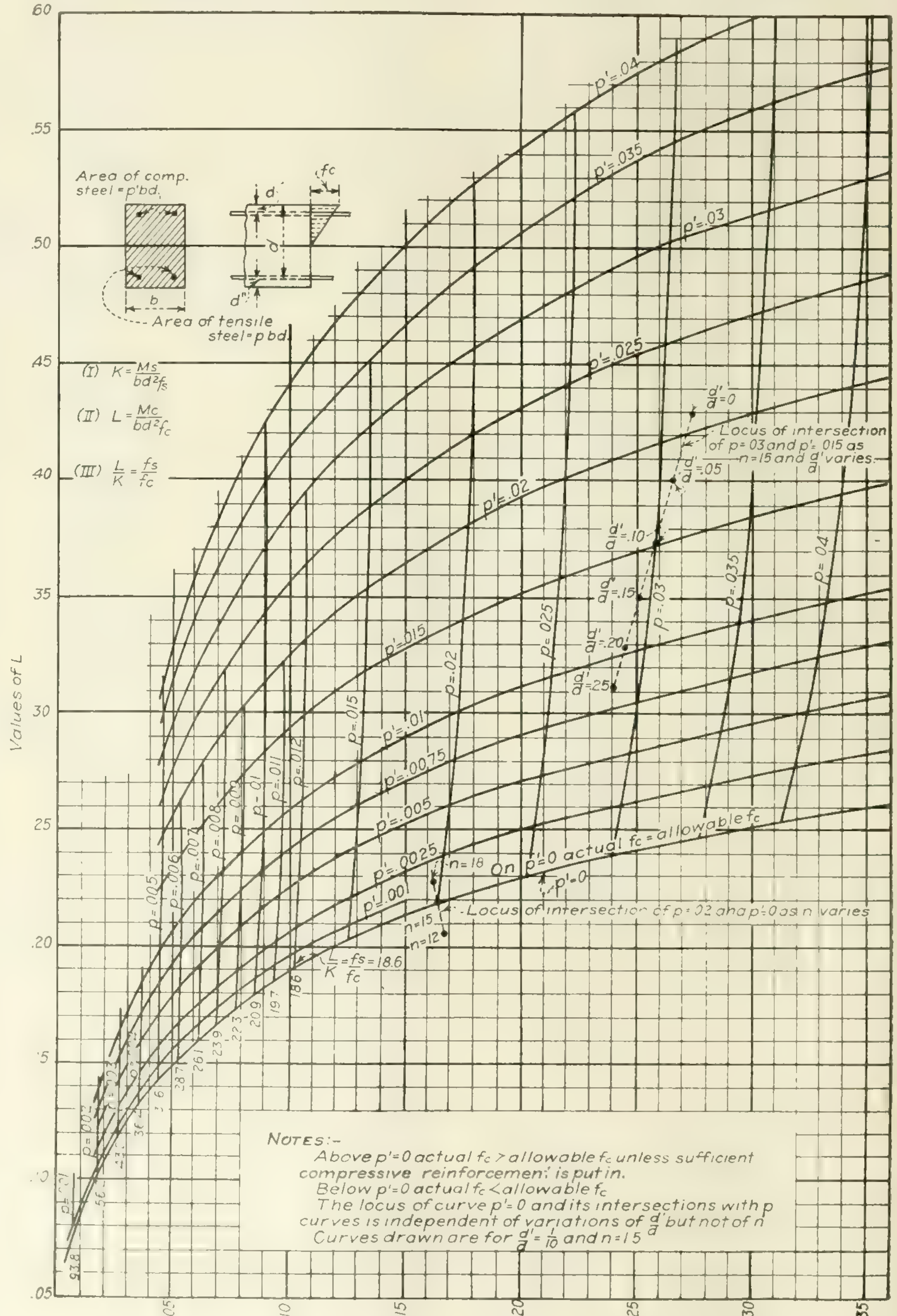


Fig. 1. Chart for Designing Reinforced Concrete Beams with Either Single or Double Reinforcement. This Chart Is Explanatory of That Shown in Fig. 2.

$p' = 0$. This is probably the best and cheapest method in practically all cases.

The Design of Beams in Which Economy is not the Governing Factor.—Example 1. Given M, b, d, the allowable f_s, and the allowable f_c. It is required to find p and p'.

Solve equation I for K and equation II (see Fig. 2) for L.

Locate on the chart the point whose co-ordinates are K and L. If the point comes Fig. 2). The value of p to use is that of the p curve passing through the point found on the curve $p' = 0$.

If the point comes above the curve $p' = 0$, the values of p and p' to use are the ones whose curves pass through that point.

If the point comes only a short distance above the curve $p' = 0$, and it is desired **not** to use any compressive reinforcement nor to over-run the allowable f_c, run horizontally

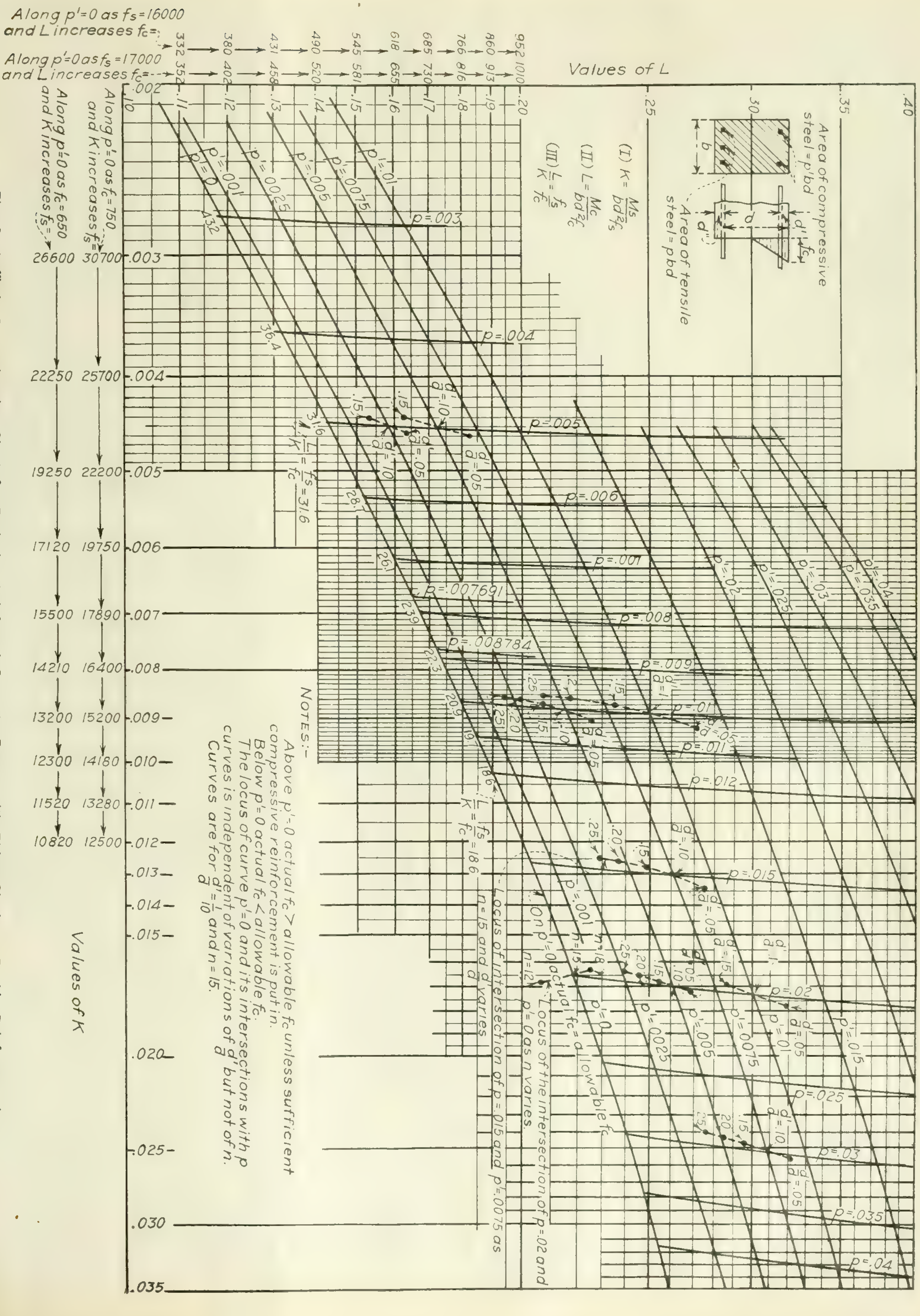


Fig. 2. Leffler's Comprehensive Chart for Designing Reinforced Concrete Beams with Either Single or Double Reinforcement.

from the point to a point on the curve $p'=0$. The value of p of the p curve passing through this point will be the value to use, and the co-ordinates of the point substituted in equation III will give the actual f_s.

Example II.—Given M, b, d, p and p'. It is required to find the actual values of f_s and f_c.

Read off on the chart the values of K and L given by the intersection of the curves for p and p'.

Then $f_s = \dfrac{M}{bd^2K}$ and $f_c = \dfrac{M}{bd^2L}$.

Example III.—Given M, b, d and the allowable f_s, also $p'=0$. It is required to find the actual value of f_c.

Solve $K = \dfrac{M}{bd^2f_s}$ Read off on the chart the ordinate L of the point on the $p'=0$ curve whose abscissa is K. Then $f_c = \dfrac{Kf_s}{L}$. The value of p to use is the one whose curve passes through the point found on the curve $p'=0$. If f_c is found to be too high, it can be decreased by changing p', or see Example IV.

Example IV.—Given, M, b, d, the allowable f_c, and $p'=0$. It is required to find f_s.

Solve $L = \dfrac{M}{bd^2f_c}$. Read off on the chart the abscissa K of the point on the $p'=0$ curve whose ordinate is L. Then $f_s = \dfrac{Lf_c}{K}$ and the value of p to use is the one whose curve passes through the point found on the curve $p'=0$. If f_s is found to be too high, it can be decreased by changing p', or by increasing p until the K of the intersection of that p curve with $p'=0$ is equal to $\dfrac{M}{bd^2f_s}$

SUGGESTIONS.

For drafting room use a pamphlet containing the discussion with charts for $\dfrac{d'}{d}=0.05$, $\dfrac{d'}{d}=0.1$, $\dfrac{d'}{d}=0.15$ and $\dfrac{d'}{d}=0.2$, made to a logarithmic scale and printed on durable paper will be found convenient. Each chart will require the solution of about 1,000 algebraic expressions of the second degree. However, the chart for $\dfrac{d'}{d}=0.1$, shown herewith, will, with the aid of judicious approximation be found sufficient for nearly all problems.

A chart for $\dfrac{d'}{d}=0$ would be found interesting. With the aid of suitable correction formulas it might be made to do the work of the charts for the other values of $\dfrac{d'}{d}$. However, it would probably prove too cumbersome for drafting room use.

Paints for Portland Cement Surfaces.

It is sometimes desirable to paint Portland cement structures with moisture-proof coatings, not only to give a more pleasing appearance, but also to protect their surfaces. Statements are occasionally made which give the impression that such surfaces are highly alkaline and cannot be decorated satisfactorily with oil paints. The following article, which was abstracted from a paper by H. A. Gardner, presented before the annual meeting of the American Society for Testing Materials, gives some data on this subject which tend to show that pure oil paints give satisfactory results when applied to Portland cement structures. The article does not refer to the waterproofing of concrete surfaces which are subjected to hydrostatic pressure.

Probably too much importance has been attached to the alleged destructive action of lime upon cement surfaces. It is well known that setting Portland cement develops free lime, but the amount to be found upon a Portland cement structure should be considered as negligible in most instances, as far as it might affect a well-designed paint. If indeed it is advisable to neutralize this small amount of lime, previous to painting the cement surface, such a result is not to be properly accomplished with an organic substance, but rather with an inorganic material which readily reacts therewith. A solution of zinc sulphate has proved most efficient for this purpose and has been used for many years with practical results, especially upon freshly laid cement. It might be well to point out that the priming or ground-coating of cement is often improperly carried out when clear primers of any kind are used. If, for instance, a clear varnish is applied to a cement surface, the primed surface is difficult to differentiate from the untreated area. Consequently, the workmen using clear primers often leave untreated laps or "holidays," as they are technically known. For this reason the use of color primers should be adopted wherever possible.

PAINTS FOR CEMENT FLOORS.

The dusting of cement floors, which is brought about by abrasion, may be effectively stopped through the use of oil-pigment paints. If the floor has been freshly laid and is damp the possibility of lime reaction may be removed by treating the surface with a solution of tinted zinc sulphate. The oil paint may then be applied. Boiled linseed oil, sometimes mixed with Chinese wood oil, may be used as the liquid portion of the paint. These oils have a remarkable binding action when applied to a cement surface. When mixed with pigment they form paints which are eminently suited as first coaters for cement floors. The first coat will dry rapidly and form a dense surface. Over this may be applied a second coat and, if a high gloss surface is desired, a portion of varnish may be added to this final coat. The various floors of the author's laboratory were treated in this manner, with prepared floor paints made from such materials, one week after the placement of the cement. The zinc-sulphate primer was used only upon the damp areas. The floors have since been subjected to much abrasion from constant walking and the moving of heavy apparatus. Oils and chemicals of various kinds have come in contact with the floors, and soap and water have been used upon them very often for cleaning purposes. After three years' service the floors have not dusted and have never required repainting. The paint films are still in good condition. This service record would tend to show that placed Portland cement may be made dust and wear-proof and highly desirable as a flooring material, through the application of oil paints.

DESCRIPTION AND RESULTS OF EXPOSURE TESTS.

In April, 1912, the writer instituted a series of tests to determine the durability of various types of paint upon Portland cement surfaces exposed to the weather. The panels for the tests were prepared by constructing a long board wall to which was fastened expanded metal. A mixture of 1 part Portland cement and 2 parts clean Potomac River sand was made and applied to the expanded metal, forming a cement wall 3 ins. in thickness. The wall was divided into 35 sections, or panels, each 30 ins. wide and 40 ins. high. Three coats of paint were applied to each panel by a practical journeyman painter. In order to make the test more severe nearly all of the paints were applied in white. (Tinted paints are known to be much more durable than white paints.) A stripe of chrome green, 6 ins. wide, was placed over the top of the third coat of paint, to determine whether the lime which might be present on the surface of the cement would have any effect upon the paint coating. Fading of the green to a yellow would indicate such action.

The general results of the tests at the end of a two-year period, together with an outline of the composition of the paints tested, is given as follows:

Class No. 1.—Single-pigment paints made with white lead or zinc oxide ground in pure linseed oil.

These paints are in very good condition throughout.

Class No. 2.—Combination-pigment paints made of mixtures of white lead, zinc oxide or similar pigments ground in pure linseed oil.

These paints generally are in excellent condition.

Class No. 3.—Combination-pigment paints ground with mixtures of raw and heavily-bodied linseed oil or with treated Chinese wood oil. The viscosity of these oils requires the use of considerable turpentine or other thinner in the manufacture of such paints in order to make them of the right viscosity for application. Semi-flat surfaces are therefore produced during the drying.

Most of these paints are in excellent condition.

Class No. 4.—Single and combination pigments ground in oil varnishes containing acid resins.

These paints are checking and scaling in many spots. Such varnish paints apparently are not suited to exterior exposure.

Class No. 5.—Paints containing resins dissolved in volatile spirits (spirit varnishes), with or without pigments.

These paints are not giving very satisfactory service, the clear varnishes having entirely decayed in some cases. Those to which pigment has been added are in somewhat better condition.

Class No. 6.—Paints made with single and combination pigments ground in a water medium containing glue or casein as a binder.

These paints are chalking rapidly and are not moisture-proofing the cement. The pigment binder has been destroyed by the weather.

CONCLUSIONS.

Opaque white pigments, such as basic-sulphate white lead, basic-carbonate white lead, zinc oxide, and lithopone, were present in the paints which gave the best results. In some of these paints there was also present a percentage of inert pigments, such as barytes, asbestine, whiting, china clay, gypsum and silica.

The results of these tests are in agreement with those obtained by Ware and Schott in a series of paint exposure tests made upon exterior concrete surfaces. They also agree with previous long-time exposure tests made by the author.

As a result, therefore, it can be stated that concrete surfaces may be decorated with excellent results through the use of high-grade oil paints. When the cement surface is freshly laid and is damp such paints may be safely applied after treating the cement with a zinc-sulphate primer.

The Economical Design of Factory Buildings.

On account of the apparent simplicity of some types of factory buildings their design and the arrangement of their machinery and equipment have been given little consideration. It is only after the plant has been in operation for some time that the importance of carefully planning such buildings is fully realized by those in charge. There has probably been a greater waste due to insufficient factory building construction than in any other type of construction. The following article on the economical design of factory buildings was abstracted from a paper by W. E. King, presented before the Civil Engineers' Society of St. Paul. The article gives emphasis to a number of features which generally receive too little attention.

The economical design of factories involves the solution of a large number of simple engineering problems. It is the intention of this paper to discuss these problems in their relation to each other in order to give a general view of the economics of this form of construction.

Factory design requires a broader knowledge of engineering than for most types of buildings. For this reason it is the work of an industrial engineer rather than an architect.

The industrial engineer should be an engineer with a fair working knowledge of architecture, civil, structural, mechanical and electrical engineering. It is essential to the economical design of any building that is to house machinery and human beings doing work which involves repetition of the same act that the designer of that building should have a very clear working knowledge of all the machinery installed and the processes which are to be carried on in the finished structure.

In organizations which do a considerable industrial engineering business it is customary to maintain engineers familiar with the various manufacturing processes. Such an organization usually maintains mechanical, electrical, civil engineering and architectural departments.

A purely engineering organization will tend to minimize the importance of the architectural requirements of the work devoting its time to the considerations of economy of construction and efficiency of operation. Both phases of the question should be considered in the design of any building, but certainly the esthetic considerations in the design of a factory are not as important as the parts of which the study must necessarily be made by an engineer.

Whatever may be the title by which the designer of such a building be called, it is at least fair to state that all parts of the structure should actually be designed in his office and such work be paid for by him as a part of the service for which his clients pay him. The practice of some persons in the business of forcing the contractor for structural steel, or reinforced concrete, or other parts of the building to prepare the designs is certainly unfair to the client. There are other disadvantages in allowing the contractor to do the engineering.

LOCATION.

The work of the industrial engineer should commence, if possible, before the location of the factory is determined upon. The first consideration is, of course, the proper locality for the industry. This is largely an economic problem which the engineer may or may not be required to study.

The cost of any manufactured commodity to the retailer consists of the following items: Cost of raw materials, cost of the transportation of these raw materials to the factory, cost of labor on materials, cost of power, overhead charges, including interest on money invested, depreciation of plant, insurance, office time and advertising, cost of distribution and profit to the manufacturer.

Assuming the price to be received for any finished commodity to be fixed by competition, then that project which will pay the largest profits, is, of course, the one where the sum of the first six charges is a minimum. This does not necessarily mean that any one item should be reduced to a minimum, but that the sum of all the items taken together is the least possible. The usual difficulty is that some one man almost always plans each project with the idea of reducing some one item to a minimum. For instance, a man who had spent the larger part of his time in handling of workmen will insist that the plant be so located that there will be an abundance of cheap labor. If he had at one time been a purchasing agent he would plan his plant to save all freight possible on raw materials. The sales manager is interested in the location of the factory with respect to the market. The man who furnishes the money is sometimes unduly interested in cutting the first cost down to a minimum, without regard to whether the interest on his money might be larger if more money were invested.

It should be the duty of the engineer to study these questions and to present them so that they will occupy their proper rank of importance. This rank is, of course, different in different kinds of factories. In the fabrication of structural steel, for instance, perhaps the most important factor is freight. This includes freight from the rolling mill to the factory and freight on the finished product from the factory to the consumer. In some parts of the United States this freight amounts to more than half the cost of the finished product. However, in the manufacture of candy, for instance, the freight is of small importance and the proximity to the market and the cost of labor is of greater importance.

USE OF CHARTS.

Where freight is one of the chief considerations, charts may be prepared showing the zone in which a product may be profitably marketed. The boundary of each zone will be determined by considering the sum of the freights on raw and finished materials for the proposed location as compared with other possible locations. The properly prepared chart will show the overlapping territories where competing factories sell on an equal basis. It will show the area where the factory in question has the advantage and it will also indicate the areas which cannot profitably be reached.

The matter of available market probably reaches its greatest importance when the capacity of a profitable factory is about to be increased. It may be that the selling organization is now reaching all of the profitable market zone and that to increase the sales the product must be marketed at a disadvantage.

Some industries use large amounts of fuel or power which requirement is the determining factor in their location. Many of our rolling mills are located at Pittsburgh because of available coal supply. The cheap water power at St. Anthony Falls partly determined the location of our flour milling industry. This tendency to group factories around water power sites will probably not be as marked in the future because our modern methods of electrical transmission allow power to be delivered economically at a considerable distance.

Having determined on the vicinity where the factory is to be built the next consideration is the purchase of the exact site necessary for the project. The exact area of land which is necessary is usually a troublesome one. Most successful projects are hampered by lack of room to provide for their growing needs. On the other hand, it is a serious handicap to a young industry to be burdened with heavy interest charges and taxes on land not at the time in use.

The size and shape of the area necessary for present needs is usually determined by making a preliminary plan of the whole project. If the engineer is unfamiliar with the need of the industry in question, this will usually involve quite an extended study of the methods of manufacture used by the particular organization and of similar organizations in other places. If, however, the designer has already prepared plans for other similar plants the tentative preliminary plans involve only a study of the peculiar requirements of the special case. This preliminary plan should take in the reasonable growth of the industry, which usually may be approximately obtained by a comparison with similar industries in other communities. With the approximate area required clearly in mind a search of the locality will usually show a number of available sites. For projects of some importance plats are usually prepared showing how the proposed sites may be developed. These plats should show the approximate grades and elevations of all adjacent streets, the location of sewer, gas, water and power connections and the available connection to adjacent railroads or sidetracks. If grading of streets or of the lots will be necessary this should be estimated and added to the comparative price of the lots. For the purposes of comparison the cost of the sidetrack, including necessary grading, the cost of sewer, water, gas and power connections should be obtained. Very often the owners will buy a lot first, without considering the cost of those items which must be added to make the lot available, and in so doing they fail to get the most economical site. Plats showing the proposed sidetrack should be submitted to the railways interested and assurance should be had from their engineering and contracting departments that they are willing to put in the desired connection. If the side track must cross the public road it is well to be sure of the permit before putting money into the lot.

SIDE TRACKS.

At this time a study should be made to determine the number and length of side tracks which will be required. In general the track should be long enough to hold as many 40-ft. cars as the company will need to load and unload in any one day. In isolated places where the cars are not set as often as that, the side track must be long enough to allow for all the unloading and loading which must be done between each setting by the switch engine. Side tracks for loading and unloading should in general be level. The rules of the railroad in question, of the state and interstate railway commissions, and the state labor laws are the determining factors in the amount of room required for side tracks.

BUILDING PLANS.

After the exact location of the site is determined then the plans of the buildings may be prepared. It is, of course, a mistake to make the final plans of any building before its definite location is settled. The natural grades of the land itself, the streets, the points of the compass, and the condition of the subsoil almost invariably change the plans to such an extent that they must be revised or redrawn. All of these conditions should be determined by an exact survey before work on the plans commences.

As before stated, the basis for the design of the factory building should be a complete understanding of the processes to be carried on in the building. Too many factories are built first and the machinery just put in, one piece at a time, after the building is completed. This usually results in the uneconomical use of the floor space; unusued spaces occur in some parts and a congested condition results in other parts.

The first plans to be prepared should be complete machinery plans. A study should be made of the progress of the materials through the shop. In general the manufacturing processes should be so arranged that there will be no lost motion. The various materials which go to make the finished product should all travel through the various parts of the factory in such a way that they will arrive at the assembling room without having traveled any greater distance and without having been transferred more times than is absolutely necessary. After leaving the assembling room the materials should go by the shortest possible route to the storage and shipping rooms. This part of the work is best planned with the prospective superintendent of the shop. It is sometimes difficult to get the benefit of this man's detailed knowledge and experience without letting his narrowness of viewpoint blind the designer to the broader phase of the question.

As a rule a good factory superintendent has spent the larger part of his life in some one factory. He probably has made that factory a success. That leads him to think that he knows all there is to know about that business. At least he thinks he knows more than any engineer whom the owners can hire. That is generally true, but his difficulty is that he is so close to his job that his perspective is warped. For instance, if ten years ago he tried a belt conveyor in his factory which he bought and installed improperly himself, and then afterward abandoned because it did not do the work required, he is convinced that he does not want a belt conveyor in his new factory. The fact that belt conveyors have been improved since he tried them and that there are thousands of them working satisfactorily under similar conditions, will impress him only if you can overcome his prejudices. If you can make him feel that he and the engineer are working together to get the best possible design and that you realize the value of his suggestions, then, generally, it is possible to get him to listen to yours.

It is a common thing to have one of these so-called "practical men" say: "You build me a building so big and I will put the machinery in myself, without any plans." Even with a man competent to do this, there is considerable advantage in arranging the machinery on paper first. It is easier to see where the waste spaces in the room occur and to correct them on paper than it is with the full-sized building. Then, too, these machines usually have foundations, shafts, electric wiring, and these things have a tendency to interfere with each other and to require the cutting of unsightly holes in the completed building.

The building should be built to suit the machinery. The columns and beams, the height of stories, the location of heating and plumbing pipes, the sprinkler system, and the natural and artificial lighting should all be arranged to suit the machinery.

The economical arrangement of the structural parts of the building should also be taken into consideration in the arrangement of the machinery. If possible the columns should not be spaced to suit special machines unless there is some very decided advantage in doing so. It must be remembered that the life of a building is several times the life of the machinery installed and that the machinery of the future may be entirely different.

TYPES OF FACTORY BUILDINGS.

There are two types of factory buildings which are here considered separately. The first is the ordinary one-story building with a hip roof which may or may not be surmounted by a monitor. It usually has large unobstructed floor space to provide for the movement of cranes and other large machinery. The second type is the warehouse type of one or more stories in height. Industries which require a clear floor space of more than 25 ft. in either direction are usually housed in one-story buildings, because it is expensive to carry the weight of upper floors on long spans. Where the materials manufactured are of such size that columns spaced from 16 to 25 ft. on centers are not objectionable the building of several stories is usually more economical.

A one-story building costs the most per square foot of floor area. This cost per square foot decreases somewhat with the number of floors built, up to four stories. Above that height the cost per square foot gradually increases. There is comparatively little difference in the cost per square foot of floor area between a three-story and an eight-story building.

If basement floor space is suitable it is the cheapest which can be obtained, except where the loads to be carried on the first floor are extremely heavy. A one-story shop building in fireproof construction will cost from $1.25 to $2 per square foot of floor area, depending upon the height of the story, depth of footings, lengths of spans and kind of exterior finish used. Fireproof buildings of more than one story may be built for as little as 50 cts. per square foot of floor area. These approximate figures do not contemplate any sort of plaster or interior finish except whitewash. They do include a properly finished cement floor. The cost per square foot of course decreases as the size of the ground plan increases. It is more for a long, narrow building than for a square building. However, a factory building should not be made too wide on account of the difficulty in properly lighting the interior. For ordinary factory work from 40 to 50 ft. is the best width. A building of this width can be lighted with a story height of from 12 to 14 ft. If the width of the building be made from 75 to 100 ft. then the story height should be increased to from 14 to 16 ft., the windows being placed as high as possible.

One-story shop buildings are usually built of what may be termed semi-fireproof construction. They are usually built of materials which will not burn but cannot be said to be entirely fireproof because the steel trusses are usually left unprotected, so that they might be damaged in case of fire occurring in the contents of the building. As before stated, the one-story plan is usually adopted where large, unobstructed floor spaces are required. This results in long-span steel trusses supporting the roof.

The most common type of roof is the "A" shaped roof. This roof has many advantages. It is easy to keep water tight, it clears itself of snow easily, and with monitors or ventilators at the peak it provides good ventilation for the factory. If these monitors are made wide enough and are provided with windows they admit considerable light, but if the building is high and wide, monitor windows usually do not admit a satisfactory light.

A better type of roof, where light is essential, is the saw-tooth roof. This roof is made up of a series of pitched roofs, rising towards the north and stepping down with a vertical step, in which windows are installed. These windows, facing towards the north, admit a diffused light which illuminates the floor below without casting shadows. If the windows in the saw-tooth construction are arranged to swing, they provide as good ventilation as the old monitor type. The disadvantage with saw-tooth construction is that it presents a number of valleys where snow may lodge. In some cases steam pipes have been installed to melt the snow. This serves the purpose, but it is rather expensive. In buildings where there is considerable steam in the air condensation gutters are necessary under monitor and saw-tooth windows.

THE ROOF.

The most unsatisfactory problem in shop building is probably the roof. It first must be water tight; second, if the building is to be heated in winter it must be of such material that condensation will not occur on the under side; third, it should be fireproof; fourth, it must compete with a large number of cheap roofs which are lacking in one or all of these qualifications. A standard roof construction consists of 3-in. hollow book tiles laid on steel T-beams. These tiles are covered with some good prepared roofing which is cemented and tacked to the tiles. This roof is very expensive, but it fulfills all the requirements stated above. It costs, including supports, about 30 cts. per square foot.

Another good roof consists of 2-in. dressed and matched sheathing, laid on wood or steel purlins and covered with a good prepared roofing. It is practically as good and much cheaper than a book-tile roof, but, of course, is not fireproof. It will cost about 20 cts. per square foot, including supports.

In some cases a thin concrete slab laid on steel or concrete purlins has been used. Considerable condensation occurs under such a roof in cold weather. Furthermore, it is very difficult to keep a thin roof slab from being damaged by frost while being laid in cold weather.

If the shop is not to be heated in winter corrugated iron laid on steel purlins makes a very inexpensive fireproof roof, costing about 12 cts. per square foot in place, including supports. It is fairly water tight but, of course, is very cold in winter.

There are, of course, many other kinds of roofs, but the price for any roof comes between the limits here given.

There is not so much choice in the materials for constructing the side walls of a building as the roof, although they may be of brick, stone, concrete, corrugated iron or glass. Buildings with high stories are usually made with steel frames, the walls being simply curtain walls, bricked in between the columns. Hollow brick should be used for the inside layer to prevent condensation of the side walls.

Concrete for side walls is often more expensive and is less satisfactory than brick. Concrete blocks are sometimes used and are satisfactory where enough cement is put in the blocks. Such walls are, however, weak, due to the lack of bonding between the blocks.

A 12-in. common brick wall in this part of the country will cost about 38 cts. per square foot in place. With a good facing brick and some architectural decoration the cost may be increased to from 40 to 60 cts. per square foot.

In the modern factory building the selection of the material for the outside walls is not such an important feature because from 75 to 100 per cent of the wall area is occupied with windows and doors. The old style shop building did not, as a rule, admit enough light. Some of our new buildings probably admit too much. It is a mistake to assume that a workman needs as much light to work by as there is out under the open sky. Too much light is almost as bad for the eyes as too little. Most of the inconvenience of working indoors comes from working with a strong light from one side which casts shadows. Windows should be so arranged that light will reach every point from at least two directions and be of as near the same intensity in both directions as possible.

Another question upon which there is usually some argument is the kind of windows to be used. The three types most used are the standard wooden sash, the rolled steel sash and the fire underwriters' sash of sheet steel or copper. The underwriters' sash is very little used for shop buildings because of its high cost. It will, however, greatly reduce the insurance rate upon such walls of the building as have a bad fire exposure. Underwriters' sash cost about $1 per square foot, glazed with ¼-in. wire glass and set in place.

The most satisfactory sash at the present time for factory work is the rolled steel sash. Where large areas are to be glazed the small size of the steel muntins and mullions permits the maximum amount of light to enter the building. Several factories have been built with the side walls almost entirely of glass, the only obstructions in the walls being the columns and the brickwork at the floor line. The cost of steel sash set in place and glazed with double-strength glass runs from 40 to 45 cts. per square foot, depending upon conditions. A steel sash has a few disadvantages which should be taken into consideration. The ventilation is usually secured by pivoting a part of the sash near the middle. In factories where screens are necessary it is not possible to have ventilation because the screen will not permit the ventilator to swing. In northern climates a storm sash is desirable because of the loss of heat through the glass by conduction. A steel sash is too heavy and too expensive to use for storm sash. If wooden sash be used the advantages obtained by the use of steel inside sash are lost.

The cheapest sash to use is undoubtedly the double hung wooden sash with which we are all familiar. Its cost is from 25 to 50 cts. per square foot, in place. It allows of 50 per cent opening for ventilation. The details of window openings and their connections to the remainder of the building have been worked out through long use and can be made to fit any opening without additional cost. Where the window areas are large the wooden mullions and muntins must be made of considerable size for strength. This results in a considerable obstruction of light.

GENERAL CLASSES OF FACTORY BUILDINGS.

Considering factory buildings of more than one story, they naturally divide themselves into four general classes, according to the materials of which they are constructed. This classification is really made by the fire underwriters inasmuch as the different types take different insurance rates. In fact, the rate of insurance is the consideration which most often determines the type of construction.

These classes are: Frame construction; slow burning timber construction; structural steel; and reinforced concrete.

In the frame construction class should be included all buildings having either brick or timber walls, wherein the floors are of wood and the joists are narrow and closely spaced. Such buildings are, of course, the cheapest which can be built. By far the larger number of the present factory buildings are of this type. When an industry is in an experimental stage, and where the processes of manufacture and the machinery are likely to be changed with experience, it is more economical to build in this manner. If a building is anything more than a temporary structure the extreme fire hazard, the danger of employes upon the upper floors, and the lack of rigidity for supporting machinery are disadvantages which should be taken into consideration.

The insurance rate for frame constructed factories without sprinkler systems varies from $1.25 to $2.50 per year per $100 of insurance. The value of the contents of the building may equal that of the buildings, and the rate on the contents will usually not be less than on the building.

If the frame type is to be used the insurance rates may be reduced about 60 per cent by the installation of the standard sprinkler equipment. This equipment will cost from 5 to 15 cts. per square foot of floor area, depending upon conditions. That is, in a cheap building the cost of the sprinkler equipment will add to the cost of the building from 10 to 20 per cent, but this cost is usually paid for in the first few years by the decreased insurance rates.

There is another loss to be considered which cannot be covered by fire insurance, namely, the loss of profits due to the interruption of business. This includes not only the current profits but also the loss of customers. In a growing concern this loss is generally more serious than the damage done by the fire.

In the slow-burning mill building construction, as described by the fire underwriters, the walls must be of brick or stone. It differs from the frame construction in that the joists are spaced from 3 to 6 ft. apart and are timbers of considerable size. The floors are matched planking. All stair and elevator hatchways must be enclosed, with doors at each floor opening. This construction is somewhat more expensive than the frame construction. Its principal advantage is that it takes an insurance rate about 20 per cent less than frame construction. If a sprinkler system is installed this rate may be reduced to about 48 cts. The first cost of the building will be from 10 to 30 per cent more than for frame construction, with brick walls.

Any timber construction has several advantages over more permanent types, such as concrete or steel. Alterations in the buildings, due to changes in processes of manufacture and to the installation of new machinery, are much more cheaply and rapidly made. The expense of attaching shafting and machinery to the finished structure is considerably less. Wooden buildings are much more rapidly constructed than either reinforced concrete or structural steel buildings.

The columns in buildings with wood beams should be spaced from 12 to 18 ft. on centers. If a greater column spacing than this is required it is usually more economical to make the beams which span the longer direction of steel. These beams may rest on cast-iron or steel columns, the remainder of the construction being of wood.

A better construction consists of steel columns and beams throughout. The floors may then be made of reinforced concrete or tile. If the columns and beams are then covered with fireproof material such as tile or concrete the building may be regarded as the best type of building which modern civilization has produced. It is also the most expensive, costing from 5 to 20 per cent more than a reinforced concrete building. In such a building the steel columns do not occupy so large a percentage of the floor area as do concrete columns. Exact stresses in a steel frame building are more easily computed. The chance for variation in the strength of the material due to faulty workmanship or design is not nearly so great. Alterations of the building are more economically made in a steel than in a concrete building. The insurance rate for such a building is compared with a frame building, taking a $1.50 rate, would be about 90 cts. per $100 without sprinkler system. With a sprinkler system this rate would be reduced to something like 36 cts. per $100. In fact, many buildings of this type take a rate of from 10 to 20 cts. per $100.

The most popular type of factory building in the vicinity of St. Paul and Minneapolis is reinforced concrete. In this respect we are way in advance of other localities. In fact, we probably have larger proportion of reinforced concrete factory buildings in the Twin Cities than any other locality in the United States. This is largely because there are a number of contracting engineers of unusual ability in this vicinity who are financially interested in the sale of the reinforcing steel entering into these buildings. Their influence is evident in the building ordinances of the Twin Cities, which allow higher unit stresses in both the concrete and the steel than are common in the cities of the east.

A properly designed concrete building is the best building which can be put up for many industries. It is entirely fireproof and takes the same rate of insurance as a fireproof steel building. Such buildings are probably the most rigid type which can be constructed. The material will stand a large amount of abuse in the way of faulty workmanship and design. Other types of buildings deteriorate with age, but a properly constructed concrete building increases in strength.

The floor spans of a concrete building may economically be made from 16 to 24 ft. The exact span for minimum cost depends upon the expense of the foundations. The more expensive the foundation piers the longer may be the economical span. We find that the flat plate type of column pier is considerably less expensive than the old-fashioned masonry piers of the pyramid type.

The statements made concerning the exterior walls of one-story buildings are true in regard to buildings with a greater number of stories. It is usually economical to build self-supporting, exterior walls for buildings up to three stories in height. For buildings higher than three stories the walls are often made 12 ins. thick and are carried upon the steel or concrete frame of the building.

DRAINAGE AND IRRIGATION

Methods and Plant Employed in Pumping 400,000,000 Gallons of Water and Mud to Drain Kerr Lake, Cobalt, Ont.

The principal operations described here are outlined by Robert Livermore in Transactions, American Institute of Mining Engineers, for July, 1914.

Kerr Lake originally covered 45 acres. Of this the Kerr Lake Mining Co. owned 12, the Drummond Mine 7, and the Crown Reserve Mining Co. the remainder of 26 acres. Since the latter's property was originally entirely under water, it was necessary for the owners to make land for buildings and shaft room. Accordingly, in 1908, a trench was blasted out which deepened the outlet, and lowered the lake 8 ft.

As many of the rich veins of the Crown Reserve and Kerr Lake companies were under water, mining was pursued under some disadvantages. Fortunately the rock is tight and solid, and mining has been done sufficiently far from the surface to avoid unnecessary risk. Nevertheless, there was always danger of encountering open seams, through which too large a flow of water for comfortable working might come. Careful soundings were made over the veins, by means of steel-shod pipe, through mud and water to bedrock, but there was sometimes an element of uncertainty as to whether actual bedrock had been reached. Furthermore, the necessity of leaving safe backs between workings and lake tied up large quantities of ore, and made development in certain directions hazardous. The desirability of complete draining of the lake was early recognized, and several plans were proposed for its accomplishment, but owing to various difficulties, both of an engineering nature, and of securing agreement between the various companies affected, did not come to fruition.

During the summer of 1912, the Crown Reserve and Kerr Lake companies gave serious thought to the subject, and came to the conclusion that if the dewatering were to be done, it could best be accomplished by pumping rather than by an earlier plan of tunneling, both on the score of simplicity and expense. Preliminary surveys were made over the route the water was to follow, viz.—through the Kerr Lake outlet, to Glen and Giroux Lakes, and thence by the outlet stream of the latter to the Montreal River, a total distance of 8 miles.

In April, 1913, an act of Parliament permitted the draining of such bodies of water as were an obstruction to mining operations, which act removed one of the hindrances hitherto existing to this, and similar plans: and in May, 1913, the purchase of the 7 acres owned by the Drummond interests, under Kerr Lake, removed the last vital objection to the undertaking. Permission was granted by the mining commissioner in May, 1913, and work was begun at once.

Kerr Lake at this time covered an area of 30.35 acres having been reduced from the original area of 45 acres by the work of the Crown Reserve Co., above mentioned, and by filling in by the waste dumps of the two companies. Of the total area 18.5 acres belonged to the Crown Reserve, 6.54 to Kerr Lake, and 5.31 acres formerly of the Drummond Mine, to the two first-named companies jointly.

There are no inlets of importance in Kerr Lake, and its one outlet carried off water running at the rate of 300 gals. per minute in the freshet season to nothing in the dry season. It was not thought that the lake was spring fed to any extent, but that it maintained its level simply from rain and melting snow. Soundings had established its greatest depth at 100 ft., nearly 20 of which was soft mud. It was estimated that the lake contained 400,000,000 gals. of water and liquid mud in all. In any plan for dewatering, the mud had to be taken into account, as a large deposit of this left behind would leave parts of the exposed surface in as bad shape as ever for mining purposes.

It had been planned in the first surveys to pump water through the old outlet, whence it would run through natural channels by Glen and Giroux Lakes to the Montreal River, but while this would have been a simple and feasible plan for the water only, the problem involved by the proper disposal of the mud prevented its adoption. Hence the final surveys were run in a direct line from Kerr to Giroux Lake. A pipe was to take the water over this line, crossing the Kerr Lake property, the township highway, the tracks of the Temiskaming & Northern Ontario Ry., and several rights of way of power and compressed-air lines, before entering Giroux Lake. The greatest elevation of the line above Kerr Lake was 53 ft., and the linear distance from lake to lake, 2,400 ft. The difference in elevation between the two lakes was 20 ft.

Giroux Lake covers about 230 acres, and is of great depth, with an ample outlet, so that the disadvantages and objections of possible blocking of channels and flooding of other properties met with in the original plan, were done away with.

The problem which then presented itself was the installation of a pumping plant capable of handling both water and an indeterminate amount of solids against a static head varying from 53 ft. at the start to 153 ft. at the end of operations, when the lake should have been completely drained, through a pipe line large enough to eliminate the friction factor as much as possible, yet small enough to give sufficient velocity to mud-laden water. Allowing an ample friction factor, it was estimated that the total static and dynamic head would approximate 185 ft. The pumping plant, further, must have a variable base since the shore line would be constantly changing, and the pumps must have a great range of action to maintain an even flow of liquid of changing density against an ever increasing head.

PUMPS PLACED ON SCOW.

On account of the steep and irregular shore and bottom of the lake, and the cumbersome nature of the machinery, a plant mounted on a movable base ashore presented obvious disadvantages, so it was decided to place the pumps upon a scow. This scow was to be kept near the shore because if, anchored in mid-lake a good many pontoons would have been necessary to support the heavy pipe; also, since flexibility of the pipe line at some point was essential, on account of the changing level, it would have been difficult to anchor both scow and pontoons so as to be flexible enough and yet not too susceptible to wind and movement of the waters. Further, constant changing of mooring cables at many points would have been necessary as the level of the lake changed.

The scow on which the pumps were to be mounted was designed to be as compact as possible, yet to allow plenty of working room when machinery and fittings had been installed. Stability, rigid construction, light draft, and carrying capacity up to 70 tons displacement were all essential, and were taken into account in the plans.

The hull was built of Western fir throughout, dimensions over all being 40x20 ft., depth of sides 4 ft. and dimensions of bottom 31 ft. 6 ins., allowing a 4-ft. 3-in. overhang at bow and stern. For the bottom, 3-in. planks laid lengthwise were used; across these other 3-in. planks were laid at 2-ft. 3-in. centers as sills for the uprights supporting the deck timbers. The boat was divided longitudinally into three sections; the sides, the center, and the two intermediate divisions of the framework were built of 6x6-in. timbers at top and bottom, each division spaced 4 ft. 9 ins. apart. The uprights were 6x6-in. posts set on the bed planks at 2-ft. 3-in. centers and bolted to the longitudinal timbers. The deck beams, also 6x6 ins., rested on the latter at 2-ft. 3-in. centers, and were bolted to them and to the supporting posts. Deck and sides were of 3-in. planks laid lengthwise, and spiked and bolted to the frame. A 2-in. flush was given to the deck, and hatches provided fore and aft for entrance to the hold. The whole boat was thoroughly calked with tar and oakum. Two anchoring spuds were placed at each side of the stern, for holding the scow firmly against the thrust of the pumps. These were 16x10-in. fir timbers, 30 ft. long, iron shod, and held in place by guides 14x10 ins., which were tied by iron rods to each other above and below water. The spuds were raised and lowered by rack and pinion with spoke attachment. Beside the spuds ⅝-in. wire hawsers were provided, to connect with shore at each corner of the scow. In practice it was found that on account of the shifting nature of the mud, and the difficulty of finding firm bottom for the spuds, the hawsers were more useful, and with the aid of small yacht capstans set up on the boat and on shore, were ample to hold the scow in any desired position.

After launching, which was accomplished without trouble, by jacking up the boat to the proper angle, and placing greased ways beneath, it was towed to the east end of the lake and ballasted with about 10 tons of bagged gravel. A temporary crane was rigged on deck for handling the pumps, motors, and heavier pipe and valve fittings, which were hauled by wagon from the cars, and shipped on board at this point. The pumps and motors were assembled on their base, the bed plates of each unit bolted through 8-in. sills to the deck, and the pipe and valve connections between the pumps made. The scow was then towed back to the yard for housing and final fitting. A weather-tight house of light construction was erected, to cover all of the deck except a small space at the end, where room was needed for operation of the spuds and capstans.

The plant consists of four single-stage centrifugal pumps, arranged in two units (Figs. 1 and 2). Each unit comprises a compound pumping outfit of two pumps, direct connected by a flexible coupling to the motor which is placed between the two. These have 12-in. side suctions leading outboard, and 10-in. discharges facing inboard. The pump shells are 1½ in. in thickness, with removable side disks. The runners are of the inclosed type, of heavy design, and capable of delivering solids up to 4 ins. in diameter. The shafts are of steel extended on each side of the pumps, and carried by outboard ring oiling bearings, fitted with removable babbit-lined shells. Adjustable thrust bearings are also provided to take possible unbalanced end thrusts.

Each unit was designed to deliver not less than 3,000 gals. per minute at the greatest elevation encountered during the operation, with a mechanical efficiency of not less than 60 per cent. At the start the four pumps were to work in parallel (Fig. 1), each delivering through its 10-in. discharge into a central pipe, the flow from the two sternmost pumps carrying through a length of 14-in. pipe until abreast of the forward pumps, when the flow from all four entered the main 20-in. line. All pipe connections on the scow were specially cast for the work. For the parallel connection a flanged Y joined the stern pump discharges to the 14-in. pipe above mentioned, which extended along the center of the scow to another Y bored to receive it, and connecting the forward pumps to the main line. Gate valves were set in between each discharge and the main line, so that by disconnecting the motor and closing the valve any one pump could be closed off without affecting the others.

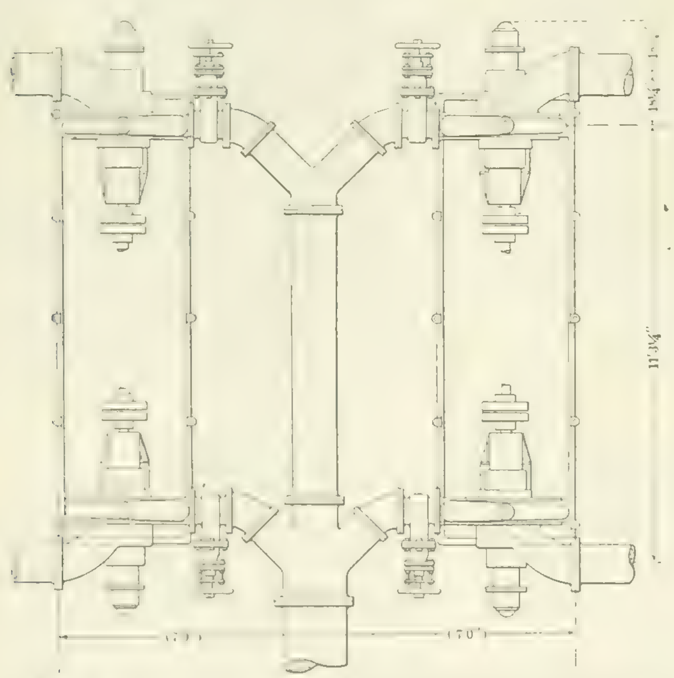


Fig. 1. Plan of Piping for Parallel Operation of Pumps.

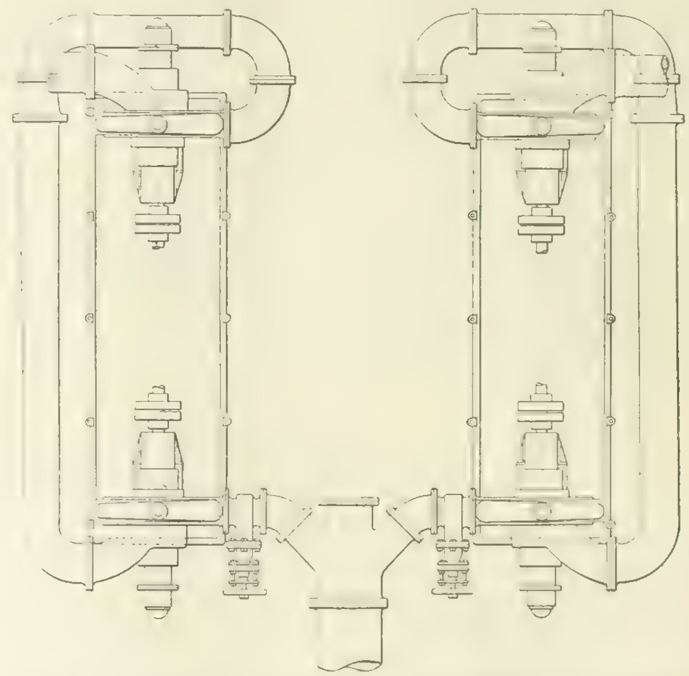


Fig. 2. Plan of Piping for Series Operation of Pumps.

A change from parallel to series operation (Fig. 2) could be made at whatever point in the undertaking the increase of head and density of material made it advisable. This was arranged as follows: The forward suctions, stern Y, and center length of 14-in. pipe were to be removed. Cast U-shaped lengths of pipe were provided to carry the discharge from the stern pumps outside and around to the intakes of the forward pumps, where a blank flange fitted to the 14-in. orifice in the forward Y threw the water forward to the main line. By this arrangement two units, each of a two-stage tandem pump, were obtained at some sacrifice of volume, but at a decided gain in efficiency.

For priming and keeping pressure on the impeller bearings, a 5¼x3½x5-in. air-driven plunger pump was installed on the scow, which took its power from the mine compressed-air supply, and its water from Kerr Lake at the start, but after its waters became too muddy, from the auxiliary supply from Giroux Lake. The main suction pipes were connected to the pump shell by 90° elbows, and extended to the water's edge where 8-ft. lengths of smooth-bore suction hose continued the intake. The suctions were fitted with flap foot valves and strainers having 3-in. apertures. The suctions were arranged on loose threads to act as a swivel joint, which, with the aid of a tackle, allowed the intakes to act at any desired depth within an 8-ft. radius.

The motors to drive the pumps are two in number, one for each unit, and connected to the pumps directly, as above described. They are 250-HP. squirrel-cage synchronous induction motors, operating at 1,200 rev. per minute, with 2,200-volt, three-phase, 60-cycle current.

The power was obtained from the line of the Northern Ontario Light & Power Co., which passes through the property. A 2,200-volt line was carried without transforming, to a switch house built for the purpose, on shore near the scow. Here, oil and knife switches, meters, etc., were installed, whence heavily insulated copper cables were led to the scow. The connections on the scow were made through oil-switch, panel-mounted knife switches, and starting compensators for each motor. Lightning arresters were placed outside the deck house. Cables inside the house were carried in pipe, and below deck where possible, to the various connections.

MAIN LINE OF SPIRAL RIVETED PIPE.

A 20-in., 14-gage, spiral riveted pipe was used for the main drainage line. Bolted steel joints were used, except for a few flanged lengths on angles, and between the scow and shore. This type of joint was used on account of its great flexibility and capacity for taking up expansion and contraction. These qualities were very desirable here, where extreme changes of temperature occur, and where portions of the line, both at the feed and discharge ends, had to be shifted from time to time. This pipe was supplied in 32-ft. lengths, weighing 1,500 lbs. to the length. The pipe was laid either on the ground or on simple bents, two to each length, keeping it in as straight a line, and as free from hollows as possible. What few angles there were, were calculated in the survey, and were met by specially cast flanged elbows, to which lengths of pipe flanged at one end were bolted. A little ditch work, and rock drilling and blasting were necessary, especially where the line crossed under the main highway by culvert, but in the main, inequalities of the ground were made up by the bents. The chief engineering difficulty met with was in carrying the pipe over the tracks of the Temiskaming & Northern Ontario Ry. at the minimum height allowed above rail, of 22 ft. 6 ins., an action made necessary to avoid causing a hollow in the line if carried under the tracks.

At this point two wooden towers of the ordinary tram type, built of 8x8-in. posts on a 16-ft. square base, well tied and cross braced, were erected on each side of the track. The angle of crossing made a span from one support to the other of 90 ft. Saddles for the pipe were provided 4 ft. below the top of the towers. Over the tops, which were iron-shod cross pieces, two ⅞-in. steel-wire cables were passed, 20 ins. apart, and carried to ground. These were anchored by "dead-men" made of stout logs sunk to a depth of 6 ft., and weighted with stone. Turnbuckles were provided, by which the cables were stretched as taut as possible.

The pipe was then laid on the bents and joined as far as the first tower. Three flanged lengths were then laid alongside on the bents and bolted together to make the span. One end of the joined lengths was slung in a carrier, made of an iron loop ending in sheaves to run on the two cables, and pushed out over the railway to the far tower. Connection was then made with the pipe already laid. The span of pipe across the railway was supported by 2x½-in. strap-iron hangers passing under the pipe and hooking over the cables at 8-ft. intervals. Each hanger had a turn screw and nut, to take out all sag in the pipe. An angle flange and two connecting lengths carried the line from the tower to a rock-filled crib pier where another angle flange was connected, and anchored by long eyebolts passing to a "dead-man" to guard against displacement of the pipe by the force of the water coming over the sudden drop from the tower.

The continuing line was carried on bents, as before, across a swamp and down the final incline to Giroux Lake.

Flexibility of connection between the floating scow and the fixed pipe on shore was secured as follows: The Kerr Lake end of the pipe, which reached shore at an angle of 15°, was anchored by strap and bolts set in a cement pier at water line. The end of the pipe facing the water was flanged, and to this was bolted a 20-in. flexible ball joint having a maximum swing of 27. To this joint in turn two expansion joints giving a lateral play of 16 ins. were bolted. At the start, one 32-ft. length of flanged pipe was connected to these, and to a similar ball joint coupled inboard on the scow at the main discharge. A 12-in. bypass, with a gate valve, was inserted on the flanged length, for draining the line.

DRAINAGE OPERATIONS.

The Kerr Lake pumps were started on Aug. 28, 1912. Some difficulties of a temporary nature were met with at the start, but there were, on the whole, few delays or hitches in the operation. It was found that too sudden stopping of the pumps caused vacuums to form which made one or two lengths of pipe show a tendency to collapse, but this was remedied by the insertion of check valves at threatened points. A tendency of the pipe to sag after receiving the full weight of water was observed on the span over the railway, and the structure was strengthened by the addition of two more ⅞-in. cables stretched and anchored in the same way as the original ones, and by the construction of arm props on each tower, hung out over the track by ¾-in. rods, and set in notches on the legs, which shortened the unsupported span of pipe by 24 ft.

The water was lowered steadily until the depth under the scow became too shallow for convenience, when a new flanged length of pipe was inserted, and the scow moved out a corresponding distance. The extra length was supported by trestle bents having an adjustable block and tackle sling to allow for the falling level of the pipe. The operation was repeated as often as it became necessary to make a move until the suspended line became too cumbersome, when the shore ball joint was moved out to a new pier and the connecting pipe shortened accordingly.

It had been intended to stir up the mud as much as possible by agitation while there was plenty of water in the lake, so that the mixture flowing through the line might be as liquid as possible. Various methods were tried, such as directing a stream, pumped from the lake by an auxiliary plunger pump of 300 gals. capacity, through a 4-in. hose with monitor attachment, into the mud, both from shore and from a small scow, but it was found impracticable, since the mud was of such consistency that although the bulk of that exposed by the lowering of the water flowed into the deeper part almost without sluicing, that under water had a gluelike tendency to stay in banked masses near the shores as long as any water remained in the lake to hold it back. Such mud as was left above the water soon dried and became compact to a short depth, so that it offered little difficulty to mining or prospecting, but that in the center of the lake constantly increased in depth with the influx from the sides, and as rapidly diminished the clearwater area. Tests made on this mud showed that its water content was very high, nearly 80 per cent, so that it was decided to pump out the remaining water entirely, and with the suctions resting in the mud, trust to the pumps handling it with the aid of the small streams of water flowing into the basin from the mines and mills. The change of the pumps from parallel to tandem operation, provided for as above described, was made in order to obtain better efficiency with the heavier material, and after some experimenting with the proper mixture of mud and water, and with the size of the strainer openings, a fairly steady stream of liquid mud varying from 6 to 20 per cent solids was maintained through the line.

Some trouble was had from the muddying of the water in Giroux Lake near the mine supply pumps. This was met by carrying the pipe line on a curve of 45°, to which the flexible nature of the bolted joints adapted it admirably, to a wooden flume, built along shore, and extending to a remote cove of the lake, after which no more trouble was experienced.

At the time of the first proposals for the draining it had been feared that the health of the people in the rather thickly settled vicinity of Kerr Lake might suffer, but fortunately these fears had no justification; in fact, if anything the reverse was the case, since the more than doubtful water supply of Kerr Lake was replaced by the purer water of Giroux. At one period when most of the clear water had been pumped away, some embarrassment was caused by the large number of fish which had been smothered by the mud and came to the surface. Great numbers ranging in size from small perch to eels and pike 30 ins. long lay everywhere, and even clogged the suctions and entered the valves. Fortunately, the situation was well taken care of by the gathering of great numbers of scavengering gulls who soon disposed of the fish, to the mutual advantage of themselves and the operators.

The pumps were run through September and October, and at intervals during the month of November, but during the latter month the increasing cold made operation difficult because of the freezing of valves and of the surface of the mud, so that work was stopped for the season the last of the month. The operations to this point were successful from the standpoint of the companies concerned. The water was removed from the greater part of the important reserves of the mines, thus leaving them free for stoping and development. About 325,000,000 gals. of mud and water were pumped, at an average of 6,000 gals. per minute for 38 actual working days.

It was demonstrated that the liquid mud lying in the deeper parts of the lake could be pumped, and that eventually the ground underlying these deeper parts could be prospected and mined at will after the removal of the remaining mud in the following open season.

Method of Sealing a Battery of Irrigation Wells.

Contributed by J. W. Swaren, Hayward, Calif.

While installing a horizontal centrifugal pump drawing its supply from a battery of wells it was found necessary to cut off the flow of water from the wells. The usual arrangement of setting the pump in a pit sunk nearly to water level had been planned. The wells, which were 16 ins. in diameter and cased with stovepipe casing, had been sunk during an extremely dry season. On completion, the water level was found to be 21 ft. below the surface of the ground. A pit, plan of which is shown in sketch Fig. 1, was dug, and lined with concrete, its floor being 14 ft. below the ground level. This allowed for a rise of water level in the wells of 7 ft., which was thought, based on past experience in the same vicinity, to be more than sufficient. No bulkhead was built between the well pit and the machinery pit.

An exceptionally heavy rainfall occurred the following winter, and fortunately before the installation of the machinery. At the close of the rainy season an examination of the pit showed that the water level had risen in the wells until the water stood to a depth of 41 ins. in the pit, or a rise of water table amounting to 10 ft. 5 ins. By the time the machinery was assembled at the pump site the water level had dropped to 37 ins. in the pit and still remained, several weeks later, at this point. As the driving motor is mounted on the same sub-base with the pump, not only was it necessary to unwater the pump pit, but some means must be provided to keep the water from entering the machinery pit during future periods of high water.

The specifications called for a suction pipe, 8 ins. in diameter and 30 ft. long, in each well, all connected to a common header, which in turn, by a special side opening cross, over the center well, is connected directly to the suction opening of the pump. The center line of this opening was but 22 ins. above the bottom of the sub-base. Thus the header would be below the water level, as it now stood, when installed.

It was decided to build a bulkhead, 5 ft. high, between the machinery pit and the well pit, with a short run of pipe passing through the bulkhead, between pump and well header. With this bulkhead in place, the machinery pit easily could be kept dry, while the water could rise and fall at will in the well pit. In order to build this bulkhead, both pits had to be freed from water and the flow shut off from the wells. Several methods of plugging the wells were tried, but when the unwatering pumps were started, the pressure of the water forced out the plugs. Finally the scheme shown in Fig.. 2 was evolved.

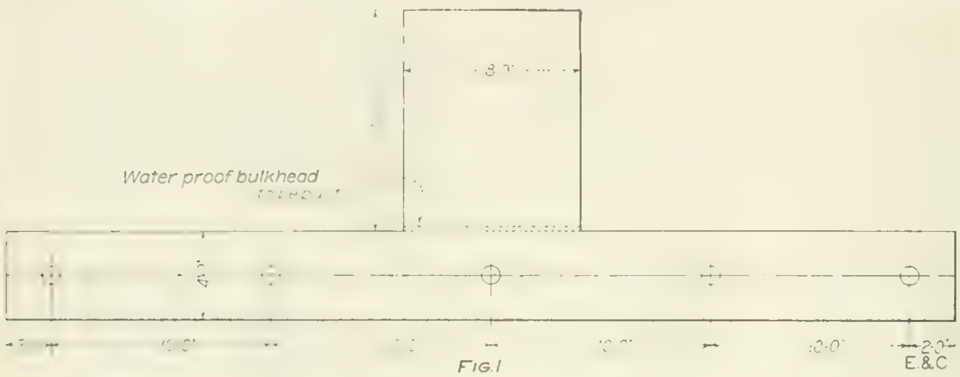
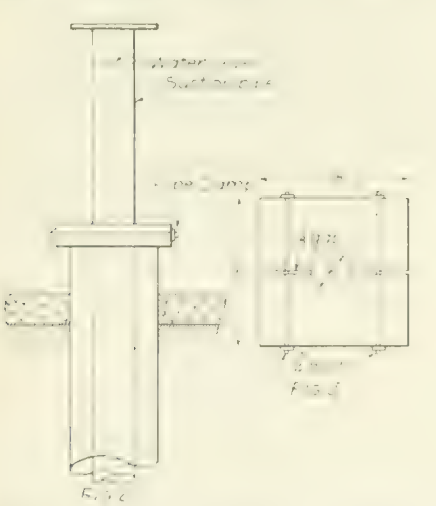


Fig. 1



Sketches Showing Method of Sealing a Battery of Wells.

The stovepipe casing of each well had been separated at a joint, leaving a projection of from 3 to 8 ins. above the floor of the pit, except at the center well, which casing top was nearly flush with the floor.

Pipe clamps, as shown in Fig. 3, were made from 4-in. soft wood. These clamps being 18 ins. over all, entirely covered the stovepipe casing when properly spaced. One of these clamps was clamped tightly on each run of suction pipe, spaced approximately 4 ft. below the header flange. The saddle of the clamp was cut accurately, making a water tight joint between wood and pipe when drawn up tight. The joint between the halves of the clamps was made ⅛-in. scant so that the clamps could be drawn very tight by the through bolts. Afterward, the opening at this joint was closed by wooden wedges, dipped in waterproof shingle compound, and driven from the lower side.

Each suction pipe was then lowered into its respective well, being supported by the clamp resting on the projecting portion of the stovepipe casing. The weight of the pipe forced the thin edge of the casing into the soft wood of the clamp, thus making an effectual seal. This method was successful with all the wells, except the center one, where, the casing being flush with the floor, the clamp rested on the concrete. When the unwatering pump was started, it was found that the leakage from the middle well was more than could be handled. The pipe of this well was drawn up and a gasket made of braided oakum and wrapped with burlap was fastened to the under side of the clamp. When again lowered into position, the center well was found to be satisfactorily sealed.

After the pit was unwatered, the bulkhead built, and the machinery installed, the header flanges on the suction pipe were brought in line, and the header assembled. Then by simultaneously slacking the through bolts on all the clamps, the header was gradually lowered to place. The only flange bolted up under water was that connecting the header to the suction opening of the pump.

Dam Foundations.

To the Editors:—I would like to say a few words concerning Mr. Edward Godfrey's discussion of the above subject on page 692 of Engineering and Contracting, June 17th.

I still agree with him that upward pressure under dams has caused or contributed to many dam failures, but contend that many successful dams lighter than he advocates also prove that upward pressure is not *always* dangerously operative. I still believe there is something in the old saying about "an ounce of prevention," etc. Therefore I believe that where it is practicable to prevent the occurrence of serious upward pressure under dams, it is better engineering to do that than to blindly design every dam heavy enough to hold down the upward pressure without first taking the trouble to find whether there need be any such serious pressure. It certainly is unsafe to *assume* in general that there will be *no* upward pressure, but it also seems to me poor engineering to assume that there *must always* be serious upward pressure that cannot possibly be prevented.

Some engineering *superstructures* can be safely and correctly designed entirely in the office, but I think there are few dams or engineering sub-structures of much magnitude that can be safely and economically designed without careful field investigation of the foundation.

The more facts Mr. Godfrey and the rest of us can learn and publish along these lines the better, but each dam or substructure is a special case and the same rule will not apply in all cases.

Mr. Godfrey seems to think that the heavy masonry dam with the broad base is a guarantee against damaging underflow. In harboring that theory I think he is *courting a danger* second only to that of neglecting to *consider* upward pressure under dams. A dam will not compress sandy or gravelly soil enough to materially reduce the volume of voids therein. See Engineering News, January 21, 1904, page 62, where it tells of the sand jack used in lowering into place the shaft of the Battle Monument at West Point. It says that the total pressure on the sand was 22½ tons per square foot, and the compression of the sand only 0.78 per cent. Twenty-two and a half tons per square foot corresponds to 300 feet height of concrete of constant horizontal section with no counteracting upward pressure of water. I cannot understand how any one could advocate building a dam of any kind on sand or gravel (which would practically always contain between 25 per cent and 40 per cent of voids) without a cut-off wall or its equivalent penetrating to some more impervious material.

There are many places in the west where rivers sink from sight, flowing below the surface, and rise again lower down. I understand a dam of considerable height was built in southern California that failed to impound any water because it had no cut-off wall, and the water continued on its unobstructed way through the dam, doing no damage to that structure. Certainly this dam was not lifted by upward pressure because the water never rose up behind it to develop a pressure head.

If cut-off walls may not be effective in preventing serious upward pressure as well as underflow, why did not Stony River Dam fail where it was highest and had the greatest head of water behind it? It did not fail there because the cut-off wall went to rock and was effective, but did fail where the cut-off wall stopped many feet above the rock, even though the height of dam was not maximum at that point.

Take the case of a heavy dam built on a generally good foundation but with occasional pockets or seams of pervious material. The weight of the dam will be carried by the good foundation and the dam will "bridge over" the looser material instead of compressing it, thus leaving a chance for underflow which may increase and undermine the dam, while careful examination of the foundation beforehand might have disclosed the danger and indicated the proper preventive remedy.

It is true as Mr. Godfrey says that "springs in soft soil have existed for generations, but they carry little sediment." On the other hand those "springs in soft soil" are not supplied by open water reservoirs in close proximity, giving a great pressure head, but are supplied by ground water which has in turn generally precolated long distances away from open bodies of water, if indeed it has any direct connection with such open bodies of water.

The instances I gave of boiling springs caused by high water against Ohio River levees were cases where considerable sediment was carried through by the water where the head was not over ten to eighteen feet and the levees were not lifted either.

Except where a cut-off wall or its equivalent near *down stream* toe of dam prevented the exit of water, I do not think it necessary or even reasonable to assume that the dams that failed were actually *lifted* preliminary to final destruction.

I think in most cases, except as above noted, that the dam will move down stream or settle due to underwash, before the upward pressure becomes great enough to actually lift it, and certainly the lifting is not always a necessary preliminary to the passage of water underneath a dam resting on porous soil or seamy rock.

H. P. Boardman,
Professor of Civil Engineering, University of Nevada.
Reno, Nev., July 23, 1914.

Joint Meeting of Highway Associations in 1915.—At a special meeting of the board of directors of the American Road Builders' Association held in New York July 17 it was unanimously voted to hold a joint meeting with the American Highway Association in 1915, either in San Francisco or in Oakland, during the Panama-Pacific Exposition. The management of this joint meeting will be in the hands of a committee of five made up of two members from each association, the fifth member to be selected by these four.

The city of Seattle by a vote of almost two to one defeated the new city charter which provided a commission-city manager form of government.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design and to Methods and Cost of Construction

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The Adverse Freight Rate Decision as an Incentive to Speedy Completion of Railway Appraisals.

The time required to appraise all the railways in America has been put at five to eight years, according to those acting for the Interstate Commerce Commission. Yet it is not impracticable for every railway to finish its own appraisal within less than two years, and it would now seem to be desirable in most cases to do so. The somewhat adverse freight rate decision has left the eastern railways with scant hope of an advance in rates until appraisals demonstrate the necessity of greater income, if there is to be a "fair return" on the value of their investments.

Several months ago we expressed doubt as to favorable ruling on railway rates in general until such time as appraisals make such rulings imperative. The granting of a scant 1.5 per cent increase in average rates where 5 per cent was asked bears out our prediction and emphasizes the necessity of rapid work on the appraisals.

Comparatively few people, even among investors in railway securities, know that there exists a published appraisal of the New York, New Haven & Hartford Ry. (See ENGINEERING AND CONTRACTING, Feb. 21, 1912, for the unit prices used in the appraisal by Prof. Swain.) Still fewer have taken pains to analyze the results, else there would be less pessimism as to the ultimate earning power of that property. Certainly if rates are to be based on cost of reproduction, the "New Haven" system is entitled to much higher rates than it now enjoys. We doubt not that the same holds true of most railway properties.

An appraisal of the Lehigh Valley Ry. has been completed, and it has been rumored that appraisals of several other large railway systems are practically completed. Even those railways that have thus far done little toward an appraisal could have a complete valuation prepared by the end of 1915, were the necessity apparent. To us there seems to be necessity for such action on the part of every railway. And as fast as each railway completes its appraisal it should ask for increments in rates—both interstate and intrastate—if existing rates do not yield a "fair return."

Classification of the Steel Production in 1913.

Data collected by the Bureau of Statistics of the American Iron and Steel Institute give some interesting information concerning the production of steel ingots and castings. The total production of all kinds of steel ingots and castings in 1913 was 31,300,874 tons, as against 31,251,303 tons in 1912. Of the 1913 production of steel, 30,280,130 tons were ingots and 1,020,744 tons were castings, while in 1912, 30,284,682 tons of ingots and 966,621 tons of castings were produced. The 1913 production of steel consisted of 9,545,706 tons of Bessemer steel, 21,599,931 tons of open-hearth steel and 155,237 tons of crucible and other kinds of steel. This output was produced by 304 works in 30 states, the District of Columbia and the Panama Canal Zone. The increasing use of alloy steels is shown by the fact that in 1913 about 714,357 tons of ingots and castings were treated with ferro-titanium, ferro-vanadium, ferro-chrome, nickel, or some other alloy, of which 625,430 tons were ingots and 88,927 tons were castings. Of the total production of alloy steels in 1913, 74,924 tons were Bessemer steel ingots or castings, 599,890 tons were open-hearth steel, 28,279 tons were crucible, 11,254 tons were electric, and 10 tons were produced by miscellaneous processes.

The total production of Bessemer steel in 1913—9,545,706 tons—consisted of 9,465,200 tons of ingots and 80,506 tons of castings. Of this tonnage, 9,465,882 tons were made by the standard Bessemer process, 42,408 tons by the Tropenas process, and 37,416 tons by other modifications of the standard Bessemer process. In connection with this output it is interesting to note that the production of Bessemer steel was a maximum in 1906, when 12,275,830 tons were produced.

The 1913 output of open-hearth steel—21,599,931 tons—consisted of 20,689,715 tons of ingots and 910,216 tons of castings. In 1908 the production of open-hearth steel for the first time exceeded that of Bessemer steel, the excess amounting to 1,719,974 tons, or about 28 per cent. In 1913 this excess amounted to 12,054,225 tons, or about 126 per cent, which shows the increasing popularity of open-hearth steel.

Of the total production of crucible steel in 1913—121,226 tons—about 28,279 tons consisted of various alloy steels, of which 25,281 tons were ingots and 2,998 tons were castings. The maximum production of crucible steel was reached in 1907, when 131,234 tons were produced.

Included in the 1913 output of electric steel —30,180 tons—which consisted of 20,973 tons of ingots and 9,207 tons of castings, were about 11,254 tons of alloy steels, consisting of 10,821 tons of ingots and 433 tons of castings. The production of steel by the electric process reached its maximum value—52,141 tons—in 1910. In 1912 the total production was only 18,309 tons. On Dec. 31, 1913, there were 19 plants equipped to manufacture steel by the electric process, as compared with 14 at the close of 1912. In addition to those in operation in 1913, 3 plants were being built and 7 were projected.

The production of open-hearth ingots in 1913 by the basic process was 19,884,465 tons, while the production by the acid process was only 805,250 tons. Of the 1913 output of open-hearth steel castings, 460,161 tons were made by the basic process and 450,055 tons by the acid process.

Included in the 20,344,626 tons of basic open-hearth steel ingots and castings produced in 1913 are 2,210,718 tons, which were produced from metal partly purified in Bessemer converters and finally reduced in basic open-hearth steel furnaces. The increase in the amount of steel produced by this duplex process in 1913 over that produced in 1912 was about 54 per cent. In 1913 duplex steel ingots and castings were produced by nine works, as against seven works in 1912.

At the close of 1913 there were 183 completed open-hearth steel plants; while 110 plants were equipped to make steel by the Bessemer process. At that time seven open-hearth steel plants were being built and five were projected; while only one Bessemer plant was being built, although eight modified Bessemer plants were projected.

The Use and Abuse of Road Standards.

Standards for road and bridge details are issued by most of the highway commissions. These standards comprise general and detailed plans of various kinds, illustrating typical structures frequently encountered in road, culvert and bridge design and are in effect a recommended method of design and construction for these structures.

It is perhaps somewhat unfortunate that the word "standards" should have been chosen to designate these plans. Strictly interpreted, the meaning would indicate that the standard design was the best design. This is by no means the case—nor is it intended to mean this. Standards are merely recommended designs which are to be adhered to unless conditions indicate that a variation in the design would meet them better. They are used to secure uniformity in laying out and building structures with the idea that, thereby simplicity of construction is furthered, the work of designing reduced and greater efficiency and better workmanship secured in construction.

And yet these standards are accepted as criterions by many engineers. Doubtless they are, in most cases, excellent designs if conditions under which it is proposed to use them are similar to those for which they were first planned. The first step in the preparation of any design is necessarily the imposing of certain limitations such as cost, capacity, foundation conditions, materials and construction plant requirements. These conditions must be met whether the design is a "standard" or otherwise. For each different combination of conditions there is perhaps one design which fulfills the requirements of that combination. The question arises as to the frequency with which certain combinations of conditions occur. If frequently encountered, then a standard design leads to economy and one should be prepared and used.

If this point is kept in mind there are many advantages to be derived from a thorough knowledge of the standards in use in various sections of the country. As a rule they are designs prepared by engineers of wide experience and of prominence in their profession and they represent the chrystallization of ideas tempered by mature judgment and years of observation. A young engineer and an old engineer, inexperienced in the class of work to which they refer, may compare and study them with profit. There is no quicker or better method of becoming familiar with what is commonly called "good practice." But in this study the engineer must acquaint himself thoroughly with the conditions for which the standards were prepared.

For example, selecting the article on Iowa road standards, recently published in this journal, a number of questions must be answered in order to secure the greatest benefit from a study of the article. What are the soil and climatic conditions existing in that state, how are available materials distributed, what are the labor conditions, of what type and how heavy is the road traffic, what is the average value of land, in what occupations are the people principally engaged, are points which should be considered. With a knowledge of conditions, a study of designs prepared to fulfill them is very instructive.

There is, however, a grave danger attendant on the use of standards of any kind. The temptation is to neglect the detailed study of local conditions and use a standard structure. This often results not only in an unwarranted increase in the cost of a suitable structure, but may result in a type of construction which fits but poorly the location where used.

The foregoing are perhaps the chief indictments against the use of standards. They are old indictments, as are those against the use of formulæ and cost data. And yet in spite of criticism standards still remain the tools of the engineer, with a definite work-a-day value which is appreciated by the practitioner.

BRIDGES

The Designing of Reinforced Concrete Retaining Walls by Comparison With the Determined Ratios of the Various Functions of the Height and Unit Pressures.

Contributed by S. M. Cotter, State Engineer Office, Phoenix, Ariz.

Having recently had occasion to design a number of reinforced concrete retaining and wing walls of the cantilever type, for various heights and unit pressures, it occurred to the writer that ratios might be established between the height of the wall and the several functions of the height and between the unit pressures and these functions, which would greatly lessen the labor incident to a considerable number of designs. To be more specific, it was desired to establish ratios so that, having designed one wall for any given height and unit pressure, it would be possible to design a wall of the same type for any height or unit pressure whatever, by using these ratios in conjunction with the properties determined for the first wall.

In the case where the unit pressure is a constant and the height is a variable, it was, of course, desired to express all possible properties of the wall as a function of the height. Regarding the height as a constant and the unit pressure as the variable, it was likewise desired to express these properties as a function of the pressure. It was found that all the required properties, except the quantity of concrete and the weight of steel for a given section, could be so expressed, and that the time and labor which could be saved thereby in subsequent designs more than exceeded the writer's best hopes.

The wall under discussion is constructed of reinforced concrete, cantilever type, and is designed to withstand an equivalent fluid pressure in accordance with the discussion and determinations given in Turneaure & Maurer's "Principles of Reinforced Concrete," which will not be repeated here, except such parts as are considered necessary to the sequence of this discussion. The type of wall and the forces acting upon it, except the upward force on the toe whose effect in this design is too negligible to warrant attention, are shown in Fig. 1.

CONSTANT UNIT PRESSURE AND VARIABLE HEIGHT.

Discussion.—Letting the unit pressure, p, remain constant, with the height as the variable, the required ratios and their derivation are as follows:

Referring to Fig. 1 and to the formulas developed in Turneaure & Maurer—

$P = \frac{1}{2} ph^2$. But p is constant; hence $P \propto h^2$.

$l = 0.087 \sqrt{p}\, h$. But p is constant; hence $l \propto h$.

Likewise, since [illegible], $m \propto l$; and since $l \propto h$, $m \propto h$.

For a similar reason, $Q \propto h$.

[illegible] unit weight of earth. But [illegible]

Pressure on toe per square foot, $E =$ [illegible] where [illegible] weight of concrete, and [illegible] weight of earth. But [illegible] approximately as h^2, and is small compared to [illegible] and [illegible] hence [illegible] (The theory of this approximation calls for too long a discussion to be entered into here, but it can be shown by cases of independent design that the result obtained by its use is sufficiently exact for the purpose.)

$D = \frac{2}{3} E \frac{m}{2} = E \frac{m}{3}$. But [illegible] h and [illegible] h; hence $D \propto h^2$.

$M_p = P \times$ arm. But $P \propto h^2$ and arm $\propto h$, hence $M_p \propto h^3$.

$M_c = C \times$ arm. But $C \propto h^2$, arm $\propto m$, $m \propto h$, hence $M_c \propto h^3$.

$M_d = D \times$ arm. But $D \propto h^2$, arm $\propto m$, $m \propto h$, hence $M_d \propto h^3$.

Since M_c and $M_d \propto h^3$, $(M_c - M_d) \propto h^3$.

From Turneaure & Maurer, $R = \frac{M_{max}}{bd^2}$.

But since R is constant in any one design and b is constant, $d^2 \propto M_{max}$. But $M_{max} \propto h^3$; hence $d^2 \propto h^3$.

Also from Turneaure & Maurer, area of steel, $A = bdp$. But since b and p are constants, $A \propto d$.

Also, the spacing of rods varies inversely as their area.

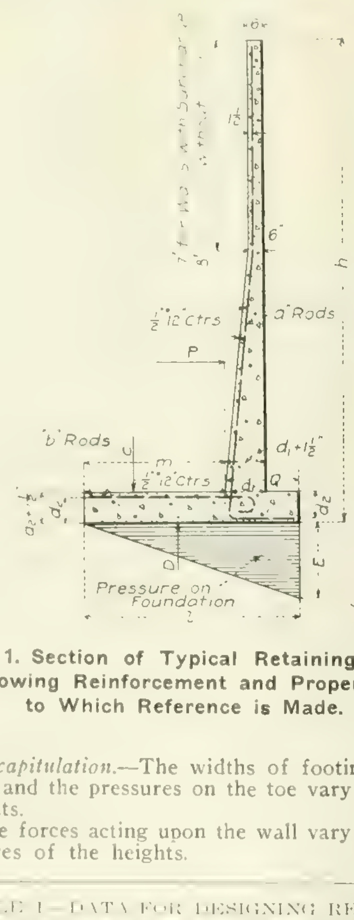


Fig. 1. Section of Typical Retaining Wall Showing Reinforcement and Properties to Which Reference is Made.

Recapitulation.—The widths of footing and heel, and the pressures on the toe vary as the heights.

The forces acting upon the wall vary as the squares of the heights.

The moments of these forces vary as the cubes of the heights.

The square of the depth of the steel varies as the cube of the height; the areas of steel vary as the depths; and the spacing of rods varies inversely as their area.

Also, when $x \propto y$, $x = ky$ (where k is a constant), which readily gives values of x for different values of y.

Having designed a wall for a given height, with these ratios known, one can easily and rapidly design walls of the same type for any heights whatever, directly from the properties of the one design. The method is particularly applicable to the computation of standard tables for retaining wall whose height is not uniform, and for wing walls whose thickness and footing are designed to decrease as their height decreases.

CONSTANT HEIGHT AND VARIABLE UNIT PRESSURE.

Discussion.—Letting the height remain constant, with the unit pressure, p, as the variable, the required ratios and their derivation are as follows:

$P = \frac{1}{2} ph^2$. But h is constant; hence $P \propto p$.

$l = 0.087 \sqrt{p}\, h$. But h is constant; hence $l \propto \sqrt{p}$.

Likewise, since $m = \frac{2}{3} l$, $m \propto l$; and since $l \propto \sqrt{p}$, $m \propto \sqrt{p}$.

For a similar reason, $Q \propto \sqrt{p}$.

$C = w_e hm$. But w_e and h are constant, and $m \propto \sqrt{p}$; hence $C \propto \sqrt{p}$.

Pressure on toe per square foot, $E = 2\frac{W_1 + W_2}{l}$. But $W_1 \propto \sqrt{p}$ (approximately), and $W_2 = C$ (nearly), and $\propto \sqrt{p}$; also $l \propto \sqrt{p}$; hence $E \propto \frac{\sqrt{p}}{\sqrt{p}}$ (= unity).

$D = \frac{2}{3} E \frac{m}{2} = E \frac{m}{3}$. But E is constant and $m \propto \sqrt{p}$; hence $D \propto \sqrt{p}$.

$M_p = P \times$ arm. But $P \propto p$ and arm is constant; hence $M_p \propto p$.

$M_c = C \times$ arm. But $C \propto \sqrt{p}$, arm $\propto m$, and $m \propto \sqrt{p}$; hence $M_c \propto p$.

$M_d = D \times$ arm. But $D \propto \sqrt{p}$, arm $\propto m$, and $m \propto \sqrt{p}$; hence $M_d \propto p$.

Since both M_c and $M_d \propto p$, $(M_c - M_d) \propto p$.

TABLE I.—DATA FOR DESIGNING RETAINING WALLS OF TYPE SHOWN IN FIG. 1 FOR A FLUID PRESSURE OF 20 LBS. PER CUBIC FOOT AND FOR VARIOUS HEIGHTS.

h, ft.	l, ft. ins.	m, ft. ins.	E, lbs. per sq. ft.	d_1, inches	d_2, inches	Rods "a," size and spacing.	Rods "b," size and spacing.	Cu. yds. of concrete per lin. ft.	Lbs. of steel per lin. ft.
5	1 11	1 3½	980	4½	2½	½ in. sq. at 12 ins.	½ in. sq. at 18 ins.	0.11	9.71
[illegible]	[illegible]	1 [illegible]	1,180	4½	2½	½ in. sq. at 12 ins.	½ in. sq. at 18 ins.	0.13	12.85
[illegible]	[illegible]	[illegible]	1,370	4½	2½	½ in. sq. at 12 ins.	½ in. sq. at 14 ins.	0.15	16.46
[illegible]	[illegible]	[illegible]	1,570	4½	3½	½ in. sq. at 8¾ ins.	½ in. sq. at 11½ ins.	0.18	21.73
[illegible]	[illegible]	2 4	1,760	5	4	½ in. sq. at 7¼ ins.	½ in. sq. at 9½ ins.	0.20	27.04
10	3 10	[illegible]	1,960	6	4½	½ in. sq. at 6¼ ins.	½ in. sq. at 8¼ ins.	0.24	32.56
[illegible]	[illegible]	[illegible] 10	2,160	7	5½	¾ in. sq. at 12 ins.	½ in. sq. at 7 ins.	0.28	40.07
[illegible]	[illegible]	[illegible]	2,360	8	6	¾ in. sq. at 10¾ ins.	½ in. sq. at 6 ins.	0.32	47.76
[illegible]	[illegible]	3 1	[illegible]	9	7	¾ in. sq. at 9½ ins.	¾ in. sq. at 12¼ ins.	0.37	56.97
[illegible]	[illegible]	3 [illegible]	2,750	10	7½	¾ in. sq. at 8½ ins.	¾ in. sq. at 11 ins.	0.43	66.10
[illegible]	5 10	3 10	2,950	11	8½	¾ in. sq. at 7¼ ins.	¾ in. sq. at 10 ins.	0.49	76.40
16	[illegible]	4 2	3,140	12	9	¾ in. sq. at 7 ins.	¾ in. sq. at 9 ins.	0.55	88.58
[illegible]	[illegible]	4 5	3,340	13	10	¾ in. sq. at 6½ ins.	¾ in. sq. at 8¼ ins.	0.63	100.76
18	[illegible]	4 8	3,540	14	11	¾ in. sq. at 6 ins.	¾ in. sq. at 7¼ ins.	0.71	112.78

As previously stated, $d^2 \propto M$. But $M \propto p$; hence $d^2 \propto p$, or $d \propto \sqrt{p}$.

Also, $A \propto d$. But since $d \propto \sqrt{p}$, $A \propto \sqrt{p}$.

Also, the spacing of rods, S, varies inversely as A. But since $A \propto \sqrt{p}$, $S \propto \frac{1}{\sqrt{p}}$.

Recapitulation.—The widths of footing and heel vary as the square roots of the unit pressures.

The horizontal forces vary as the unit pressures.

The vertical forces vary as the square roots of the unit pressures.

The depths and the areas of steel vary as the square roots of the unit pressures; and the spacing of rods varies inversely as the square roots of the unit pressures.

Also, when $x \propto y$, $x_1 : x_2 :: y_1 : y_2$, and

$$x_2 = x_1 \frac{y_2}{y_1} = x_1 k, \text{ (where } k \text{ is a constant).}$$

Having designed a wall of a given height (or any number of walls of given heights) and of constant unit pressure, these ratios enable one to modify the design to correspond to any unit pressure whatever.

TABULAR DATA.

Another feature of both these systems of ratios, which is worthy of comment, is with reference to an approximate design for estimating purposes. Curves may be constructed, passing through only a few determined points, which will serve a considerable range of heights or of pressures, from which the values for intermediate points may be found with sufficient exactness.

TABLE II.—DATA FOR DESIGNING RETAINING WALLS OF TYPE SHOWN IN FIG. 1 FOR A FLUID PRESSURE OF 26.6 LBS. PER CUBIC FOOT AND FOR VARIOUS HEIGHTS.

h, ft.	l, ft. ins.	m, ft. ins.	E, lbs. per sq. ft.	d_1, inches	d_2, inches	Rods "a," size and spacing.	Rods "b," size and spacing.	Cu. yds. of concrete per lin. ft.	Lbs. of steel per lin. ft.
5	2 3	1 6	980	4½	2½	½ in. sq. at 15 ins.	½ in. sq. at 18 ins.	0.11	10.60
6	2 8½	1 9½	1,180	4½	2½	½ in. sq. at 11½ ins.	½ in. sq. at 15 ins.	0.14	14.17
7	3 2	2 1	1,370	4½	3	½ in. sq. at 9¼ ins.	½ in. sq. at 12 ins.	0.17	18.13
8	3 8½	2 5½	1,570	5	3½	½ in. sq. at 7½ ins.	½ in. sq. at 9¾ ins.	0.19	24.38
9	4 0½	2 8½	1,760	6	4½	½ in. sq. at 6¼ ins.	½ in. sq. at 8¼ ins.	0.23	30.23
10	4 6	3 0	1,960	7	5	½ in. sq. at 5½ ins.	½ in. sq. at 7 ins.	0.26	36.35
11	4 11½	3 3½	2,160	8	6	¾ in. sq. at 10½ ins.	½ in. sq. at 6 ins.	0.32	45.76
12	5 4½	3 7	2,360	9	7	¾ in. sq. at 9¼ ins.	½ in. sq. at 5¼ ins.	0.37	54.55
13	5 10	3 11	2,560	10	8	¾ in. sq. at 8¼ ins.	¾ in. sq. at 10½ ins.	0.43	65.10
14	6 3½	4 2½	2,750	11	9	¾ in. sq. at 7¼ ins.	¾ in. sq. at 9½ ins.	0.50	76.93
15	6 9	4 6	2,950	12½	9½	¾ in. sq. at 6½ ins.	¾ in. sq. at 8½ ins.	0.57	89.57
16	7 2	4 9	3,140	14	10½	¾ in. sq. at 6 ins.	¾ in. sq. at 7¾ ins.	0.65	101.90
17	7 7½	5 1	3,340	15	11½	¾ in. sq. at 5½ ins.	¾ in. sq. at 7 ins.	0.74	117.30
18	8 1	5 5	3,540	16½	12½	¾ in. sq. at 5¼ ins.	¾ in. sq. at 6½ ins.	0.82	129.46

computed, although retaining the ratio of 4/3 for the surcharged pressure. The other values used in the designs were: weight of earth, 100 lbs. per cubic foot; $f_s = 16{,}000$ lbs. per square inch; $f_c = 600$ lbs. per square inch; $n = 15$; and percentage of steel, 0.68.

The minimum thickness of wall was taken

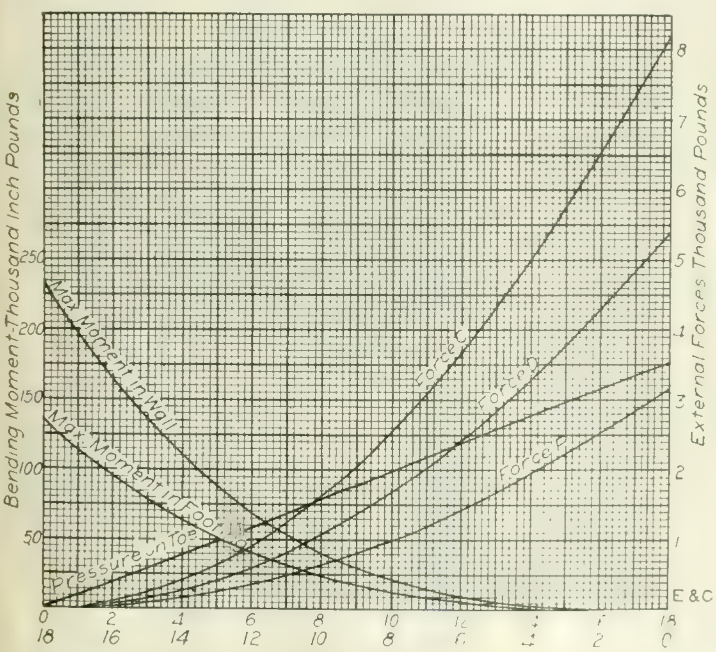


Fig. 2. Curves Showing Variations with Height, of Bending Moments and External Forces for Equivalent Fluid Pressure of 20 Lbs. per Cubic Foot.

ments in the wall vary as the cubes of the distances from the top, the quantity of steel in the wall above the point of maximum moment is in excess of the quantity required by an increasing amount. Investigations were made to determine the economy to be effected, if any, by decreasing the area of steel to conform with the requirement of some higher section and of lengthening the arm at the

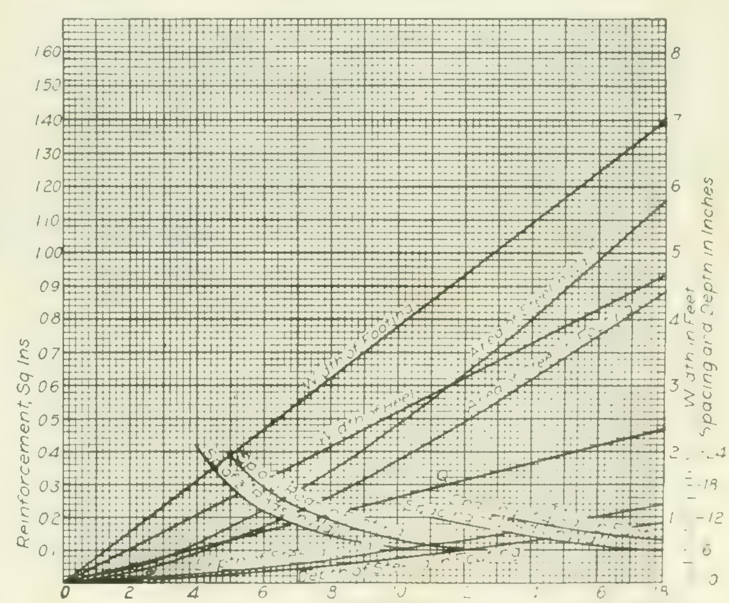


Fig. 3. Curves Showing Variations with Height, of Various Functions for Equivalent Fluid Pressure of 20 Lbs. per Cubic Foot.

Tables I and II give the essential data for walls designed to withstand unit fluid pressures of 20 and 26.6 lbs. per cubic foot, respectively, varying in height from 5 ft. to 18 ft. These unit pressures were selected as representing the pressures on a wall without and with surcharge (earth, slope 1½:1), respectively, although they probably err slightly on the side of safety. The wedge theory of earth pressure gives 16 and 21 lbs. per cubic foot equivalent fluid pressure, respectively, the resultant passing through the edge of the middle third. The Illinois Highway Commission uses 15 and 20 lbs. per cubic foot fluid pressure in its designs, and states that the results have been entirely satisfactory. It was thought best, however, to increase the minimum pressure to 20 lbs. per cubic foot, in the designs for which these tables were

as 6 ins., and the minimum thickness of footing as 4 ins., although these values are in excess of theoretical requirements. For surcharged walls the upper 7 ft. have a constant thickness of 6 ins., and for walls without surcharge the upper 8 ft. have this minimum thickness of 6 ins. At the top of the footing the thickness of the wall is $d_1 + 1½$ ins. Between these points the back wall slopes uniformly (see Fig. 1). The front wall is vertical.

Should the pressure at the toe, for any height, be too great for conditions of construction, simply add to the length of toe as may be necessary, without changing the other dimensions of the wall.

Since d_1 and the area of steel for any height of wall were designed to resist the maximum moment, and since the bending mo-

point of maximum moment to correspond with this decrease. These investigations showed conclusively that such modification would not be economical.

With reference to the weights of steel given in the Tables I and II, the length of the vertical rods was taken as $h + Q - 6$ ins.; the length of the horizontal rods in the footing as $m + 6$ ins.; while the longitudinal rods were in all cases ½-in. square bars, spaced 12 ins. on centers.

DIAGRAMS.

The curves of Fig. 2 show the variations of the external forces and the maximum bending moments in the walls and footings, for various heights of wall, for an equivalent fluid pressure of 20 lbs. per cubic foot. The curves of Fig. 3 show the variations of width of footing, area of steel in wall and footing, spacing of rods in wall and footing, and depths of steel in wall and footing, for an equivalent fluid pressure of 20 lbs. per cubic foot.

General Design Features of the Metropolis Bridge Spanning the Ohio River at Metropolis, Ill.

(Staff Article.)

The Chicago, Burlington & Quincy R. R. and the Nashville, Chattanooga & St. Louis R. R. have commenced the construction of an important bridge over the Ohio River at Metropolis, Ill., the main feature of which is a 720-ft. simple-truss span. The structure consists of six main through truss spans, one deck truss span and steel girder approaches on steel bents. The bridge is a double-track structure, with tracks spaced 13 ft. center to center. Figure 1 shows an elevation of the bridge and gives some essential data concerning its design features.

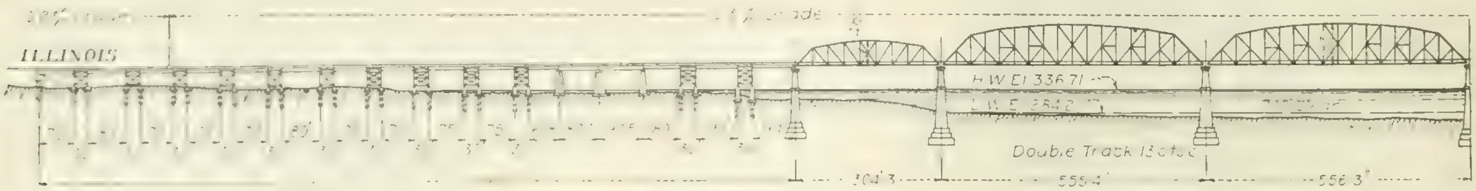


Fig. 1. General Elevation of the Metropolis Bridge Across the Ohio River

The 720-ft. simple-truss span will be the longest simple span ever constructed, exceeding in length that of the St. Louis Municipal Bridge by 52 ft. Beginning at the Illinois end of the structure the lengths of the truss spans, center to center of bearings, are: 300 ft. 0 in.; four spans at 551 ft. 3 ins.; 720 ft. 0 in.; and 250 ft. 0 in. The latter span is of the deck Pratt-truss type, the remaining spans having through Petit trusses. The distance center to center of trusses for the through spans are: 720-ft. span, 37 ft. 0 in.; 551-ft. 3-in. spans, 33 ft. 0 in.; and 300-ft. span, 32 ft. 0 in. For the 720-ft. span the maximum height of truss, center to center of chords, is 110 ft. 0 in.; for the 551-ft. 3-in. spans, 83 ft. 0 in.; and for the 300-ft. span, 48 ft. 0 in. The approximate height of the river piers is 170 ft.

The approach on the Illinois side is a 1,590-ft. steel viaduct, consisting of 70-ft. to 90.5-ft. intermediate girder spans and 30-ft. tower spans, except where the bridge crosses some railway tracks, at which points concrete abutments replace the steel towers. The approach on the Kentucky side has a length of 600 ft., and consists of 30-ft. tower spans and 75-ft. intermediate girder spans. Both approaches have ballasted floors. By referring to Fig. 1, it will be noted that the track over the 720-ft. span is level, while the remaining grade is 0.3 per cent except for a short distance at the Illinois end of the bridge where the grade is 0.2 per cent.

The pedestals of the viaduct approaches are carried on pile foundations, while the foundations for the seven river piers are sunk, by means of the pneumatic process, through a foundation consisting of firm sand and gravel. The maximum depth to which the foundations for the piers are carried is about 78 ft. below low-water level.

The bridge will form a part of a new 15-mile line between Metropolis, Ill., and Paducah, Ky. The estimated cost of the structure is about $3,500,000, and it will be completed in 1916.

The bridge is being constructed by the Paducah & Illinois R. R. Co., a subsidiary of the Nashville, Chattanooga & St. Louis R. R. and the Chicago, Burlington & Quincy R. R., of which Mr. C. H. Cartlidge, bridge engineer of the latter company is chief engineer and Mr. Ralph Modjeski, consulting engineer. The contract for the seven river piers was let to the Union Bridge & Construction Co., of Kansas City, Mo., in June, 1914. It is expected that the contract for the superstructure will be awarded in about a week, bids having been received July 20, 1914.

Construction Features of the Duplicate Reinforced Concrete Arch Bridges at Third Ave., Cedar Rapids, Iowa.

The two reinforced concrete arch bridges described in this article continue Third Ave. across the Cedar River and connect the east and west sides of Cedar Rapids, Iowa. The river at this point is divided by an island into two channels of approximately equal widths. The new bridges consist of two practically identical structures, one over each branch of the river. They replace an old steel bridge, consisting of six 117-ft. 7-in. bowstring trusses on masonry piers and abutments, three over each channel. Although the old bridge was erected in 1872, it was in sufficiently good condition to justify its re-erection at Eighth Ave., five of the six spans being used at that location. The article is based on data contained in a paper by B. J. Sweatt, in the Journal of the Western Society of Engineers.

GENERAL DESIGN FEATURES.

The following dimensions apply to the reinforced concrete arch bridge over each branch of the river:

Total length of three spans, face to face of abutments, 508 ft. 0 in.
Clear span length at spring line, 96 ft. 0 in.
Rise of arch above spring line, 11 ft. 0 in.
Height of spring line above low water, 7 ft. 1 in.
Width of solid arch ring, 64 ft. 0 in.
Width of roadway between curbs, 48 ft. 0 in.
Clear width of each sidewalk, 10 ft. 0 in.
Total width of each sidewalk, 11 ft. 9 ins.
Extreme width of arches, 71 ft. 6 ins.
Extreme width over piers, 81 ft. 0 in.
Thickness of arch ring at crown, 1 ft. 11 ins.
Thickness of arch ring at haunches, 5 ft. 0 in.

The sidewalks are carried a distance of 3 ft. 11 ins. beyond the arch rib by cantilever brackets, the portion of the sidewalks directly above the arch ring being supported on spandrel walls to provide space for gas and water mains and telephone and electric light conduits. The bridge carries concrete lamp and trolley poles.

All foundations, except the wing walls for the two main shore abutments, were carried to bed rock. The wing walls rest on pile foundations, the piles being driven to solid rock. The concrete piers are solid, but the abutments are of the compartment type below a plane 10 ft. under the spring line of the arches. The front walls are about 7 ft. thick, the side walls are 4 ft. thick, and the intermediate walls have a thickness of 3 ft. 3 ins. The intermediate walls are spaced 6 ft. 7½ ins. on centers. The specifications called for the filling of these compartments with gravel, but a very hard clay was encountered in the excavations, which permitted trenches to be excavated for the walls, and the material occupying the spaces between the walls was left in its natural state.

The concrete proportions used for the various parts of these bridges are as follows:

(a) Piers below a plane 5 ft. under the spring line, and abutments below a plane 5 ft. 6 ins. under the spring line and 7 ft. back from the face at the bottom are a 1:3:6 mix, crusher-run stone having maximum dimensions of 3 ins being used.

(b) Arch rings and piers, and abutments above the levels indicated above are a 1:2:4 mix, crusher-run stone having maximum dimensions of 1½ ins., with the dust screened out, being used.

(c) Spandrel walls and wing walls of abutments are a 1:3:5 mix, crusher-run stone having maximum dimensions of 1 in. being used.

(d) Sidewalks, hand-rails, lamp posts and sidewalk brackets are composed of 1 part cement and 3 parts sand and gravel.

The main reinforcing bars for the arches were 1¼-in. plain round bars, spaced an average distance of 12 ins. apart, but corrugated bars were used for the remainder of the concrete work. The plain bars were received at the site in two sections, one of which was 33 ft. and the other 83 ft. in length, but all bending was done at the site. The two sections were fastened together with turnbuckles, to permit the adjustment of the bars. However, it was found that turnbuckles served no useful purpose except to connect the two sections of the arch bars, as no adjustments were made.

METHODS AND EQUIPMENT.

The location of the structure was ideal for construction purposes. The Iowa Railway & Light Co.'s line crossed the island at Fourth Ave. and permitted the construction of a material track from this line to the construction track, which was built approximately on the center line of the new bridge. This construction track was used for removing the old bridge, for handling a large part of the excavation, for handling falsework material, and for driving piles, etc.

The following equipment was used in the construction of the two bridges:

A 15-ton McMyler-Interstate four-wheel type locomotive crane.
A 20-HP. American hoisting engine with boom swinger.
A 16-HP. American hoisting engine.
A 22-HP. Avery undermounted traction engine.
A ¾-cu. yd. Ransome mixer.
A ¼-cu. yd. Smith mixer.
A 40-HP. Westinghouse electric motor.
A 20-HP. Allis-Chalmers electric motor.
An 8-in. Fairbank-Morse vertical centrifugal pump.
A 6-in. vertical centrifugal pump.
A small single-acting steam pump.
Two stiff-leg derricks.
A barge derrick.
A 1-cu. yd. Myler clam-shell bucket.
Three drop pile driver hammers, one weighing 2,800 lbs., one 2,000 lbs., and one 1,000 lbs. (for driving sheet piling).

Most of the concrete was mixed at a central plant and distributed through spouts suspended from a 1-in. wire cable. The mixing plant, which was located on the island, consisted of:

A 60-cu. yd. bin for crushed stone.
A 40-cu. yd. bin for sand.
A 135-ft. tower for chuting the concrete.
A 54-cu. ft. receiving hopper.
A 30-cu. ft. hoisting bucket.

The ¾-cu. yd. mixer, above referred to, was used in this plant, the plant being operated by the 22-HP. traction engine, which was fitted with a hoisting drum for elevating the bucket.

The mixing room was placed just back of the excavation for the west island abutment and at the end of the cement shed. The stone and sand bins were directly over the mixing room, and these materials were delivered by gravity to the measuring hopper through undercut bin gates. The cement was also conveyed by gravity from the cement shed to the hopper, through a metal spout. Practically the only trouble experienced with the mixing plant was due to the fact that the metal spout clogged at times. The sand and a considerable part of the crushed stone were obtained locally, being delivered in wagons and dumped directly into the bins through trap doors in the platform above the bins. About 6,000 cu. yds. of crushed stone were used, about half of which was unloaded from the cars into the wagons by shovelers (the cars being switched to the island for unloading), and the remainder was unloaded directly from the cars into the storage bin with a 1-cu. yd.

clam-shell bucket, operated by the locomotive crane.

The spouting system used is known as the "Mingo" system. The spouts furnished were nearly circular in section, there being only a 2-in. opening at the top. These spouts did not work satisfactorily, and it was necessary to take them down and widen the opening at the top to 7 ins., after which they gave fair satisfaction. The system of chain blocks used for supporting the spouts from the main cable gave satisfactory results.

The average crew of the central plant, when working at full capacity, consisted of the following:

1 foreman.
1 engineer for hoisting concrete.
1 engineer for unloading stone with the crane.
1 fireman.
4 men in the mixing room.
3 men for handling stone and sand.
2 men for handling cement.
1 man in the hoisting tower.
4 men for distributing concrete.

CONSTRUCTION FEATURES.

Falsework, Centering and Forms.—The falsework for the arches consisted of pile bents, the first bent being 6 ft. 6 ins. from the face of the piers and abutments, the second bent 12 ft. 6 ins. from the first, and the intermediate bents being spaced 14 ft. 6 ins. on centers. Oak piles were used, the piles generally being driven to bed rock. The three outside piles were spaced 6 ft. apart and the intermediate ones 8 ft. apart. The caps used were 12x12-in. yellow pine, the false caps were 6x10-in., the joists were 4x14-in. (spaced 24 ins. on centers), and the lagging was 2x8-in. The proper curve for the intrados was obtained by the use of 2-in. strips, cut to the required curve and nailed to the regular joists. Oak wedges were used between the main and false caps, these wedges being placed in pairs, the pairs being spaced about 4 ft. apart.

In constructing the centering an allowance of 1½ in. was made for camber and ½ in. for settlement after the centering was removed. The actual settlement of the crown after removing the centering was ⅜ in. Figure 1 shows a half side elevation of the centering and its supports and a cross-section of the centering and the center bent.

The forms for the abutments, piers, spandrel walls, etc., were constructed of 2x6-in. studs and 1-in. yellow pine sheeting.

Concreting.—With two exceptions the arch rings of each arch were poured in four longitudinal sections, each section containing 140 to 180 cu. yds.—an average of 150 cu. yds. The east arch of the west structure was poured in three sections, each section containing 200 cu. yds., while the east arch of the east structure was poured in six longitudinal sections. The concrete for the latter arch was mixed in the ¼-cu. yd. mixer, which was placed on the east shore, as the arch was out of reach of the spouting system.

The pouring of the sections was a continuous operation, the average time required being about 9 hours, actual working time. On account of breakdowns three of the sections required about 24 hours to pour.

The spandrel walls, sidewalks, hand-rails, pedestals, etc., were poured, partly from the central plant, partly from the ¼-cu. yd. mixer, and partly by hand, depending on circumstances. The spindles for the hand-rails were made during the winter, the dry process being used. The materials for hand-rails, pedestal caps, lamp posts, etc., were mixed by hand.

The main parts of the hand-rails, the posts, the caps, the lamp posts, and in fact every part of the structure except the spindles for the hand-rails and the tops for the trolley poles were poured in place. No attempt was made to give a trowel finish to the exposed surfaces, except the top surface of the sidewalks, hand-rails, and the caps of the pedestals. The forms for all the ornamental work were made of white pine, and they were painted with crude oil just before being filled with concrete. In addition to painting the forms it was often found advisable to rub them with hard oil, which sealed the cracks and prevented any ragged appearance at the corners.

In constructing the hand-rails, the pedestals were poured first, then the pre-cast spindles were set in place and the hand-rail run; next the caps to the pedestals were poured. Provision was made for expansion and contraction in the hand-rails by placing two thicknesses of ¼-in. corrugated paper against the ends of the sections and by covering this, and the top of that portion of the hand-rail which projected into the cap, with tar paper.

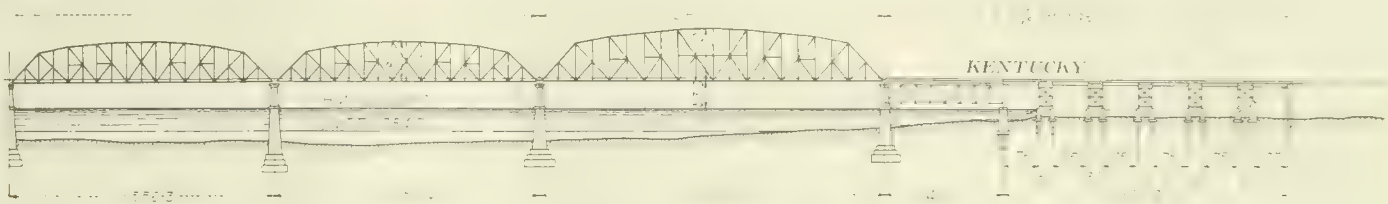
In pouring the lamp posts and trolley poles, provision was made for expansion and contraction by wrapping the trolley poles with one thickness of corrugated paper and two thicknesses of tar paper. The mixture used for the lamp posts and trolley poles was of such consistency as to permit it to be dipped and poured with a bucket. As the concrete was poured into the top of the mould, a man tapped lightly on the sides with a wooden mallet to insure all parts of the mould being filled, and to prevent, as much as possible, the formation of air pockets.

Waterproofing.—The top surface of the arches was waterproofed by coating the concrete with a semi-fluid mortar composed of 1 part cement, ½ part thoroughly slaked lime, and 2 parts sand. After this coating had thoroughly dried another coating, ¼ in. in thickness, of neat Portland cement mortar was applied. For a distance of 15 ft. back from the faces of all piers and abutments, the top surface of the arches, between curb lines, was covered with two coats of liquid asphaltum, applied hot.

Time of Completion, Quantities of Materials and Cost.—The construction work was started May 1, 1911, and during that season the west structure was completed, with the exception of the sidewalks, hand-rails and lamp posts. Of the east structure, there was also completed the two piers, part of the island abutment, the removal of the old structure, the construction of the temporary track, and practically all of the falsework piles were driven.

On March 31, 1912, high water and the breaking of an ice gorge carried away practically all of the temporary track over the east branch of the river, broke off nearly all of the falsework piles, and filled the pits which had been excavated for the island abutment. This damage delayed the completion of the work at least four months. The structure was opened to traffic Dec. 14, 1912, but it was not entirely finished until about May 1, 1913.

The quantities of the various classes of concrete used for these two bridges, exclusive



at Metropolis, Ill.—This Structure Contains the Longest Simple Span Ever Built.

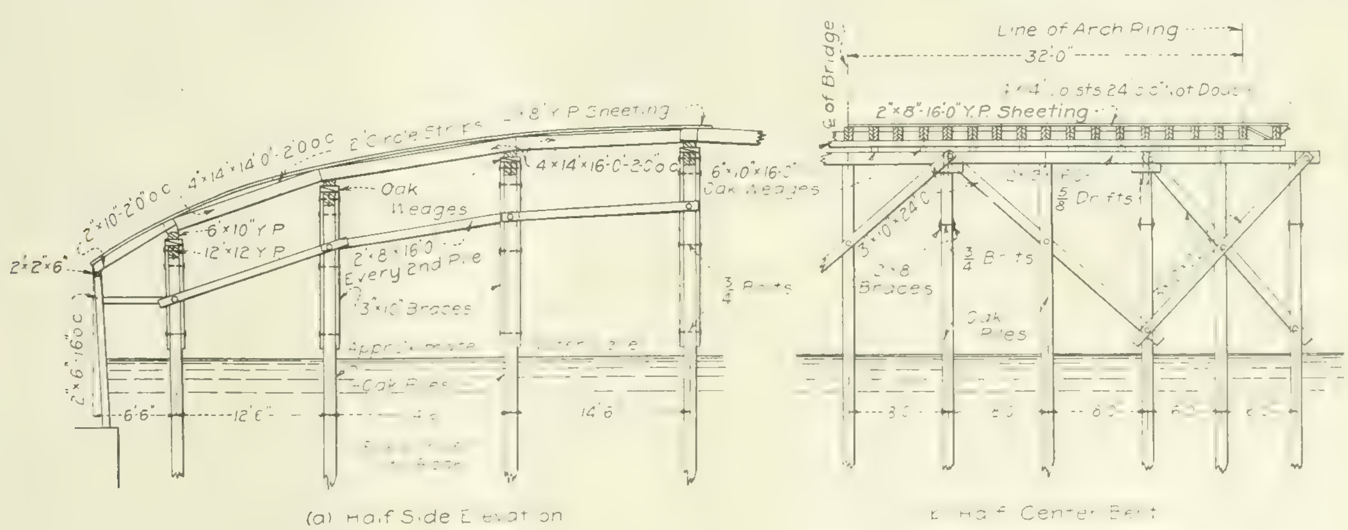


Fig. 1. Half Side Elevation and Half Cross-section of Centering Showing Center Bent for Reinforced Concrete Arches at Third Ave. Cedar Rapids, Iowa.

of hand-rails and lamp posts, were approximately as follows:

1:3:6 concrete	6,590 cu. yds.
[illegible] concrete	[illegible] cu. yds.
1:2:4 concrete	[illegible] cu. yds.
1:[illegible] concrete	[illegible] cu. yds.
Total	13,622 cu. yds.

The cost of the work to the city, not including the roadway paving, was about $148,000, or $10.87 per cubic yard of concrete.

PERSONNEL.

The two bridges were designed by Hedrick & Cochrane, of Kansas City, Mo., under the direction of Percy P. Smith, at that time commissioner of public improvement, who was succeeded in 1912 by Louis J. Zika. The construction work was done by the city of Cedar Rapids, being superintended by Thomas F. McCauley, city engineer. The removal and re-erection of the old steel bridge were in charge of T. C. Basset, assistant city engineer.

Some Data to Facilitate the Proper Proportioning of Concrete Aggregates.

It is quite common practice to specify definite proportions of cement, sand and broken stone or gravel without giving much attention to the properties of the aggregates. In bridge and building work, where large quantities of concrete are ordinarily used, a careful analysis of the aggregates and of the proper concrete proportions should always be made before any definite proportions are specified. The following article, which gives some important data on proportioning the ingredients of concrete, is based on a paper by Albert Moyer, presented before the recent annual meeting of the American Society for Testing Materials. The tabular data given will greatly facilitate the determination of the proper proportions of the aggregates available for any certain piece of work.

To arrive at a scientific and at the same time practical method by which to proportion properly the aggregates for Portland cement concrete we must examine the larger available aggregates and so graduate them in size of particles that the smallest percentage of voids results and, consequently, that less mortar is required to obtain maximum density and therefore maximum strength. The principal requisite is the making of a sufficiently dense and rich mortar to bind together pebbles or crushed stone by filling entirely the voids. The study of this subject must be more particularly directed towards the proportioning of sand, cement and water. The grains of sand must be bound together and the voids between the grains entirely filled with cement paste. It is the proportioning of this paste that we must study in order to determine the amount of cement and water which shall be added to any particular, available sand.

In figuring proportions, we must determine the amount of cement paste required to fill the voids in the sand. (It will be noted that Portland cement, dry, measured loosely, shrinks in volume when water is added, and when measured packed tightly, increases only slightly in volume when water is added. Therefore, we must consider the paste as being the element which must fill the voids in any given sand.) Experiments made by S. Warren Hartwell indicate that 110 lbs. of Portland cement mixed with 25 per cent of water are required to produce 1 cu. ft. of paste. The cement must be measured by weight and not by volume, for it has been found that 110 lbs. of Portland cement will occupy 1.12 cu. ft. if measured loosely and dry, while if packed tightly it will occupy only 0.96 cu. ft. If 94 lbs. of Portland cement is figured as 1 cu. ft., there is not enough cement in the concrete.

PROPORTIONING THE M[illegible]

There are various methods of determining the amount of Portland cement paste required to fill the voids in the sand, among which are: (1) the determination of the percentage of voids by the specific-gravity method; (2) by the water test (that is, the volume of water required to fill the voids); (3) experiments to determine directly the required amount of paste by measuring the volumes of mortars of various proportions; and (4) making up the mortars into briquettes and breaking them in 7 and 28-day periods.

Specific Gravity Method.—To be accurate, the specific gravity of the sand used should first be determined. Then weigh a given quantity of sand, compute the volume it would occupy if solid, and thus obtain the percentage of voids. It is fairly accurate, however, to assume that the specific gravity of a well-washed quartz sand is 2.65, and to base the computations for the percentage of voids in sands on this figure.

TABLE I.—WEIGHT OF CEMENT REQUIRED FOR A GIVEN VOLUME OF CEMENT PASTE.

(1 cu. ft. of paste contains 110 lbs. of cement; therefore 100 cc. of paste contains 6.22 ozs. of cement.)

Proportions by volume.	Volume of cement paste, cc.	Volume of sand, cc.	Weight of cement required, ozs.
1:1½	133	200	8.27
1:1¾	115	200	7.16
1:2	100	200	6.22
1:2¼	89	200	5.53
1:2½	80	200	4.97
1:2¾	73	200	4.55
1:3	66	200	4.11
1:3¼	62	200	3.86
1:3½	57	200	3.55
1:3¾	53	200	3.30
1:4	50	200	3.11

Water Test.—The water test for voids in sand is made as follows: Provide two 250-cc. graduated glasses. Dry the sample of sand to be tested by spreading a thin layer on a pan or piece of tin and by heating to a temperature of over 212° F.; this will determine as nearly as possible an accurate unit of measurement. When the sand is cool, pour 100 cc. slowly into a graduated class, jarring the glass by tapping on the bottom and sides while pouring; this assists in expelling the air. Level off the top with a flat-end stick, in order that the reading to 100 cc. may be accurately made.

In the other glass, measure out 100 cc. of water; pour the dry sand slowly into the glass containing the water, tapping the glass while pouring so as to expel as much of the air as possible. Note the height to which the water rises. If 100 cc. of solid matter had been placed in the glass, the water would have risen to the 200-cc. mark; therefore to ascertain the percentage of voids, deduct the number of cubic centimeters to which the water has risen from 200.

Experiments with Mortars of Various Proportions.—Provide several graduated glasses which contain 250 cc. or over; also provide a balance which will weigh accurately to 0.01 oz. Pour 200 cc. of dry sand into a glass very slowly, tapping the bottom and sides so that it will settle in as dense a mass as possible. Then pour the sand out on the balance and determine its weight to a 0.01 oz. Measure out several more 200-cc. samples of sand, weighing each time as a check on the quantity of sand in each sample. Then determine the volumes of cement paste required to be mixed with the 200-cc. samples of sand to give the various proportions under investigation, and measure out the necessary amount of cement by weight, assuming the weight of cement to be 110 lbs. per cubic foot. These data, for various proportions, are given in Table I. The following proportions, by volume, of cement paste to sand are suggested for investigation: 1:1½, 1:2, 1:2¼, 1:2½, 1:2¾, and 1:3.

Mix each sample of cement with 25 per cent of water. Then mix, first, the sample for the richest mixture with a 200-cc. sample of sand, and tamp all of the resulting mortar into the graduated glass, a little at a time, with a flat-end stick. Note the volume occupied by the tamped mortar. Similarly for the next richest mixture, and so on until all the samples have been mixed into mortar and placed in the glasses.

The object of this test is to ascertain what proportion with a given amount of sand will produce maximum density with a minimum amount of cement. Since the same amount of

TABLE II.—PERCENTAGE OF VOIDS IN CRUSHED STONE.

Weight per cubic foot of crushed stone, lbs.	Hard calcite; limestone; hard marble.	Dolomite; hard limestone.	Medium hard limestone; medium sandstone.	Quartzite; quartz pebbles.	Boston trap; hard sandstone	Hudson Co. trap.	Rockland Co., N. Y., and Ohio sandstone	Flint; hard feldspar	Hornblende
Average specific gravity	2.70	2.85	2.40	2.65	2.80	3.03	2.14	2.60	3.25
80	52.5	55.0	46.5	51.6	54.2	57.6	40.0	50.6	60.5
81	51.9	54.4	45.8	51.0	53.6	57.1	39.3	50.0	60.0
82	51.3	53.8	45.2	50.4	53.0	56.6	38.5	49.4	59.5
83	50.7	53.3	44.5	49.8	52.4	56.1	37.8	48.8	59.0
84	50.1	52.7	43.8	49.1	51.9	55.5	37.0	48.2	58.5
85	49.5	52.1	43.2	48.5	51.3	55.0	36.3	47.6	58.0
86	48.9	51.6	42.5	47.9	50.7	54.5	35.5	46.9	57.5
87	48.3	51.0	41.8	47.3	50.1	53.9	34.8	46.3	57.0
88	47.7	50.5	41.2	46.7	49.6	53.4	34.0	45.7	56.5
89	47.1	49.9	40.5	46.1	49.0	52.9	33.3	45.1	56.1
90	46.5	49.3	39.8	45.5	48.4	52.3	32.5	44.5	55.6
91	45.9	48.8	39.2	44.9	47.8	51.8	31.8	43.8	55.1
92	45.3	48.2	38.5	44.3	47.3	51.3	31.0	43.2	54.6
93	44.7	47.6	37.8	43.7	46.7	50.8	30.3	42.6	54.1
94	44.1	47.1	37.2	43.1	46.1	50.2	29.5	42.0	53.6
95	43.5	46.5	36.5	42.6	45.6	49.7	28.9	41.4	53.1
96	43.0	45.9	35.8	41.9	45.0	49.2	28.0	40.8	52.6
97	42.4	45.4	35.2	41.3	44.4	48.6	27.3	40.1	52.1
98	41.8	44.8	34.5	40.7	43.8	48.1	26.5	39.5	51.6
99	41.2	44.3	33.8	40.1	43.3	47.6	...	38.9	51.1
100	40.6	43.7	33.1	39.5	42.7	47.1	..	38.3	50.6
101	40.0	43.1	32.5	38.8	42.1	46.5	.	37.7	50.1
102	39.4	42.6	31.8	38.2	41.6	46.0		37.1	49.6
103	38.8	42.0	31.1	37.6	41.0	45.5		36.4	49.1
104	38.2	41.4	30.5	37.0	40.4	44.9	.	35.8	48.6
105	37.6	40.9	29.8	36.4	39.8	44.4		35.2	48.2
106	37.0	40.3	29.1	35.8	39.3	43.9	.	34.6	47.7
107	36.4	39.8	28.5	35.2	38.7	43.3	.	34.0	47.2
108	35.8	39.2	27.8	34.6	38.1	42.8	.	33.4	46.7
109	35.2	38.6	27.1	34.0	37.5	42.3	.	32.7	46.2
110	34.6	38.1	26.5	33.4	37.0	41.8		32.1	45.7
111	34.0	37.5	...	32.8	36.4	41.2		31.5	45.2
112	33.4	36.9	...	32.2	35.8	40.7		30.9	44.7
113	32.8	36.4	....	31.6	35.3	40.2		30.3	44.2
114	32.3	35.8	....	31.0	34.7	39.6		29.7	43.7
115	31.7	35.3	....	31.4	34.1	39.1		29.0	43.2
116	31.1	34.7	....	29.8	33.5	38.6	.	28.4	42.7
117	30.5	34.1	....	29.2	32.9	38.0	.	27.8	42.2
118	29.9	33.6	....	28.6	32.4	37.5	.	27.2	41.7
119	29.3	33.0	....	28.0	31.8	37.0		26.6	41.2
120	28.7	32.4	....	27.3	31.2	36.5			40.8
121	28.1	31.9	....	26.7	30.7	35.9			40.3
122	27.5	31.3	....	....	30.1	35.4			39.8
123	26.9	30.7	...	...	29.5	34.9	.	...	39.3
124	....	30.2	....	....	28.9	34.3	.	.	38.8

sand is used in each sample, it is evident that with too much cement the volume of the mortar will be increased. Therefore, in progression from the leanest to the richest mixture, that sample which first starts to increase the volume of the motar is the one which contains the correct proportions for that particular sand.

Tests of Briquettes.—The reason for using 200 cc. of sand in the previous experiment is to obtain sufficient mortar to make three briquettes. By inverting the glass and jarring the top against a rigid surface the mortar will be released, usually in solid mass. Make up each sample of mortar into briquettes, store in a moist closet, and break in 7 days.

This strength test is a check upon the accuracy of the above tests for the proportions of cement to sand which will produce maximum density, maximum strength and maximum bonding. The results of the four tests should agree. If they do not, the tests should be made again. If there is still a considerable variation, wash the sand and retest.

Comments.—The reliability of the four methods just described has been demonstrated by tests.

It has been proposed in various specifications to test sand by comparing its strength with that of standard Ottawa sand. This might cause the rejection of a good sand, for if the comparison is made with 1:3 mortar for each sand, the sand under investigation may not show proper strength in those proportions. The percentage of voids in each sand should be calculated; the mortars should be made up in the proportions necessary to fill the voids in each sand; and the briquettes should be tested for tensile or crushing strength. Each mortar will then be of maximum density for its respective sand, but different proportions are of necessity used. The comparison, therefore, should be made not on the basis of mortars of the same proportions but on mortars of maximum density.

If screenings or quarry tailings passing through a ¼-in. mesh sieve are used in place of sand, their voids are determined in the same manner as those of sand, previously described. The tailings, however, should be of such size that not more than 20 percent pass through a No. 50 sieve and not more than 10 per cent through a No. 100 sieve.

Trap-rock screenings make an excellent substitute for sand. Poor stone screenings, however, such as soft limestone, marble, sandstone, etc., do not make as good a mortar for concrete as does good sand.

PROPORTIONING THE LARGER AGGREGATES

Let us now consider the proportioning of the larger aggregates, keeping in mind that the voids in these aggregates are to be filled with mortar, as previously described. It is sometimes best to use two sizes of stone, a large size and a smaller size; the smaller size, however, should all be retained on a ¼-in. mesh sieve.

To ascertain the best mixture of the smaller and larger sizes of stone or pebbles make a receptacle which will hold a little over 4 cu. ft. (or use a 15-in. sewer pipe). Measure 3 cu. ft. of the larger stone and 1 cu ft. of the smaller stone. Mix well together, place in the receptacle, and note the space which it occupies Empty the receptacle, measure 2 cu. ft. of the larger stone and 2 cu. ft. of the smaller stone, mix as before, and mark the space occupied. Vary the proportions in this manner, always adhering to a total of 4 cu. ft. The mixture which occupies the least space in the receptacle will make the densest concrete.

To ascertain the percentage of voids in crushed stone or pebbles make a box of 3-cu. ft. capacity, the box to be say 1x1½x2 ft. Dry the sand or gravel, heating to over 212° F. Weigh the box empty; throw the stone into the box loose, level off the top with a straight edge, and weigh. Deduct the weight of the empty box from the gross weight and divide the net weight by 3, which will give the actual weight of 1 cu. ft. The percentage of voids may then be obtained by referring to Table II, which is based upon the average specific gravities of the stones tabulated.

In determining the percentage of voids it is essential that the stone shall contain no screenings and shall be free from sand. If any of the particles will pass through a ¼-in. mesh sieve they should be screened out and figured as sand.

The percentage of voids being known, the parts of stone required for various proportions of mortar to fill the voids can now be found. These have been tabulated in Table III.

Example.—Suppose that the sand or screenings selected for use requires a proportion of 1 part of cement and 2½ parts of sand to produce maximum density of mortar and sufficient richness for a good bond, and that the stone at hand is a hard crushed granite of 1-in. size, graduated in particles, and weighing 100 lbs. per cubic foot. From Table II we find the percentage of voids in the larger aggregates to be 43. Referring to Table III it will be seen that the proportions for maximum density in such concrete will be 1 part of Portland cement paste (110 lbs. of cement for 1 cu. ft. of paste), 2½ parts of sand, and 5¾ parts crushed granite; or stated in another way, 110 lbs. of Portland cement, 2½ cu. ft. of sand, and 5¾ cu. ft. of stone.

It may reasonably be stated, therefore, that aggregates proportioned as above described produce a concrete which is denser and stronger than aggregates not so proportioned. Thus, with a given aggregate and an arbitrary specification of 1:2:4, a weaker concrete might be obtained than if the aggregates were properly proportioned in accordance with the principles above described and the proper proportions proved to be, for example, 1:2½:4¾.

If these principles are kept constantly in view, no engineer, without previous knowledge of the aggregates that are going to be used, can conscientiously write a specification and arbitrarily specify 1:2:4, 1:3:6, or whatever his favorite formula may be.

TABLE III.—PROPORTIONS OF STONE EXPRESSED IN CUBIC FEET

Voids in stone, per cent.	Proportions of mortar. 1:1½	1:1¾	1:2	1:2¼	1:2½	1:2¾	1:3	1:3¼	1:3½	1:3¾	1:4
27	5½	6½	7½	8¼	9¼	10	11	12	13	14	14¾
28	5¾	6¼	7¼	8	9	9¾	10¾	11½	12½	13½	14¼
29	5¼	6	7	7¾	8½	9½	10½	11¼	12	13	13¾
30	5	5¾	6¾	7½	8¼	9¼	10	10¾	11¾	12½	13¼
31	5	5¾	6½	7¼	8	9	9¾	10½	11¼	12	13
32	4¾	5½	6¼	7	7¾	8½	9¼	10	11	11¾	12½
33	4½	5¼	6	6¾	7½	8¼	9	9¾	10¾	11½	12
34	4½	5¼	6	6½	7¼	8	8¾	9½	10¼	11	11¾
35	4¼	5	5¾	6½	7¼	7¾	8½	9¼	10	10¾	11½
36	4¼	5	5½	6¼	7	7¾	8	9	9¾	10½	11
37	4	4¾	5¼	6	6¾	7½	8	8¾	9½	10¼	10¾
38	4	4¾	5¼	6	6½	7¼	8¼	8½	9¼	10	10½
39	4	4½	5	5¾	6¼	7	7¾	8¼	9	9¾	10¼
40	3¾	4½	5	5¾	6¼	7	7½	8	8¾	9½	10
41	3¾	4¼	4¾	5½	6	6¾	7¼	8	8½	9¼	9¾
42	3½	4¼	4¾	5½	6	6½	7¼	7¾	8¼	9	9¼
43	3½	4	4¾	5¼	5¾	6½	7	7½	8¼	8¾	9¼
44	3½	4	4½	5¼	5¾	6¼	6¾	7½	8	8½	9
45	3¼	4	4½	5	5½	6	6¾	7¼	7¾	8¼	8¾
46	3¼	3¾	4¼	5	5½	6	6½	7	7¾	8¼	8¾
47	3¼	3¾	4¼	4¾	5¼	5¾	6¼	7	7½	8	8½
48	3¼	3¾	4	4¾	5¼	5¾	6¼	6¾	7¼	7¾	8¼
49	3	3½	4	4½	5	5½	6¼	6¾	7	7¾	8¼
50	3	3½	4	4½	5	5½	6	6½	7	7½	8
51	3	3½	3¾	4½	5	5½	6	6½	6¾	7¼	7¾
52	3	3¼	3¾	4¼	4¾	5¼	5¾	6¼	6¾	7¼	7¾
53	3	3¼	3¾	4¼	4¾	5¼	5¾	6¼	6¾	7	7½
54	3	3¼	3¾	4¼	4¾	5	5½	6	6½	7	7¼

SEWERAGE

Method and Cost of Constructing an Inverted Sewer Siphon of 12 In. Cast Iron Pipe, Encased in Concrete, Beneath Small Stream at Carlisle, Pa.

Contributed by C. A. Bryan, Resident Engineer.

(An article entitled: Method and Cost of Constructing an 18-in. Inverted Siphon for Sewer Crossing of Letort Spring, Carlisle, Pa., was published in this section for April 29, 1914. In the siphon there described a vitrified clay pipe encased in concrete was employed for the horizontal portion. The present article describes the method and cost of installing a similar structure in which cast iron pipe, encased in concrete, was employed.—Editors.)

The 10-in. branch sewer serving the northeastern section of the borough of Carlisle, Pa., joins the main outfall sewer, constructed along the east bank of the Letort Spring, at the corner of North St. and Porter Ave. In order to effect this junction it was necessary to carry the branch sewer across the spring. The original intention was to make this crossing by carrying the pipe through on a straight grade from the junction manhole at North St. and Porter Ave. to the manhole at the corner of North and East Sts., the specifications providing that the pipe beneath the bed of the spring proper be surrounded with 6 ins. of concrete. These specifications were furnished the contractors bidding upon the construction of the lateral sewer system at Carlisle in March, 1913.

The Letort Spring is a shallow stream varying in width from 12 to 30 ft., which flows through the eastern portion of the borough. The elevation of the ground laying to the east of the spring is low and this portion of the borough is at times subject to severe floods. The question of this periodic flooding has been discussed by the borough authorities for several years, finally an election was held and an appropriation authorized to remedy it. A careful study of the local conditions showed that the only way to obtain permanent relief was to convert the present winding channel in which the spring flows into a straighter channel having a proper gradient and with properly sloping sides and having a width at the water surface of 40 ft. The establishment of a proper gradient for the bed of the spring necessitated the lowering of the present bed from 1 to 3 ft., and in order that these changes might be carried out at some future date without injury to the sewer, it was decided to build an inverted siphon across the spring and also to provide a method for flushing out this siphon.

A cross-section of this inverted siphon is shown in Fig. 1.

The original design contemplated carrying the sewer directly through the stone arch bridge crossing the Letort at this point. A careful examination of the site convinced the writer that it was unnecessary to put the contractor to the expense of constructing this sewer through this bridge, especially because the bridge foundations were not solid, and it was feared that the tearing out necessary to permit such construction might result in the complete destruction of the structure. For these reasons it was decided to locate the sewer on the center line of the sidewalk on the north side of North St., as there is a ford there at present. An examination of the stream bed was also made and several borings taken which showed that it was composed of a soft clayey material for a depth of about 4 ft., followed by a stratum of a stiffer clay, and this last was underlaid with rock. On account of the proximity of the sewer line to the foundation of the stone arch bridge it was decided to excavate for the entire crossing at one time and to carry the water flowing down the spring across the excavation by a flume.

Work on the construction of this crossing was begun on Nov. 20, 1913, and the crossing was practically finished on Dec. 3, although the work of finishing the construction of the various manholes, cleaning up, etc., was not completed until Dec. 19.

The spring at the point where the crossing was made was about 33 ft. wide and about 1.9 ft. deep in the deepest portion. A timber frame work was first built from 2-in. by 10-in. plank. This frame work was 32 ft. long and 6.5 ft. wide and 3.5 ft. deep. At the center of the long sides of the frame a flume was built across it. This flume was 5.65 ft. wide and its bottom was 1 ft. 3 ins. below the top of the frame at one side and 1 ft. 6 ins. below it at the other side, thus giving the flume a pitch of 3 ins. in the width of the frame work. On the two long sides the frame was tightly boarded up and 2-in. by 10-in. plank was used for this purpose. It was left open at the ends, however. The frame was stoutly braced at its ends and also throughout its length. Two shallow trenches were then excavated across the stream 6.5 ft. apart on centers, and the frame work was thrown across from bank to bank. These trenches permitted the securing of a more even bearing for the frame work than could otherwise have been secured. When once in position the frame was weighed down with stones and two stout piles were driven at the two upstream corners to serve to hold it in place. Heavy embankments built of a stiff clay were then built from the ends of the frame work to the banks of the stream. To prevent leakage underneath the upstream face of the framework bags of sand were worked into the bed of the stream, both above and below it. Stiff red clay was then packed thoroughly around the upstream face of the frame and the joints between the timbers composing this face were carefully caulked with oakum and this face was made as watertight as possible. Bags of sand were also worked into the bed of the stream around the foot of the downstream face of the frame which tended to prevent water from leaking into the excavation underneath this face. The joints in this face of the frame were also made water tight.

The frame work when completed raised the water level in the spring at the crossing about 1.16 ft. and the depth of water flowing through the flume was 1.2 ft. It is estimated that the discharge of the spring amounted to 25 cu. ft. per second. It should be noted that under normal conditions the water level stood about 3 ins. below the top of the frame on the upstream face. Two heavy rains occurred, however, during construction which raised this level until the water stood within ½-in. of the top of the frame. The flume proved to be large enough to take care of the increased flow and no damage resulted.

The trench for the sewer was then laid out and excavation was started. It was found necessary to tight sheet the trench throughout its length. The sheeting was held in place by two rows of horizontal rangers about 4 ft. apart, vertically, the first set being placed at the elevation of the original bed of the spring. The rangers were made of 2-in. by 10-in. lumber, and the trench was braced by 4-in. by 4-in. braces spaced 5 ft. apart. The material excavated was deposited on the downstream side of the frame work. It was found that the leakage into the ditch was greater than the two gasoline driven diaphragm pumps used could handle, and that this increased as the excavation proceeded. The cause of the trouble was finally located on the downstream side of the frame at the point where the flume returned the water into the spring. The water rushing through the flume and falling upon the soft clay composing the bed of the stream had soon excavated a deep hole which extended nearly to the foot of the sheathing used on the ditch, and water was thus getting into the excavation beneath the sheathing. To remedy this fault the flume was lengthened 12 ft. and a pitch of 1 ft. given it in this distance. After this had been completed no further difficulty was experienced in handling the water leaking into the excavation by means of the diaphragm pumps. Excavation was carried to the depth shown in Fig. 1. Rock was encountered in the last 6 ins. of the excavation but was removed without resorting to the use of dynamite.

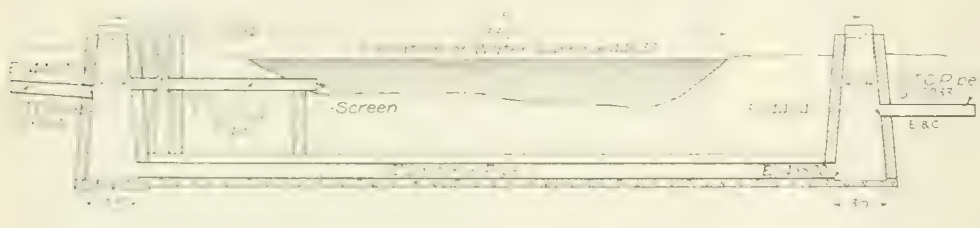


Fig. 1. Longitudinal Cross-Section of Inverted Sewer Siphon and Flushing Device at Carlisle, Pa.

It was decided to use cast iron pipe in this crossing on account of the fewer joints thus required and the greater ease and rapidity with which the contractor could lay it. The pipe was purchased from the Carlisle Gas and Water Co. and as they had a few lengths of 12-in. flanged pipe that had been in service but a short time and was in excellent condition it was decided to use this pipe. A gasket made from stiff cardboard was used at the joints.

Excavation was finished in the afternoon of one day and the concrete foundation was placed before quitting that night. This foundation was allowed to set for a day before laying the pipe upon it. The pipe was surrounded with 6 ins. of concrete as provided in the specifications. Pumping was continued until the pipe laying had been finished. During the placing of concrete, however, the pumping was stopped and the concrete was placed in the water as it rose in the ditch. The concrete was allowed to set for about three days, after which the ditch was refilled. The sheeting was allowed to remain in place both on account of the great difficulty in removing it and, again, it was feared that its removal might endanger and weaken the foundation of the stone bridge.

The excavations for the two manholes at the ends of the crossing were then started and the manholes were built as rapidly as possible. It was found necessary to sheet the sides of these excavations as well. The manholes were built of brick on a concrete foundation 6 ins. thick. As the manholes were located so close to the spring it was decided to surround them with 6 ins. of concrete from bottom to top in order to prevent leakage through them into the system. Even with these precautions it has been found that the manholes were not absolutely tight and the contractor has been put to some considerable expense in reducing the amount of water entering the system through leaks in these manholes.

To provide an adequate method of flushing out this inverted siphon and also the main outfall sewer it was decided to convey water from the spring to the manhole on the west bank for this purpose. A line of 10-in. cast iron pipe was therefore constructed. It was provided at one end with a 10-in. valve which was set in a valve box constructed near the manhole on the western bank. The end projecting into the spring was furnished with a screen to prevent foreign matters from entering and clogging the pipe. The amount of water turned into the system could thus be regulated.

The itemized cost of all work at this crossing was as follows:

Cost of labor.	Amount.
Superintendent, 16 hours at 80 cts.	$ 12.80
Foremen, 106 hours at 40 cts.	42.40
Foremen, 6 hours at 30 cts.	1.80
Excavation, 651 hours at 17½ cts.	113.93
Sheeting and bracing, 100 hours at 20 cts.	20.00
Engineer, gas pumps, 82 hours at 40 cts.	32.80
Mason, 83 hours at 30 cts.	24.90
Mason helper, 83 hours at 17½ cts.	14.53
Carpenter, 57 hours at 35 cts.	19.95
Carpenter helper, 111 hours at 17½ cts.	19.43
Sheeting and bracing, 100 hours at 17½ cts.	17.50
Laying pipe, 61 hours at 20 cts.	12.20
Laying pipe, 50 hours at 17½ cts.	8.75
Concreting, 147 hours at 17½ cts.	25.73
Backfilling, 80 hours at 17½ cts.	14.00
Miscellaneous, 60 hours at 17½ cts.	10.50
Cleaning up, 38 hours at 17½ cts.	6.65
Hauling material, 4 hours at 45 cts.	1.80
Total cost of labor to complete all work at this crossing	$399.67

Cost of materials:	
Bags of cement, 87 at 40 cts.	$ 34.80
Cu. yds. of stone, 25 at $1.25	31.25
Cu. yds. of sand, 13 at $1.75	22.75
Lumber for sheeting and piles, 2.0 M. ft. B. M. at $26	52.00
Flanged cast iron pipe, 48 lin. ft. 12-in.	61.44
Gaskets separating pipe, 6 at 5 cts.	.30
Gallons of gasoline, 80 at 15 cts.	12.00
Manhole frames and covers, 2 at $6.70	13.40
Lamphole frame and cover, 1 at $4.20	4.20
Cast iron pipe, 12 lin. ft. 10-in.	6.40
Valve, 1 10-in. at $18	18.00
Brick, 2,000 at $8.75	17.50
Diaphragms for gasoline pumps, 5 at $3.20	16.00
Gasoline torches, 3 at $1.25	3.75
Pumping with gasoline pumps, 25 days at $1.00	25.00
Total cost of materials and plant	$318.79
Total cost of labor	399.67
Total cost of crossing to contractor	718.46

The itemized cost of constructing the 10-in. flushing device to the contractor was as follows:

Labor building cofferdam, etc., 50 hours at 17½ cts.	$ 8.75
Mason labor, 30 hours at 30 cts.	9.00
Mason helper, 30 hours at 17½ cts.	5.25
Gasoline, 10 gals. at 15 cts.	1.50
Valve, 1 10-in. at $18.00	18.00
Cast iron pipe, 12 lin. ft. 10-in.	6.40
Lamphole frame and cover, 1 at $4.20	4.20
Brick, 300, $8.75 per M.	2.64
Stone, 1.5 cu. yds. at $1.25	1.88
Sand, ¾ cu. yds. at $1.75	1.31
Cement, 7 bags at 40 cts.	2.80
Lumber, 300 ft. B. M. at $26.00	7.80
Total cost of flushing device	$69.53

Deducting the cost of constructing the flushing device from the total cost of the work leaves $648.93 as the cost to the contractor of making this crossing. The length of this crossing from the center of the manhole on the east bank of the spring to the corresponding point on the west bank was 51.5 ft., or the cost of this crossing per linear foot amounted to $12.60.

The method employed in making this crossing was very efficient and it is doubtful whether it could have been installed more cheaply by any other method. A careful study of the conditions as outlined in this article has convinced the author that it could not be employed successfully in all instances.

As in a former instance, the contractor asked for extra remuneration for the extra expenses to which he was subjected by the construction of this inverted siphon, claiming that the substitution of this siphon necessitated the use of materials and plant not specifically covered in the contract. The decision rendered by the chief engineer in this instance was the same as that quoted at length in the issue of ENGINEERING & CONTRACTING for April 29, 1914, wherein an article detailing the method and cost of constructing an inverted siphon of 18-in. terra cotta pipe was presented. In accordance with this decision of the chief engineer the contractor was allowed an item of $207.25 in the final estimate rendered him as representing one-half the cost of the extra labor necessary to complete this crossing. The total expense of the construction of the flushing device was assumed by the borough and 15 per cent was added thereto as the contractor's profit.

This crossing was embraced in the construction of the sanitary sewerage system constructed at Carlisle, Pa., during the past year under the supervision of Mr. T. Chalkley Hatton, now chief engineer of the Milwaukee Sewerage Commission, Milwaukee, Wis. The general contractor for this work was the H. C. Brooks Co., Inc., of Martinsburg, W. Va. Mr. H. M. Sherwood was superintendent of construction for the company and the work here described was carried out along lines suggested by him The writer was resident engineer in charge of the construction of the system.

Method and Cost of Making a Relocation Survey of Underground Pipe Lines, Cincinnati, Ohio.

In the preparation of a comprehensive sewerage plan a record of existing sewers and an accurate topographic map are essential. Therefore, prior to entering upon the work of such a plan for Cincinnati, a thorough examination was made of existing data. It was found that the city's sewer records were both incomplete and inaccurate. Consequently a survey was made which had for its prime object the location and investigation of existing sewers. The scope of the survey was broadened to include the locating and platting of all surface and sub-surface structures in city streets between property lines. This additional work increased the cost of the survey materially, but not at all in proportion to the increase of data secured and the ultimate value of the records. The present article, which is based upon information taken from the recently issued progress report on a plan of sewerage for the city, describes the methods employed in making this survey and records the unit costs of doing the work. The portion of the report drawn upon is entitled: The Underground Survey and was prepared by Mr. O. E. Carr.

GENERAL PROCEDURE.

Initial plans required the field party simply to locate the manholes with reference to the curb, to obtain the distance between manholes, and to secure the elevations of the invert of the sewer or sewers at each manhole. The intention at that time was to make all the plats from existing records and just to fill in the information secured by the Survey. The first party in the field consisted of the chief of party and two men, and they were equipped with only a level, tape and level rod.

All field notes were made on loose leaf sheets. At the commencement of the work the chief of party was furnished with blank sheets and all information was recorded as obtained in the field. It proved unsatisfactory not to have these sheets numbered in advance, for only in that way was it possible to cross index properly in the field,—furthermore a sheet was frequently held back for some cause and in that way lost its place among contiguous sheets. It was also found advisable to have street and curb lines placed on sheets as well as street names in order to save time of the party in the field. For these reasons sheets were first laid out in the office and numbered before being given out, a series of numbers being assigned the sheets of each field party. For instance the sheets given to party No. 2 were in the 20,000 series. The loose leaf system proved entirely satisfactory for these field notes.

Change of Methods.—It was soon evident that the city could not be covered by connected plats made from any existing records. All existing plats of the city tacitly acknowledged this fact as they break off at the street line,—only one side of the street being shown on each plat. Angles on the recorded plats are given by bearings. Then, too, magnetic bearings are subject to change, and endless trouble would be encountered in joining different plats, provided all recorded plats were complete and correct in every particular. That this last assumption is a condition contrary to fact was soon discovered.

Accurate and connected plats could not be made from field notes only referenced to the curb of streets, since the streets themselves were often located by a plus or minus distance on the existing records. Therefore methods were at once changed and field parties were required to locate all sewer manholes, curbs, and street lines with the transit from a closed traverse line. Sufficient information was demanded of the field party to enable the draftsman to have an absolute check on all the field work. These later methods were embodied in Instructions to Field Parties, Underground Survey, given out in the fall of 1912. A summary of these instructions follows:

TABLE I.—COST DATA OF CINCINNATI UNDERGROUND SURVEY FOR FIELD AND OFFICE WORK.

Month.	Field work. Progress in miles.	Cost per mile for month.	Mileage sewers surveyed.	Av. cost per mile.	Estimated cost based on 500 miles.	Platting record plans. Progress in miles.	Cost per mile for month.	Mileage sewers platted.	Av. cost per mile.	Estimated cost based on 500 miles.	Total estimated cost, field and platting.
1912—											
December ..	11.1	$150.17	149.7	. .		27.4	$39.14	87.4	...		. . ..
1913—											
January ..	19.4	86.05	169.1	$75.74	$37,869.40	27.5	40.06	114.9	$52.93	$26,359.84	$64,229.24
February ...	22.5	84.72	191.6	76.79	38,395.38	25.1	42.80	140.0	51.12	2,550.00	63,955.33
March	20.9	84.38	212.5	77.53	38,766.80	33.5	36.21	173.5	48.23	25,500.00	63,889.69
April ..	26.5	64.62	239.0	76.10	38,051.38	28.5	36.94	202.0	46.64	23,321.42	61,372.80
May ..	43.0	45.17	282.0	71.39	35,694.61	17.7	50.99	219.7	47.00	23,499.29	59,193.90
June ..	47.9	43.37	329.9	67.32	33,660.11	22.5	47.20	242.2	47.02	23,510.93	57,171.04
July ..	49.3	43.88	379.2	64.27	32,136.03	33.8	37.63	276.0	45.87	22,935.94	55,071.97
August ...	44.7	48.87	423.9	62.65	31,324.40	31.5	36.87	307.5	44.95	22,475.76	53,799.66
September	47.0	43.77	470.9	60.77	30,382.59	29.4	38.01	336.9	44.34	22,171.76	52,554.35
October	17.6	69.52	488.6	61.07	30,534.04	16.9	65.05	353.8	45.33	22,666.67	53,200.71
November .	7.4	87.47	496.0	61.46	30,730.92	20.3	53.00	374.1	46.02	23,010.87	53,741.79

Note.—Costs based on payroll expenses only.

INSTRUCTIONS TO FIELD PARTIES.

Each party will be assigned to some portion of a drainage district which must be entirely completed before another assignment is made. With each assignment, the chief of party will be given the necessary field sheets for that assignment. A specimen field sheet properly filled out is shown in Fig. 1.

Order of Procedure.—1. Bench marks around an assignment. 2. Surface location from traverse points and lines and all traverses run to closures. 3. Sewer investigations at outlets, manholes, catch basins, etc. 4. Levels over manhole tops. The chief of party must personally be in charge of the above operations.

Bench Levels.—To run bench levels around an assignment first, is usually an advantage. Leave enough bench marks to check levels over manholes on cross streets. Run circuits through at least two previously established bench marks, preferably precise level bench marks. Close the circuits. Allowable error .05 $\sqrt{M}$, where M. equals the distance in miles over which levels are run, for instance in a mile circuit the allowable error is .05 ft. Take the rod readings on bench level circuits to thousandths of a foot Keep level notes in the duplicating level book. When the levels have been run on an assignment, send the original notes to the main office keeping the duplicates in the book retained by the chief of party. The chief of party will send one or two rodmen with a general sewer plat of the district over each street in the assignment to determine whether there are any manholes which cannot be found. In case manholes cannot be found, the office should be asked for further description in order to locate the manholes in the field. This work is done during the running of bench levels.

Location.—Usually, it will be found preferable to complete the traverse and location about the outer limits of the assignment, then traverse all the streets in one direction, and finally, the streets running in the opposite direction. Each traverse must be closed by both angle and distance. When the traverse about the outer limits of the assignment is finished, a sketch shall be made showing all angles and distances in this closed traverse.

Whenever possible, the sewer line shall be used for the traverse line and the centers of manhole covers for the traverse points. When the sewer line is used as the traverse line, all distances between manholes shall be measured twice and both the measurements recorded. Measurements shall be made to hundredths of a foot and the limiting allowable error shall be .01 ft. in 100 ft.

In case traffic interferes with the occupying of manholes, traverse points may be selected as best suit conditions, but preferably opposite manholes. In this case, both traverse and sewer lines shall be measured once.

In traverse and location work the Angle and Distance Method must be used. Angles must always be turned in a clockwise direction, without inverting the telescope. At each instrument station, after the last object has been located, the telescope must be turned on the back sight and the angle read, which of course should be 360°.

All streets must be located whether sewered or not, and all sewers must be located whether on streets or not. Open channels for sewage must be located with dimensions.

Location from traverse in streets must include the following: Street and curb lines, corners, etc. Sewer center lines. Manholes, sewer, electric, telephone, etc. Inlets and catch basins. Valves, water and gas. Fire hydrants and fire cisterns. Culverts (obtain size). Bridges. Electric and steam railroad tracks.

On all location other than sewer or traverse, distances shall be read to tenths of a foot only.

Investigation of Sewers and Manholes.—This investigation shall include the following: Cover; depth; diameter, top and bottom; condition.

Sewers entering or leaving manhole: Size and shape; material; depth of flow and time; condition; type.

Note shape and size of manhole cover. Depth of manhole shall be measured with care. This measurement shall be taken from the down stream edge, of rim of casing to the true invert of the sewer. In case there are other pipes entering or leaving the manhole, measure depths to each. Diameter top and bottom shall be measured. If the manhole is a summit or terminal manhole this fact shall be noted also. Locate any water or gas or conduits passing through the manhole, giving size.

Note in case the walls are in need of repairs. Also state whether the manhole shows evidence of having been flooded or is partially filled. In case of a terminal manhole note if it is a flushing manhole and state the nature and condition of flushing apparatus.

Measure horizontal and vertical diameters of sewer at outlet, and also at all manholes, and state whenever a change in size is found. In case of a special outlet structure, make a sketch showing dimensions. State whether sewer is of vitrified pipe, brick, stone or concrete. Depth of flow in sewer at manholes and at outlet structures shall be recorded together with the time observed.

Attention shall be given to the condition of the sewer, both from a structural and sanitary viewpoint. Note if any repairs are needed, and especially observe if the invert is badly worn. Note also if the sewer indicates stoppage or flooding. Determine and note the kind of sewer, whether sanitary, storm, combined, or intercepting. Note location of lamp holes and their size, and depth to sewer invert.

The investigation of sewers at manholes, outlets, etc., is a vitally important part of this survey, and the chief of party shall not leave any point of this work, until he is satisfied that he has secured and recorded all the information required or made adequate explanation on the field sheet why such information cannot be found. During the investigation at manholes the chief of party should examine each field sheet carefully to see that all data are clear and

Daily Reports.—Daily reports shall be made and sent to the office each day on cards similar to the form shown in Fig. 3.

Weekly Reports.—Each chief of party shall make a weekly report showing the number of feet of sewer surveyed during the week. This report is to be made street by street, and a copy is to be kept by the chief of party in order to avoid repetition.

In addition to this report of progress, a weekly report of defective sewerage conditions as well as complaints shall be made to the office.

FIELD WORK 1913.

A typical field party, together with rates of pay, as they were organized in 1913, is here given:

Position.	Rate per month.
Chief of party	$90
Instrumentman	60
Rodman	50
Rodman	50

The number of field parties was gradually increased from one on May 1, 1912, to five on Aug. 23, 1912, this number was continued until Jan. 16, 1913, when the sixth party was put in the field. The number of field parties was increased to eight in May, 1913, and with this field force of 32 men working during the summer, the field work was completed. The last field party was disbanded on Nov. 22, 1913.

The accompanying cost report, see Table I,

The plans for this work include the binding of cloth prints made from these tracings into loose leaf books. Sheets for contiguous territory are to be bound in the same volume, the sheets are numbered and cross numbered, as well as shown on a key map given on the first page of the book.

A contract for the making of six white cloth prints from each of these sewer record plans was entered into with the Lithoprint Company of New York. These prints cost 36 cts. each, and will be used to advantage in the various city departments.

An idea of the extent of this work is obtained from a view of mere preliminaries for it, requiring: 1. Tracings of the entire city, made from plats in the County Auditor's office. 2. Complete copies of all plats in the office of the Gas Co. These plats show sizes of gas main and their locations in the streets. 3. Complete tracings of all water information, covering the entire city were made from the records of the Water Works, showing sizes of water mains and their locations in the streets. 4. Complete information from the telephone company showing the location of their conduits and the number of ducts in each.

The plats show all lot numbers and frontages; street and curb lines and also car tracks are shown by continuous lines. Sewer information is featured, sewer lines being shown by

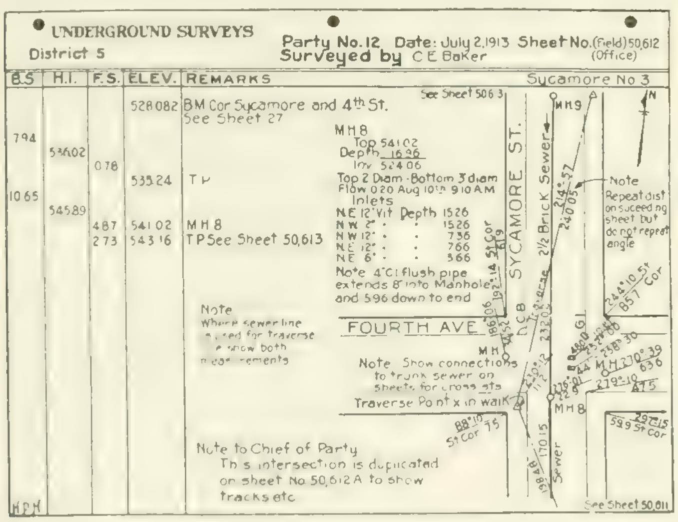


Fig. 1. Typical Field Sheet, Cincinnati Underground Surveys.

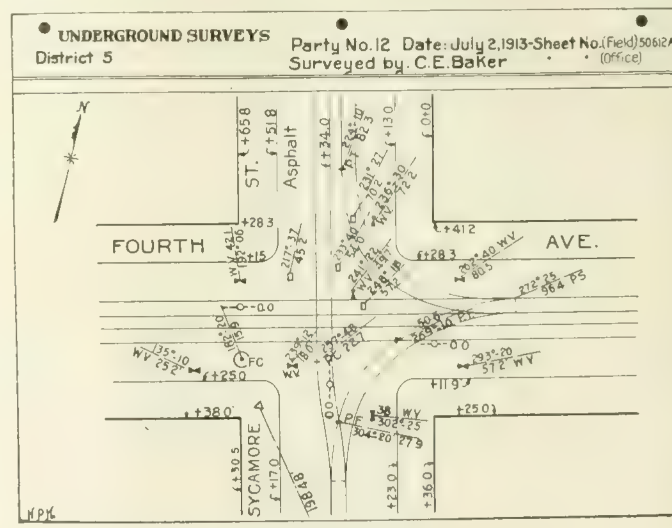


Fig. 2. Special Field Sheet for Important Intersections, Cincinnati Underground Survey.

complete as regards location and investigation. A daily record of defective sewerage complaints, etc., should be kept in a pocket note book and a written reports of these facts made every week.

Levels Over Manhole Tops.—Run all levels to and from bench marks established on bench level circuits. The elevation of the down stream side of the manhole casting should be used for the manhole elevation. Rod readings are to be taken to hundredths of a foot only. Make no erasures of observed readings, but in case of error draw a line through the incorrect figures.

Record notes as shown on the accompanying specimen field sheet, Fig. 1. Level notes must always read down the page. Elevation of sewer invert is required, not the elevation of the manhole top. Chief of party must make this reduction on the field sheets.

Field sheets should now be complete. As each street has been covered at least three times, it is assumed that all features are determined. The sheets are now ready to be turned into the office as completed work.

Field Sheets.—Each chief of party will be given field sheets only after they are prepared by drawing in the street lines, etc., and numbered. These sheets are charged to him and they are either to be returned as completed work or accounted for in some other way, in order that the files may stand complete.

Blank sheets will be furnished on which will be shown the traverse lines, street lines, car tracks, etc., of those street intersections which contain too much detail to be clearly shown on a single sheet. See specimen herewith in Fig. 2.

shows a decrease in the average cost per mile during this time from $77.53 for March to $61.20 for November or a decrease of $16.33 in the average cost. There is also a decrease of $8,166.80 in the estimated cost to complete the work. During this same time 283.5 miles of sewers were measured at a cost of $13,880.52, which gives an average per mile cost for this period of $48.96. The average costs are based on payroll charges only.

Complete information, as called for in Instructions to Field Parties, was obtained on 496 miles of sewers, together with other information pertaining to the streets of the city. This includes 42 miles of sewers within the city limits for which there were no records.

SEWER RECORD PLATS.

The primary purpose of these plats is to show correctly all information regarding the sewers:—their sizes, grades, location, inlets and branches. They include all improved portions of the city. These sheets also give information regarding all other underground structures. They show, therefore, the best location for new sewers or pipes. Formerly, it has been necessary to visit the various corporations having pipes or conduits in the streets in order to get this information.

These record plans are 23x32 ins., within the border and a binding edge of 1½ ins., is left on the lefthand end of the sheet. One portion of the city is platted on a scale of 40 ft. to the inch; all other parts of the city are platted on a scale of 50 ft. to the inch.

very heavy continuous lines, while all other sub-surface structures are named and shown by broken lines. Water and gas mains, and electric, telephone and telegraph conduits are shown in this manner. The location of gas mains depends wholly upon information obtained from the office of the Gas Company. All surface indications of underground structures, as manholes and valves were located in the field and are shown to scale on these plats. While the lines as laid may not be wholly straight between valves or manholes, the deviation probably will not exceed the error in scaling from the plat in locating an intermediate point on any line.

PROCEDURE IN PLATTING SEWER RECORD SHEETS

The controlling traverse of the area to be platted was first figured and platted by coordinates on detail paper. R. L. Gurden's Traverse Tables were used in figuring traverses; the latitudes and departures were afterwards checked by the same tables or by means of the slide rule. The traverse of cross streets was usually platted by means of the scale and protractor. An interior traverse was figured only when it failed to close by this method. This did not often occur, and when it did, was usually accounted for by a blunder in the field work. The method of platting by co-ordinates proved to be a timesaver, while at the same time it was far more satisfactory to the detailer as it always gave him some definite and fixed points on which to base his details.

The platting of the traverse was followed by the locating of curbs and street lines and the showing of other information obtained in the field, as distance between sewer manholes, size of sewer, elevations of invert of sewer, and depth and location of valves and other manholes.

The lot numbers and frontages, together with property lines were then shown as taken from the Auditor's plats. In case of disagreement the recorded plats as well as the deeds were examined. In no event could any existing records change field information as platted from a closed traverse.

The water sheets were examined, the size and location of water mains obtained and the locations were checked by position of the valves located in the field notes.

The gas sheets were examined in order to obtain the size and location of existing gas mains. No check in the field could be had on this information.

Width of telephone conduits was obtained from the company. This was now shown together with location of manholes as taken from field notes.

The grades of the sewers were then figured, after which the completed detail was ready to be traced.

The operation of checking was nearly as laborious as that of detailing, since everything except the platting of closed traverse had to be checked. A record was kept of the accession number of drawing, names of streets covered, number of feet of sewer shown on each street, together with name of detailer, tracer and checker.

Elevations of inverts of sewers at manholes are given and not the elevations of the manhole tops. This is done because the elevations of the inverts are fixed while the elevations of the tops are often changed with the improvement of the street.

The progress and costs of the year, December, 1912-November, 1913, during which time methods have remained practically unchanged, are shown in Table I. The increase in the cost of the field work at the end of the season is due wholly to vacation time and the locality in which most of this work was done.

VALUE OF SEWER RECORD PLANS

The usefulness of these sewer record plans has been clearly demonstrated during the past year. Demands have been made constantly for information found in the records of the department which is not readily accessible elsewhere.

The Water Department has found the record plans highly serviceable in the making of plats of certain outlying parts of the city, concerning which little or no information could be obtained elsewhere.

The subway surveys have utilized the information contained solely in the records of this department in regards to the depth of certain sewers along the proposed subway line.

The Street Department in the case of a proposed street improvement can at once find out whether all the necessary pipes and conduits are already laid in the street. This is much easier than to consult the records of the various public corporations or to determine the same by inspection of the street.

The City Solicitor has made profitable use of the Record Plats. On several occasions he has been able to show conclusively in the courts whether adequate sewerage facilities were or were not provided for certain parties.

Private individuals have frequently come to the office in order to obtain definite information regarding depth or location of a sewer adjacent to property which they owned.

Date *June 30.*	DAILY REPORT	Weather
Party No. 12.	Divion of Sewerage Investigations, Designs and Plans	*Cloudy. Rain. 3-5*

Kind of Investigation *Underground Surveys*

District	Working on	From	To
Cumminsville	*Apjones Line, Levels and M. H. Fergus, M. H. No. 4, Mad Anthony*		343.80
	Chase Line, Levels and M. H., Hamilton-Dane		2,015.45
	Lingo Line, Levels and M. H., Fergus,		890.00
			3,249.25

Names	How Employed	Hours
John Jones,	*Measuring Line and M. H.*	6
R. J. Coe,	" " "	6
W. J. Coe,	" " "	6
M. J. Tape,	*Levels on M. H.*	6
P. S. Cole,	" " "	6

Expenses

Remarks *Lost two hours account of rain.*

(Do not sign here)

Engineer in Charge

Fig. 3. Specimen Daily Report of Field Parties, Cincinnati Underground Survey.

Scope of Engineering Reports and Plans for Sewerage and Sewage Disposal Works in Saskatchewan.

Suggestions on designing sewage treatment works for Saskatchewan cities were published in this journal of April 8, 1914. The present article relates to the scope of engineer's reports and plans in connection with sewerage improvements as stipulated by the Bureau of Public Health of the same province. The regulations also cover estimates of cost, specifications and affidavits but those provisions are omitted from the article.

SANITARY SEWERAGE SYSTEMS.

Engineer's Report.—A report shall be submitted, prepared by the engineer acting for the municipality, giving information under the following heads, in so far as such are applicable to the proposed works:

1. Population.—The present population and the rate of increase during each of the past five years. Available data justifying a future increase of population and probable increase in next ten years. 2. Maximum population which system will provide for if fully developed. 3. Population provided for by proposed works. 4. Area and topography of municipality. 5. Water consumption in municipality per capita per 24 hours. 6. The character of the sewage (with reference to surface and roof water, trade wastes, etc.). 7. Estimated maximum rate of flow expressed in gallons per hour. 8. Estimated normal dry weather flow expressed in gallons per 24 hours. 9. The general nature of subsoil throughout the municipality. 10. Precautions to prevent infiltration in water-bearing strata. 11. The minimum grades of all sizes of sewers. 12. Provision made for flushing and the intervals of flushing. 13. Character of pipes and joints to be used. 14. Type of sewage lifts to be adopted. 15. The maximum distance between manholes. 16. Ventilation of system. 17. Storm overflow discharges and any points of discharge other than the sewage disposal plant. 18. The areas of the municipality which cannot drain by gravity into proposed system. The proposed method of providing drainage for such areas in the future. 19. Provision to be made for inspection of construction. 20. Control and operation.

In case of extensions to an existing system, the report shall in addition to describing the nature of the proposed extensions, deal with as many of the above matters as are thereby affected.

General Plans.—The following plans shall be submitted:

Plan of Municipality.—A general plan of the entire municipality to a scale of not greater than 200 and not less than 500 ft. to an inch, this plan shall show: 1. Municipal boundary. 2. All streets existing or proposed. 3. The approximate location of all habitable buildings not connected with sewers at the date of application. 4. The surface elevations of all existing or proposed street intersections. 5. The location, size and direction of flow of all existing proposed sewers. 6. The location of all existing and proposed manholes, lampholes, flush tanks, sewage lifts, sewer outlets, overflows and other appurtenances. 7. Any areas from which it is proposed to pump the sewage; these should be indicated by light coloring or shading.

Contour Plan.—A contour plan of the municipality to a suitable scale.

Detail Drawings.—Detail drawings shall be submitted to a scale which shall clearly indicate the design of all parts of the proposed system or extensions, including all manholes, lampholes, flush tanks, siphons, pump wells, inspection chambers, buildings, iron work, machinery and other appurtenances. Transverse sections of all sewers over 24 ins. in diameter shall be submitted.

Profiles.—Profiles of all sanitary sewers shall be submitted. The following scales are suggested for general adoption:

Vertical, 10 ft. to an inch, or 20 ft. to an inch.

Horizontal, 80 ft. to an inch, or 100 ft. to an inch, or 200 ft. to an inch.

Profiles shall show all manholes, lampholes, flushtanks, siphons, river and railway crossings, the size and grade of sewers, the elevation of sewer invert and ground surface at all manholes and changes of grade; elevation of river or stream beds crossing the line of sewers.

STORM SEWERAGE SYSTEMS AND EXTENSIONS.

Engineer's Report.—A report shall be submitted, prepared by the engineer acting for the municipality, giving information under the following heads, in so far as such are applicable to the proposed works:

1. Precipitation.—All available data having reference to the maximum precipitation in the municipality, particularly with reference to short periods of heavy precipitation. 2. Nature of ground surface.—The areas paved and unpaved, and in the case of the latter, the degree of porosity of the ground. 3. The area and mean slope of each district draining to a trunk sewer or point of discharge. 4. The data and assumptions upon which the computation of the size of sewers is based. 5. The estimated maximum volume of flow (in cubic feet per second) at each point of discharge, and the estimated increase when the system is fully developed. 6. The character of pipes and points to be used. 7. The nature of coating (if any) to be used on the outside of pipes. 8. Any connections which it is proposed to make with the sewers, other than for surface water. 9. Provision to be made for inspection of construction. 10. Control and operation.

In the case of extensions to an existing system, the report shall, in addition to describing the nature of the proposed extensions, deal with as many of the above matters as are thereby affected.

General Plans.—A general plan of the entire municipality shall be submitted to a scale of not greater than 200 and not less than 500 ft. to an inch; this plan shall show:

1. Municipal boundary. 2. All streets existing or proposed. 3. The surface elevations of all existing and proposed street intersections. 4. The location, size and direction of flow of all existing and proposed storm sewers. 5. The location of all existing and proposed manholes,

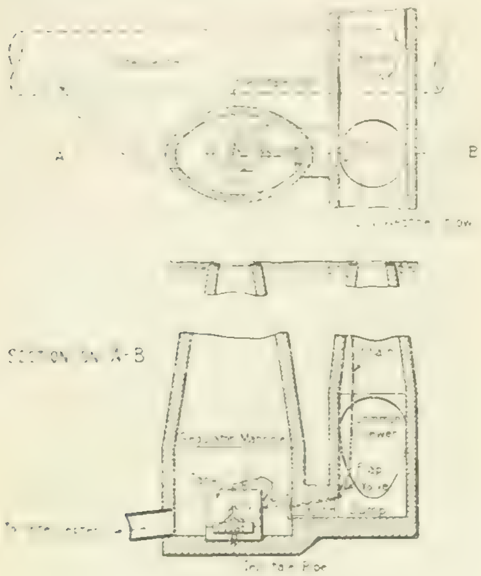


Fig. 1. Typical Sewage Flow Regulator.

catch basins and connections to the storm sewerage system, also all points of discharge and natural watercourses. 6. The extent of street paving. 7. The areas draining to each trunk sewer or point of discharge (these may be shown by distinctive coloring or shading).

Detail Drawings.—Detail drawings shall be submitted to a scale which shall clearly indicate the design of all parts of the proposed system or extensions, including all manholes, catch basins, valves, etc. Transverse sections of all sewers over 24 ins. in diameter shall be submitted.

SEWAGE DISPOSAL WORKS AND EXTENSIONS

Engineer's Report.—A report shall be submitted, prepared by the engineer acting for the municipality, giving information under the following heads, in so far as such are applicable to the proposed works:

1. Population.—The present population and the rate of increase during each of the past five years. Available data justifying an increase of population and probable increase in next ten years. 2. Maximum population which system will provide for if fully developed. 3. Population provided for by proposed works. 4. Water consumption in municipality per capita per 24 hours. 5. Estimated maximum rate of flow of sewage arriving at works, expressed in gallons per hour. 6. Estimated daily flow of sewage arriving at works. 7. The number of connections already made to sanitary sewerage system. 8. The character of the sewage (with reference to surface and roof water, trade wastes, etc.). Any excess over domestic sewage shall be estimated in gallons per hour under: (a) Roof water; (b) surface water; (c) infiltration water; (d) trade waste water. In the case of trade discharge the character of the waste shall be given. 9. The distance and location in relation to the works of any wells or underground source of water supply. In the case of the final effluent discharging into a watercourse or lake, it shall be stated whether such watercourse or lake is, or eventually becomes, a source of domestic supply or is used for watering cattle. 10. Minimum volume and velocity of stream or river into which the final effluent will be discharged. 11. Area of land acquired for sewage disposal purposes. 12. Character of subsoil through which sewerage system is laid, with particular reference to the presence of sand. 13. Character of treatment processes to be adopted. 14. Type of sewage lifts to be used at sewage disposal works. 15. Provision for dealing with sewage in the case of failure of sewage lifts. 16. Type of screens and provision for the removal of screenings. 17. Preliminary precipitation of mineral particles—velocity of flow in detritus tanks. 18. Type of sedimentation tanks, capacity and velocity of flow in tanks, inclination of base of tanks. 19. Method of drying sludge and final disposal. 20. Capacity of dosing or siphon chamber. 21. Type of filter beds; character, area and depth of filter media. Rate of filtration. Method of distribution over surface of media. 22. Retention of humus. 23. Disinfection of effluent. 24. Provision against frost. 25. Location of bypasses. 26. Provision for measuring and recording flow of sewage. 27. Future extensions. 28. Provision for grading, surfacing, etc. 29. Control and operation.

In the case of extensions to existing works, the report shall, in addition to describing the nature of the proposed extensions, deal with as many of the above matters as are thereby affected.

General Plans.—The following plans shall be submitted:

Location Plan.—A location plan to a suitable scale showing:

1. The location of the works in relation to the site of municipality. 2. The route of the main outfall sewer. 3. The layout of the various units of the sewage disposal works, including all drains, pipe lines, etc. 4. The area of land to be utilized for sewage disposal purposes. 5. The lake, watercourse or subsoil, into which the final effluent will be discharged. 6. Elevation of high, mean and low water in watercourse or lake. 7. Highest known flood elevation of watercourse. 8. Elevation of effluent drain at point of discharge and of outfall sewer at entrance to works.

Sketch Plans.—Preliminary sketch plans shall be submitted to a scale which shall indicate the type and capacity of the various tanks, filter beds, etc.

The principal elevations and measurements of the various parts of the work shall be indicated on these plans. Such preliminary plans shall give sufficient information to enable the commissioner to arrive at a conclusion as to the efficiency of the proposed processes of treatment and design, and structural details may be omitted therefrom. Sketch plans need not be submitted if the detailed drawings required accompany the application.

Detail Drawings.—Detail drawings shall be submitted to a scale which shall clearly indicate the design of all parts of the proposed works. Longitudinal and transverse sections of all tanks and filters shall be shown.

Profiles.—Profiles of all pipe lines and drains shall be submitted drawn to a suitable scale.

Notes on the Design of Regulators and Storm Water Overflows for Sewers.

As stated in the article on the design of the new intercepting sewers for the city of Cincinnati, published June 17, 1914, adequate storm water overflows must be provided to convey to the nearest water course the excess storm water beyond the quantities which the intercepting sewers may be able to receive. The subject of storm water overflows and sewage flow regulation is discussed in the present article, the matter given being taken from the portion of the Cincinnati sewerage report which relates to the design of intercepters. This portion of the report was prepared by Mr. E. J. Miner.

The function of a storm water overflow is to supply a means of removing storm water in excess of a certain definite flow in the sewer to the nearest water course, or to a storm water channel provided for that purpose; in other words, to allow the escape from the sewer of all excess over a fixed quantity of sewage. A sewage flow regulator, conversely, is intended to prevent surcharging of an intercepting sewer by partly or wholly closing the inlet connections from the branch trunk sewers. The general object to be attained is the same in both cases, namely, to allow the admission of domestic sewage to the intercepting sewer and to divert the storm water to other channels; but an overflow provides an outlet for storm water from a trunk sewer, while a regulator prevents the admission of storm water to an intercepter.

Overflows and regulators are frequently used in combination, the former allowing the excess above a certain flow to be discharged into storm water drains, the latter entirely cutting off inflow to the intercepter when the quantity of sewage in the latter has reached

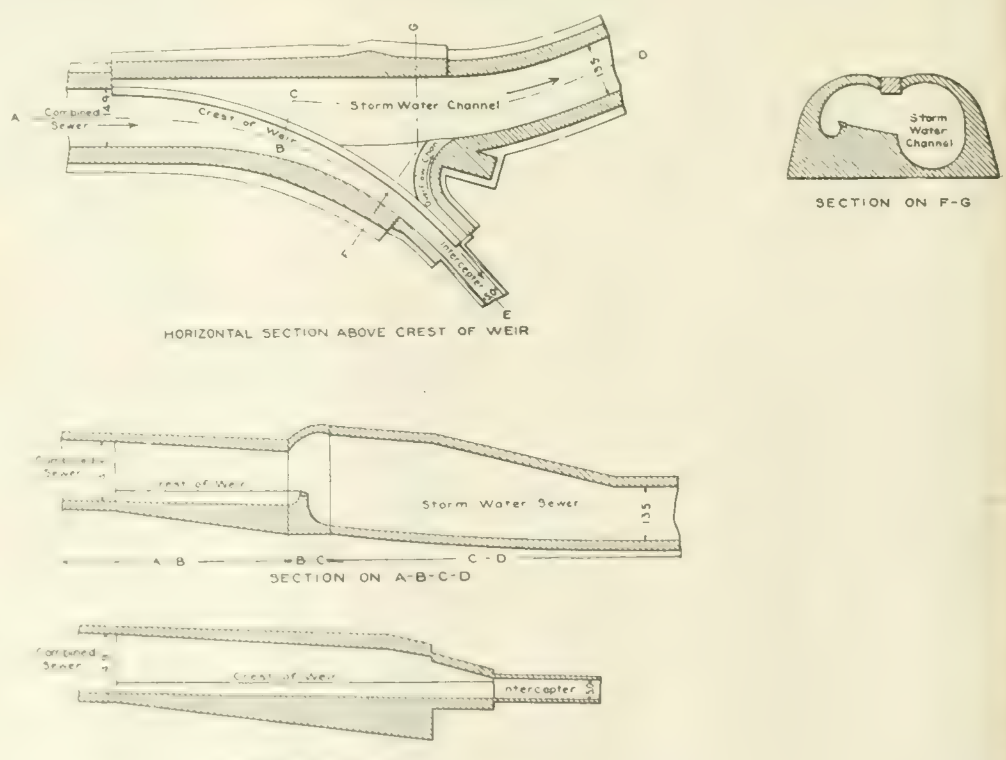
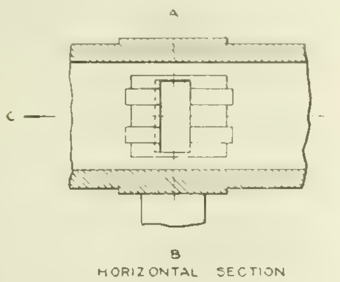


Fig. 2. Details of Overfall Weir for Storm Water Overflow, Walworth Sewer, Cleveland, Ohio.

a certain point and when it is in danger of being surcharged.

Regulators.—A regulator is a mechanical appliance by which a gate is automatically closed by the rising of a float whose motion is controlled by the elevation of the sewage in the intercepter. A typical regulator is shown in Fig. 1.

The use of regulators of this type is open to the objection that frequent attendance is required in order to keep the apparatus in proper working condition. The regulators must be inspected and cleaned at short intervals, and after each storm must be carefully overhauled, if they are to be depended upon to perform their function.



tion leading to the intercepting sewer, while the storm waters leap the opening and pass on through the storm water channel.

A typical leaping weir is shown in Fig. 3, which illustrates an inlet constructed in connection with the Menomonie Intercepter at Milwaukee, Wis., from designs of George H. Benzenberg. Like the overflow weir, this type should require no attention, unless the opening becomes clogged by planks or other large substances which are not supposed to find their way into sewers, although they sometimes do. It is open to the same objection as the over-fall weir, that the gravel and sand are washed into the intercepter, although there is more likelihood with this type of overflow that a portion of the silt may be discharged with the storm water.

In view of the attention necessary to maintain regulators in proper working order, it is desirable to avoid their use as far as possible. Wherever found possible by studies of the details of design, the leaping weir type of overflow for the connections of the smaller trunk sewers and the over-fall weir for the larger sewers should be used rather than float operated regulators.

It must not be forgotten that in the case of trunk sewer outlets which are below the level of high water in the river so-called tide-gates

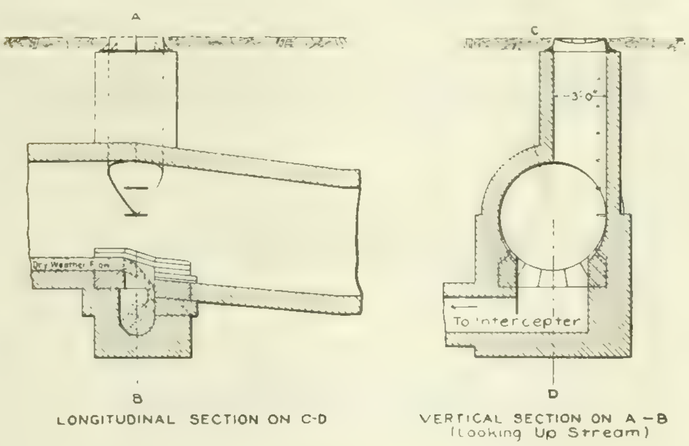


Fig. 3. Details of Leaping Weir for Storm Water Overflow, Menomonie Intercepter, Milwaukee, Wis.

Over-Fall Weirs.—There are two types of storm overflow, over-fall weirs and leaping weirs. An over-fall weir, as the name implies, is an outlet with a sill or crest arranged either across the sewer or longitudinally in one side of the structure, so designed that the excess flow, after the sewage reaches the height of the crest, is taken to a storm water channel, while the ordinary flow passes the weir and is discharged into the intercepter.

An excellent example of an overflow of this type is shown in Fig. 2, which illustrates the storm water overflow constructed in connection with the Walworth sewer in Cleveland, according to the designs of W. C. Parmley. In the case of large sewers, such as the one illustrated, an over-fall of considerable length is required. In this particular example the length of the crest was approximately 100 ft.

An overflow of this type, if properly designed, should be absolutely automatic in its action and should require no particular attention. The only objection to its use is that gravel and sand washes along the bottom of the sewer and the larger stones brought down by storm water do not pass over the weir, but follow the line of the intercepting sewer, which, therefore, receives practically all of the grit and is more likely to be obstructed by accumulations than if the heavy material brought down by storm water could be discharged with the overflow. This also involves undue wear upon the invert of the intercepter.

Leaping Weirs.—The leaping weir type of overflow is suitable only for comparatively small sewers. As the name implies, it consists of a weir or crest over which the ordinary or dry weather flow falls into a connec-

must be provided, particularly in any project which involves pumping and treating the sewage collected by the intercepters; otherwise river water will at times enter the intercepter through the storm water overflows.

Contest for Neighborhood Center Plans.—A competition is being held by the City Club of Chicago for plans for a neighborhood center.

There is to be a preliminary competition open to everybody, which will close on Nov. 9, 1914, and a final open to not less than eight or more than sixteen selected by the jury in the preliminary, closing on Jan. 25, 1915. The plans are to be shown as the special feature of a Neighborhood Center exhibition and series of conferences next February.

The sum of $600 has been offered by one of the past presidents of the club for the author of the best plans.

A series of fortnightly meetings of the participants for discussion of the various phases of the problem will begin early in July at the club.

Not only American architects, city planners, landscape architects, and engineers will be invited to participate in the contest, but specialists in Europe as well.

Last year in connection with its housing exhibition, the City Club held a competition for plans for laying out a typical quarter section of land in the outskirts of Chicago. Thirty-eight sets of drawings were submitted, the majority of which are now being prepared for publication by the club. The volume will be ready the coming fall.

Securing Low Bids on Baltimore Sewers.

The following excerpts from the annual report of the Sewerage Commission of the city of Baltimore describe a method employed by the chief engineer, Mr. Calvin W. Hendrick, to secure low bids on sewer construction.

On account of the lack of information regarding underground structures, the very extensive ramifications of the sewerage system, the effort to interfere as little as possible with existing structures, and in order to give the contractor as much information as possible in making his bid, the engineering staff has made extensive investigations of the records of all the city departments and public service corporations; and, in addition, sunk numerous testpits. This has resulted in adding to the engineering cost, but it has been the means of effecting large savings to the city, by the contractors giving lower prices on account of having this information, and will result in the city having one of the most complete underground maps of any city in the country. It has also enabled the sewerage staff to do its work with the least possible interference with the structures of other city departments and public service corporations. This has involved, since the beginning of the work, the preparation of 10,500 drawings. Included in this number are 107 record plats which have been entirely completed, and 383 partially completed. These plats, which form the foundation of the contract drawings, show street, lot and curb lines, existing sewers and drains, tracks, pipes and other structures, with depths, sizes and materials.

In planning and executing such work as the Baltimore sewerage system, either one or two diametrically opposed policies may be followed:

First—The plans and preliminary investigations may be of the most general character, leaving it to the contractor to make detailed investigations and to assume all risks of unknown conditions. If this plan is followed the contractor's prices are necessarily high, in order to cover contingencies.

Second—The plans and preliminary studies may be made in great detail, showing, so far as possible, all underground conditions and obstructions, and leaving as few things as possible to be covered by the contractor's allowance for contingencies. This method results in the lowest prices from the contractors and has been adopted by the Sewerage Commission with the result that very wide competition and generally low prices have been secured on all of the work which has been done. The saving secured by the lower prices has been many times greater than the additional cost of preparing the plans along the above lines.

Railway Ties for India from Pacific Coast.—The Indian Railway Board recently placed orders for two shipments of railway ties from the Pacific Coast. One is of creosoted Oregon pine at a cost of $1.44 per tie, and the other of uncreosoted California redwood at $1.20 per tie. The Indian government requires between 500,000 and 1,200,000 ties a year for the state railways of the country. Formerly Australian jarrah has been used largely, but the cost of that material has risen to $2.80 per tie, and recent experiments with Pacific Coast timbers have proved satisfactory.

The steel work for the "Tower of Jewels" of the Panama-Pacific International Exposition has been completed. The placing of the highest steel column makes the top 435 feet above the ground. More than 1,500 tons of steel have been used in the frame and 1,600,000 board feet of lumber will be used in completing the structure.

A big floating dock is being constructed at Odessa which is said to be the largest in the world. It will cost about $2,000,000 and will be capable of carrying a vessel of 40,000 tons.

ROADS AND STREETS

Width and Arrangement of Residential Streets in a Small English Town.

Lytham, England, is a coast town of about 10,000 population, a summer resort relying more on its wealth of foliage, pleasant surroundings and residential qualifications than upon the amusements which form the main attractions for the majority of holiday seekers. In a paper before the Institution of Municipal and County Engineers which was abstracted in the London Surveyor, Arthur J. Price, city engineer of Lytham, states that efforts have been made to preserve the sylvan aspect of the town by planting trees on many of the new streets and protecting those which now line the chief shopping and residential streets.

The majority of the streets are paved with tar macadam. On the principal streets granite is used, being more durable and less dusty and slippery than limestone. In the purely residential streets limestone is used, this material being very suitable for light traffic, as it wears very smoothly, and from the hygienic and economic point of view is almost an ideal road material when properly mixed and laid.

Much of the success of the tar-macadam work is due to the excellent quality of the tar supplied from the municipal gasworks.

The tar occasionally varies considerably, according to the quality of coal used and the amount of light oils extracted, and it is necessary for the road foreman and man in charge of the boiling to exercise great care in the boiling so as to obtain tar of the proper consistency, and suitable for the situation in which it is to be laid.

The position of the street with regard to the sun and wind, the extent and nature of the traffic, the difference of temperature in winter and summer, all make it difficult to adopt a universal standard for tar. The personal factor cannot altogether be eliminated, for the man in charge of the tar boiling will have to make the tar harder—with more pitch in its composition—for a road where it is exposed all day to the sun and is subjected to a heavy traffic. In a back street, where neither sun nor much traffic finds its way, less of the lighter oils should be distilled or evaporated, for more "life" is required in the tar in such a position, to avoid brittleness and subsequent disintegration of the road, which is the chief danger in such a position.

Not more than half a dozen bad consignments of tar during the past 12 years have been received and these were due to the inferior quality of the coal used, or too great a demand for tar having led to the tar tank getting too low, and water being supplied instead of tar. It was the latter cause chiefly that in 1912 led to a failure of one short length of road which had been tar-painted, though a period of very wet weather was also, to some extent, responsible.

A small drying hearth is used for drying stone in wet weather, but if possible the stone is dried naturally, as this is not only more economical, but avoids any danger of the stone being overheated and converting the tar into pitch.

In tar painting, a tar with a larger proportion of the lighter oil left in it than for tar macadam work is necessary. When tar-painting a new road we use an ordinary 160-gal. tar boiler, which is convertible into a sprinkler with gas piping perforated to form a spray; but for repairs we chiefly use a small 80-gal. boiler fitted with a pump and movable spray.

In 1903 the principal street was paved with 4-in. Jarrah wood blocks on a 6-in. concrete foundation. The blocks were laid with a close joint directly on the concrete foundation, after being dipped into a mixture of pitch and tar, and were grouted with tar. Practically, this street has needed no repairs during the past ten years, and the blocks are in so good a condition that they will probably last another ten years.

The sidewalks in the main street are laid with concrete flagging, and portions of the other main streets with natural stone flagging. The residential streets are chiefly tar-asphalt, and country roads have footpaths of gravel or fine clinker. Some very interesting cobble-paved sidewalks are to be seen in the streets leading from the beach; those in Bath St. have the date at which they were laid—1831—with designs of a ship, compass, anchor, rose, etc., picked out in white pebbles, and are in a very fine state of preservation.

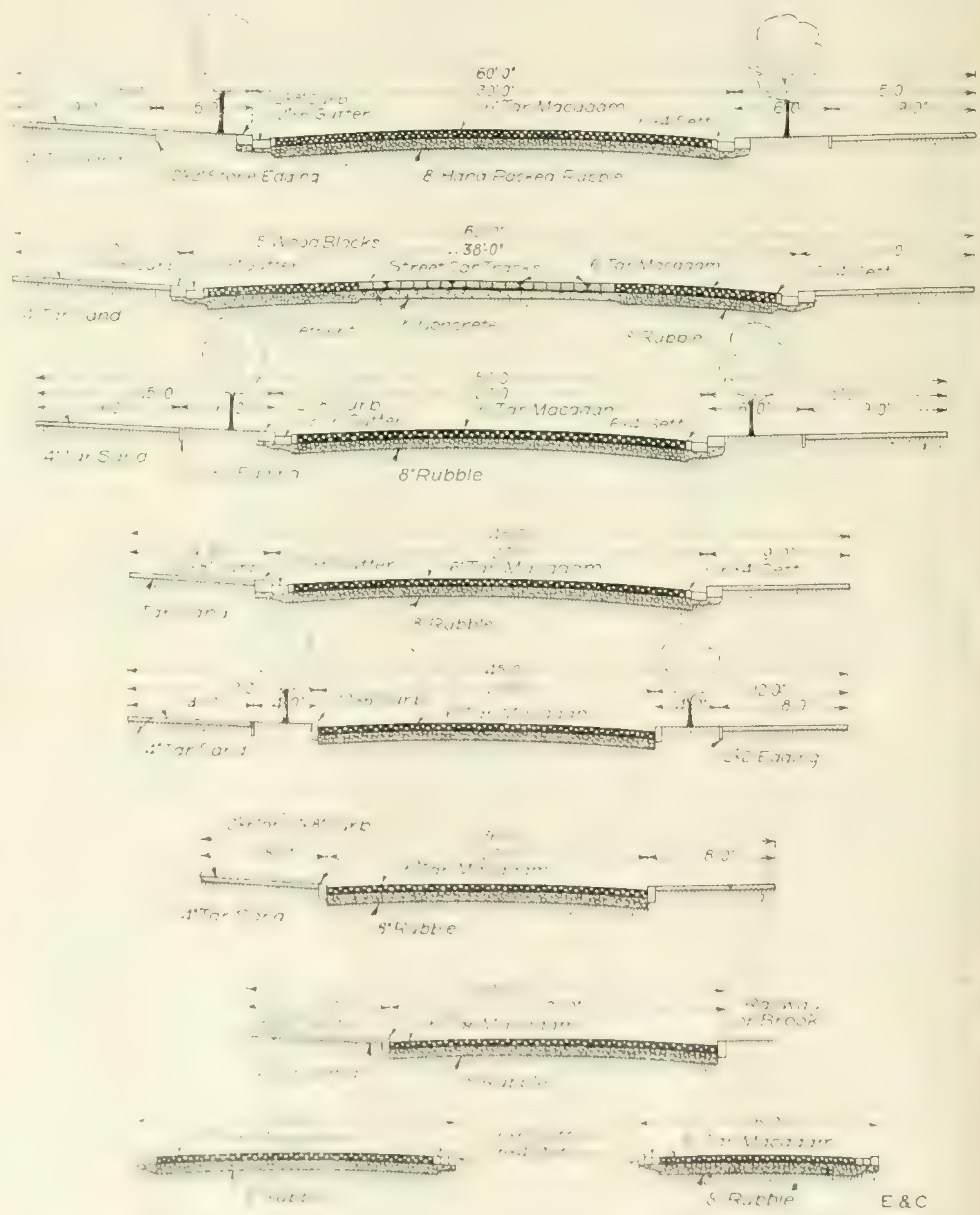


Fig. 1. Cross Sections of Typical Residence Streets.

The width and lay-out of the streets are shown in Fig. 1. Widths vary from 60 ft. to 30 ft. for front streets, with 20-ft. and 15-ft. back streets.

It will be noted that where the streets are 45 ft. or greater width the footpaths have a 4-ft. or 5-ft. wide plot set apart for the plantation of trees. These trees have grown well in most situations, and are much appreciated by the residents in the avenues where they are planted.

Methods of Sampling Materials of Construction Used by the New York Highway Commission.

All materials used in the construction of state roads in New York must be tested and accepted by the Bureau of Tests, State Highway Commission, Albany, N. Y. Instruction for sampling and forwarding test samples to the bureau are specific and inclusive. Samples are ordinarily taken by an employee of the bureau. If, however, the material has not been sampled by a bureau employee the engineer in charge of construction is notified and he takes the samples. No material is used until the construction engineer is officially notified of its acceptance by the bureau. If during construction an inspector notes a variation in quality of materials used, additional samples are taken.

In unloading bituminous material the condition of the shipment shall be noted. All barrels received shall be consecutively numbered, beginning with 1; the number shall be painted on the barrel in large white figures, and the barrel number and gallons contained shall be entered in a proper record book as the barrels are emptied into the tank. All barrels destroyed must be so recorded.

An officially sealed car has both doors sealed with a numbered lead seal and wire. A card stating that the car has been officially sealed by an inspector of the bureau will be tacked to the sides of the car near the seal. When the car has been opened by the engineer these forms and seals should be removed from both sides of the car.

When an officially sealed car arrives with the seal broken or in a damaged condition, and the engineer has reason to believe that the material therein is not the same or in the same condition as previously sampled, or if for any reason he believes that the material received is not satisfactory, he should promptly submit samples for test.

SAMPLING.

Samples of material will be taken by a duly authorized employee of the department, in its place of occurrence or manufacture or delivery by carrier. These samples must be taken from different parts of the lot of material to be tested, so as to be fairly representative, and must be unmixed with foreign substances and placed in clean and safe receptacles; and they must conform in all respects to the requirements given under the special headings. They must be carefully and securely packed, enclosing notification slip properly protected from wear and injury, and sent by express "collect" to the bureau, a postal card notice being mailed at the same time. Envelopes, scoops, cans, thermometers, etc., for use in taking the samples are provided.

In the case of materials sampled at place of manufacture, check samples may be required, these are to be taken and treated the same as ordinary samples, except that the packages must be marked "Check Samples," and the use of the material need not be prohibited pending the results of the check tests.

Sand and Gravel.—The character of the supply—whether from stream bed, bank, crusher bins, etc.,—is to be stated; also the use for which it is intended—whether for concrete foundations or other structures, binder for water-bound macadam, filler or wearing carpet or blotter for bituminous macadam, or for aggregate in water-bound or bituminous macadam, etc.

Material which will all pass through a ¼-in. screen will be considered sand. Each sample of sand or screenings shall be ¾ cu. ft. in volume; of gravel 1½ cu. ft. A small sample shall be taken from each test sample sent, and be kept on the contract as a measure of the quality of material. Each sample is to be shipped in a tight box or in a clean, closely woven bag from which there will be no leakage; the usual identification slip is to be enclosed. In numbering samples, sand and gravel are to be treated as one material, not as two.

Notification of acceptance or rejection may be expected to arrive at the division office 20 days after submission of the samples and data, providing the need of a retest does not cause delay.

Cement.—One sample is to be taken from at least every 10 bbls. or every 40 bags, care being taken to properly distribute the sampling over the lot. Each sample shall be not less than twenty-seven cubic inches in volume or enough to fill a 3-in. cube. Whenever possible, samples should be forwarded in envelopes furnished by the commission for that purpose, the envelopes being filled to the line marked thereon.

The individual samples are not to be numbered, but each group or lot of these samples representing a single boat load or car load is to be given a lot number, and these lot numbers are to run consecutively. Not more than one boat load or car load is to be represented by one lot number.

Receipt of notification of acceptance or rejection of cement sampled at destination may be expected to arrive at the division engineer's office 20 days after the submission of the samples and data. If cement is held for 28-day tests the division engineer will be notified accordingly.

Concrete.—The concrete on each highway must be sampled for testing, the samples being taken at random from the batches used and being moulded at the place and time of mixing. The work need not be delayed pending the results of the tests.

Each sample shall be a pair of cubes measuring 6 ins. on the edge or of cylinders 8 ins. in diameter and 16 ins. long; the sample is to be made in such manner as to fairly represent the concrete going into the structure. At least one sample is to be taken, and as many more as seem to be required by changes in the character of any ingredient or by any other consideration.

In concrete pavement work (whether foundation or top course) one pair of cubes or cylinders should be sent for every 500 cu. yds. Not less than two pairs are to be sent, however small the pavement. The sample must remain in the mold two days, then be buried in clean sand to age under the same conditions as the material in the structure. On the twenty-first day the sample shall be taken out and shipped. Each sample is to have its number painted on each piece, and is to be shipped in a box, properly protected from breakage and surface chipping, accompanied by the usual included identification slip and the postal notification. Especially must the class of concreate, the purpose for which it is used (kind of structure and portion), and the date and time of day when sample was mixed, be stated.

Bituminous Material.—When material is shipped in barrels one sample is to be taken for every 20 or 25 barrels, the sampling being properly distributed over the lot.

When material is shipped in tank cars one sample is to be taken for every 2,000 or 2,500 gallons, the samples being taken from equally distributed levels in the car.

When mineral bitumen is shipped in loose bulk, one sample is to be taken for every 5 or 6 tons, the samples being taken from different levels and different locations in the lot and never from the surface of the material. Each sample shall be not less than fourteen cubic inches in volume, which volume is slightly less than one-half pint or about the size of a one pound paint can.

It should be remembered that the bituminous material will flow at summer temperature or thereabouts, and consequently great care should be used in sealing cans and doing up packages. Whenever possible, samples should be forwarded in the cans furnished by the commission for the purpose.

The individual samples are not to be numbered, but each group or lot representing a single boat load or car load is to be given a lot number, and these lot numbers are to run consecutively; not more than one boat load or car load of material is to be represented by one lot number

In order to check the weighing and marking of bituminous material shipped in barrels, one unopened barrel out of every carload of approximately 65 bbls., or a proportionate number of barrels for each boat load, is to be selected at random and weighed. The gross weight found, and the gross weight marked on the barrel, are to be entered on monthly bituminous material reports or the information may be recorded elsewhere and submitted to the bureau of tests. Any noticeable difference between the gallonage marked on a barrel and the gallonage found therein, must be reported.

The unit of measure for bituminous material is the gallon measured at the temperature of 60° F. If the volume of material is measured when hot, allowance should be made for expansion according to the following table, which will apply approximately to all of the different classes of bituminous material at present used on the state highways:

Increase in volume of various classes of bituminous material when heated from 60° F. to 400° F. is approximately 12 per cent; to 350° F. is approximately 10 per cent; to 300° F. is approximately 8 per cent; to 250° F. is approximately 6 per cent; to 200° F. is approximately 4 per cent; to 150° F. is approximately 2 per cent.

Stone.—Rotten or partially disintegrated stone, or weathered specimens from the surface of a quarry or ledge, are not to be submitted. Samples of quarry or ledge stone must be representative of the sound, fresh, interior stone of the ledge or quarry. Such samples may be secured either by blasting or by breaking up with the sledge. If all material is of the same variety, texture, etc., one sample will suffice. If, however, there are different varieties, separate samples are to be taken of each and report made as to the extent, giving details as to location and position for use. All field stone, whether in walls, piles, or scattered over the ground, which might be used, must be examined and a representative sample taken. When two or more varieties of great difference in quality or texture are observed to exist, separate samples are to be taken of each, and report made as to the percentage of each kind, the amount of small stone which might run through the crusher without action, and the percentage of disintegrated or badly weathered rock present.

In taking samples from the output of crushers, fifteen pounds of crushed material not smaller than 1½ in. in size shall be taken, and also one piece at least 3x4x5 inches shall be procured from the source of supply.

Each sample shall weigh not less than 25 lbs. nor more than 35 lbs. If the entire sample submitted is a single piece of stone it should be remembered that a piece about the size of a man's head will weigh 25 or 30 lbs. While not less than 25 lbs. are absolutely necessary in each sample, care should be taken to see that the samples do not weigh over 35 lbs. One piece of each sample shall be at least 3x4x5 inches.

Paving Brick.—A sufficient number of samples in every case is to be taken to insure the use of brick of proper quality, but it should also be borne in mind that the charges for transportation and testing of brick are high, and only the smallest number of samples necessary for this purpose should be submitted. At least one sample is to be taken from every 200,000 brick or less. Each sample shall consist of 30 bricks.

If in a shipment or several shipments of the same make and kind of brick there appear to be different classes of brick—such as brick of different degrees of burning, for example—a full sample of each class is to be taken. Each brick selected for the sample is to be free from cracks or other defects which would prevent passing inspection at the road, for the sample must represent bricks which will not be culled out. Especially is it forbidden that any person financially interested in the manufacture or use of brick be present when samples are taken.

Each sample (consisting of 30 bricks) shall receive a number, the numbers to run consecutively for each road. The sample shall be shipped in wooden boxes, not more than 10 or 12 bricks being put in one box on account of weight and strength of package.

Notification of acceptance or rejection of brick sampled at destination may be expected to arrive at the division engineer's office nine days after submission of samples and data, providing the need of a retest does not cause delay.

Asphalt Block.—A sufficient number of samples in every case is to be taken to insure the use of block of proper quality, but it should also be borne in mind that transportation and testing costs are high, and only the smallest number of samples necessary should be submitted. At least one sample is to be taken from every 100,000 blocks or less. Each sample shall consist of two blocks.

If in a shipment or several shipments of the same make and kind of block there appear to be different classes of block, a full sample of each class is to be taken. Each block selected for the sample is to be free from every defect that would prevent its passing inspection at the road, for the sample must represent blocks which will not be culled out. Each sample (consisting of two blocks) shall receive a number, the numbers to run consecutively for each road. The sample shall be shipped in a wooden box, with usual identification card and postal notice.

Notification of acceptance or rejection of block sampled at destination may be expected to reach the division engineer's office fourteen days after submission of samples and data, providing the need of a retest does not cause delay.

ACCEPTANCE.

Upon completion of the testing of any set of samples the division engineer is notified of the acceptance or rejection of the material, and transmits the statement to the engineer in charge of the contract.

Making Cuts in Pavements Having a Concrete Base by Means of a Hammer Drill.

(Staff Article.)

Cutting through a concrete pavement base is slow and expensive work when accomplished by hand picking. On an asphalt pavement repair job in progress in Salt Lake City, Utah, the contractor, J. P. Moran, has been using a hammer drill for this purpose. The economy of the use of a tool of this kind when working along a street car track, where a picking gang is subject to constant interruption by passing street cars, is apparent.

The method used in making these cuts, Fig. 1, was as follows: A line marked one foot from the car tracks was channeled through the asphalt with a Sullivan DC-19 drill operated by one man, a special fan-shaped channeling bit being used. When a sufficient distance had been channeled a gadding bit was substituted and the asphalt removed from the concrete base. The exposed concrete base was broken up with the same bit, holding the machine in a nearly vertical position, Fig. 2. The concrete was broken into pieces from 4 to 8 ins. square, which were readily thrown out by hand or removed with a shovel.

The drill broke up an average of 1 sq. ft. of asphalt and concrete in 2½ minutes. By the old method three men with sledges and picks averaged 1 sq. ft. in 6½ minutes, or 19½ minutes per man per square foot removed.

The drill was operated by steam obtained from a compressor mounted so that it could be dragged along the street and supplied with steam by a portable boiler.

A rough estimate of the cost of similar

Fig. 1. Method of Channeling Asphalt Pavement.

work using slightly different equipment and a comparison with hand labor costs is given below:

MACHINE WORK.

[illegible] blacksmith tools, etc., about...	$2,000.00
[illegible]	[illegible]
Depreciation, 15 per cent.........	300.00
Total plant expense per year..	$420.00
Operating expense, 175 days per year, per [illegible]	2.40
[illegible]	3.50
Two drill operators [illegible]	[illegible]
[illegible] at 23 [illegible]	4.60
Oil, waste, etc.	.50
Daily cost of operation	$16.00
Progress per day of 8 hours, 384 sq. ft.	
Cost per square foot..............	$0.0416

HAND WORK COST.

Six laborers at $2.25 per day.....	13.50	
Progress per day, 8 hours, 144 sq. ft.		
Cost per square foot.............		.0937
Saving in cost per square foot, by use of portable air compressor and hammer drills		$0.0521

The information and photographs used in preparing this article were furnished by J. H. Henning of Salt Lake City, Utah.

Construction Methods and Costs and Service Records for Concrete Roads in Ohio.

Since July, 1911, 39.63 miles of concrete road have been built under the supervision of the Ohio State Highway Department and 25.06 miles are now under contract, making a total state mileage of 69.64 miles. Twenty-nine sections of road varying from 0.7 to 2.7 miles in length were constructed in different portions of the state. The tables and data given here supplement the description of this work contained in the issue of ENGINEERING AND CONTRACTING for March 4, 1914.

Materials.—Materials for concrete aggregate are well distributed throughout the state. In character they vary from limestone in the western half of the state to furnace slag in the eastern portion, glacial gravel being abundant throughout the northern and western sections. It may be said that materials exist within reasonable hauling distance of nearly every mile of road in the state. Some 30 commercial sand and gravel working plants and 150 limestone quarries are in active operation. In addition there are local sand and gravel deposits and small quarries.

All materials used were tested in the state laboratory. A special effort was made to secure good sand, the specifications requiring that the tensile strength of mortar test briquettes made therefrom be not less than that of briquettes in which standard Ottawa sand and the same cement were used.

TYPES OF CONSTRUCTION.

The type of construction generally used was a single-course plain concrete pavement laid on the natural earth foundation (with the exception noted in Table I), and surface treated with bitumen.

Cross Sections.—Three types of cross section were used, no set standard having been adopted, viz., (a) uniform thickness with equal crown on surface and subgrade, (b) greater thickness at center with slight crown in subgrade, and (c) a flat subgrade with greater thickness at center. Figure 1 shows graphically the difference between these sections. The standard crown was ¼ in. to 1 ft., which has been found sufficient. At the present time, the type of cross section appears to bear little relation to the permanency of the pavement, as is shown by Table II, which was prepared from the inspection report of H. D. Bruning and Harwood Lersch, division engineers of the state department.

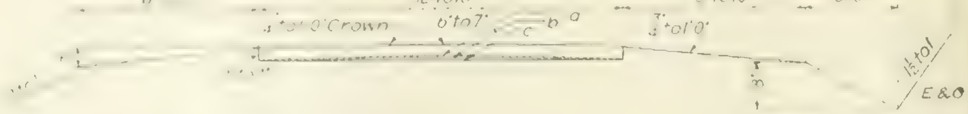


Fig. 1. Cross Section of Typical Concrete Road Built by the Ohio Highway Commission.

METHODS OF CONSTRUCTION.

Concrete Proportions.—A 1:1½:3 mixture of concrete was generally used, the proportion of mortar (1:1½) being maintained uniformly but a slight variation in the aggregate allowed to secure a 10 per cent excess of mortar. It has been the experience of the department that less care in mixing and placing is required with this mixture than when a leaner one is used.

Mixing Concrete.—Aggregate was usually measured in gaged wheelbarrows although in several cases boxes of 1 cu. ft. capacity were used. Batch mixers were employed. Some trouble was experienced in counteracting the tendency of contractors to use too wet a mixture. On heavy grades a dryer mixture was used avoiding the wavy surface which results from the creeping of sloppy concrete.

Laying Concrete Surface.—The wearing surface was wood-floated to compact and seal the surface. No depression greater than ½ in. below a 10-ft. straightedge applied parallel to the surface was allowed. Two methods of curing were used: (1) the surface was covered with canvas within 1 hr. after floating and left in place 12 hrs., and (2) the surface was sprinkled for 5 days. In both

Fig. 2. Method of Breaking Concrete Base.

cases a sand, straw, or dirt covering was used to assist in retaining the moisture.

Joints.—Joints were generally spaced 30 feet. Failure usually occurred at poured joints. Tarred felt joints have not failed so often. Transverse cracks appear in numbers except in rare cases, when the joint spacing exceeds 30 feet. To date of inspection cracks do not show appreciable wear.

A monolithic, 12-ft. pavement 6,946 ft. long, was completed in Sept., 1912, in Huron Co. One-half of the pavement is 6 ins. thick on a clay foundation, the remainder 5 ins., the minimum thickness of cut being laid on an old sandstone macadam foundation. Limestone and Canadian sand were used in the concrete allowing 6 sacks of cement per cubic yard of concrete. An inspection in January, 1914, showed 85 transverse cracks ranging from 9 to 159 ft. in spacing. The general conditions as regards number and spacing of cracks are the same in both halves of the road the cracks being irregular and several badly worn.

COST.

The average cost of the concrete pavements completed to date is $1.128 per square yard, as set forth in Table I. This cost is the contract price paid and does not include the cost and expense of engineering and inspection, but does include the cost of grading and finishing of shoulders and ditches and a small amount of accessories. Deducting from this average cost the average cost of surface treatment, which is 9½ cts. per square yard, there remains $1.033 as the average cost per square yard of plain concrete including grading, shoulders and accessories. The average cost of grading, finishing of shoulders and ditches and accessories is $0.167 per square yard of paved surface. Deducting this from $1.033 we have 86.6 cts. as the average cost per square yard of the plain pavement alone exclusive of engineering and inspection costs.

Rubber Paving in London.—Permission has been granted by the City of London Corporation to the acting agent of the Federated Malay states to lay an experimental strip of rubber paving in Cannon St.

TABLE I.—TABULAR DESCRIPTION OF OHIO CONCRETE ROAD CONSTRUCTION.

Number.	County.	Name of road.	Road completed.	Contract price.	Length, feet.	Length, miles.	Width of pavement	Depth, side, ins.	Depth, center, ins.	Square yards.	Cost per mile.	Cost, per sq. yd.	Subsoil.	Plain or surface treated.	Materials. Cement.	Materials. Fine aggregate.	Materials. Coarse aggregate	Materials. Proportions.
1	Ashtabula	Andover	Oct., 1912	$12,285.00	8,523	1.61	16	6 4	7 4	15,152	$7,630	$0.811	Clay	Surface treated	Universal	Lake sand	Washed river gravel	1:1½:3½
2	Ashtabula	Geneva	Aug., 1913	9,746.00	5,280	1.00	16	6	7	9,386	9,746	1.038	Sandy loam	Surface treated	Universal	Lake sand	Silica pebbles	1:1½:3½
3	Erie	Lake Shore	Sept., 1913	7,099.00	5,300	1.00	12	6	6	7,067	7,039	1.004	Clay	Surface treated	Medusa	Lake sand	Crushed limestone	1:1½:3½
4	Franklin	Sec. 3, Winchester	Nov., 1913	21,000.00	11,320	2.14	14	6	7	17,609	9,813	1.192	Clay	Surface treated	Universal	Unscreened river gravel		1:1½:3½
5	Harrison	Piedmont	Nov., 1912	10,432.00	6,600	1.25	14	6	6	10,070	8,346	1.036	Clay & gravel	Plain	Lehigh	Bank sand	Screened gravel	1:2½:5
6	Harrison	Scio	Nov., 1913	23,289.00	13,600	2.54	10	7	7	15,111	9,169	1.541	Clay	Surface treated	Universal	Wash. riv. sand	Slag and washed river gravel	1:1½:3½
7	Holmes	Sec. 3, Mlsbrg & Berlin	Oct., 1913	5,750.00	1,795	0.34	16	6	7	3,192	16,912	1.800	Clay & shale	Surface treated	Universal	Bank sand	Crushed limestone	1:1½:3½
8	Lake	Madison Southern	June, 1913	11,970.00	7,465	1.41	12	6	6	9,953	8,489	1.202	Clay	Surface treated	Lehigh	Lake sand	Crushed limestone	1:1½:3½
9	Lake	Weed's Corner Extension	Aug., 1913	25,010.00	11,686	2.21	12	2 4	2 4	15,581	11,317	1.605	Clay	Surface treated	Crescent	Local bank Unscreened 2d quality gravel	Crushed boulders	1:1½:3 1:2½:5
10	Lucas	Monroe Turnpike	Dec., 1913	31,92[illegible].00	10,807	2.05	16	6	7	19,189	15,572	1.661	Wet clay	Surface treated	*	Canada sand	Crushed slag	1:1½:3½
11	Mahoning	Youngstown & New Bed.	Aug., 1913	15,645.00	6,600	1.25	14	6	6	10,243	12,516	1.527	Clay	Surface treated	New Castle	Wash. riv. sand	Washed river gravel	1:1½:3½
12	Mercer	Reservoir	Jan., 1913	12,880.42	8,450	1.61	14	6	7	13,154	8,000	0.980	Clay	Plain	Peninsular	Washed sand	Washed gravel	1:1½:3½
13	Miami	Sec. 1, Piqua-Troy	Dec., 1912	16,237.00	9,800	1.86	16	6	7	17,422	8,729	0.932	Gravel	Surface treated	Universal	Washed unscreened gravel		1:1½:3½
14	Miami	Sec. 3, Piqua-Troy	Dec., 1913	36,006.00	21,909	4.15	16	6	7	38,950	8,676	0.924	Gravel	Plain	Superior	Washed unscreened gravel		1:1½:3½
15	Muskingum	Dresden Tramway	Nov., 1912	7,140.00	5,650	1.07	16	6	6	9,867	7,149	0.754	Gravel & sand	Surface treated	Universal	Unscreened bank gravel		1:2:4
16	Pickaway	Circleville & Washington	May, 1913	11,961.18	5,121	0.97	20	7	7	11,387	12,331	1.050	Clay & gravel	Surface treated	Ironton & Alpha	Wash. bk. sand	Washed gravel	1:1½:3½
17	Pickaway	Circleville & London	Sept., 1913	11,014.12	5,280	1.00	16	6	6	9,387	11,014	1.173	Gravel	Surface treated	Alpha	Wash. bk. sand	Crushed boulders	1:1½:3
18	Pickaway	Cincinnati & Zanesville	Dec., 1913	13,977.00	5,600	1.06	16	7	7	9,956	13,183	1.103	Gravel	Plain	Alpha	Wash. bk. sand	Washed gravel	1:1½:3
19	Pike	Columbus & Portsmouth	July, 1911	924.24	1,280	0.24	12	6	6	1,707	3,851	0.541	Sandy loam	Plain	Superior	Unscreened bank gravel		1:2½:5
20	Pike	Columbus & Ports. Ex.	Oct., 1912	1,167.06	920	0.17	12	6	6	1,227	8,629	1.195	Sandy loam	Surface treated	Universal	Unscreened bank gravel		1:1½:3½
21	Putman	Kalida Lima	Nov., 1913	5,261.00	2,175	0.41	16 20	6	7	4,523	12,831	1.163	Loam	Plain	Diamond	Canada sand	Crushed limestone	1:1½:3
22	Putman	Rimer	Aug., 1913	1,486.00	1,020	0.20	14	6	7	1,587	7,430	0.936	Clay	Plain	....	Canada sand	Crushed limestone	1:1½:3½
23	Ross	Frankfort	Nov., 1912	10,925.00	6,969	1.31	16	5	7	12,389	8,339	0.881	Gravel	Surface treated	Superior	Unscreened river gravel		1:1½:3½
24	Ross	Chillicothe & Jackson	Jan., 1913	11,518.62	5,280	1.00	16	5	7	10,137	11,518	1.136	Sandy loam	Surface treated	Alpha	Wash. bk. sand	Crushed boulders	1:1½:3½
25	Ross	Columbus	Nov., 1913	9,671.58	5,280	1.00	16	6	7	9,386	9,671	1.030	Gravel	Surface treated	Superior	Unscreened river gravel		1:1½:3
26	Summit	Ghent	Nov., 1913	17,889.89	2,516	2.28	14	6	6	19,516	7,516	0.916	Clay	Surface treated	Universal	Creek sand	Crushed slag	1:1½:3½
27	Trumbull	Hartford Northern	Nov., 1913	18,400.00	14,200	2.69	12	6	6	18,933	6,840	0.972	Clay	Surface treated	Universal	Silica sand	Crushed slag	1:1½:3½
28	Washington	New Matamoras	Dec., 1912	12,125.00	5,280	1.00	16	6	7	9,387	12,125	1.292	Sand & gravel	Surface treated	Alpha	Wash. riv. sand	Washed river gravel	1:1½:3½
29	Franklin	Columbus & Lancaster	Sept., 1913	7,332.75	3,750	0.71	14	6	7	5,833	10,327	1.258	Sandy loam	Surface treated	Atlas	Unscreened river gravel		1:1½:3

Totals: Miles, 39.63; square yards, 337,291; contract prices, $380,666.37.

Average cost, exclusive of engineering, but including grading: Per mile, $9,606; per square yard, $1.128.

*Universal and Wyandotte.

The Selection of Asphaltic Materials for Road Construction.

The fundamental requirements of a bitumen satisfactory for road use are simple. In digesting the mass of detail, however, that complicates the selection of the most suitable binder the engineer is apt to lose sight of these fundamentals unless they have been well impressed on his mind.

A paper discussing the fundamental requirements of asphaltic materials by Alfred A. Berkowitz of the Sheffield Scientific School is given here. This paper was a prize essay in a recent competition held by the Barber Asphalt Company.

THE NEED OF STANDARDS OF COMPARISON.

To specify an asphalt by its chemical or its physical properties alone is not enough. The chemical properties of two asphalts might be the same, while physically they differed markedly. Moreover, a relationship exists among certain properties, and there is danger of assigning limitations to the several properties covered in a given specification without considering the effect of each limitation upon the other properties included. For instance, the higher the ductility required of an asphalt the more susceptible it is to change in temperature. Furthermore, the engineer cannot be governed entirely by theoretical considerations, but must keep in mind the ability of the manufacturers to meet his requirements. He should avoid making requirements that are superfluous, inconsistent, unnecessarily increase the cost, or sometimes even impossible to carry out. Numerous instances can be cited where the combinations of properties required have not existed in any commercial material. These impossible requirements usually result when the engineer selects clauses here and there from several other specifications which he knows to have given satisfaction. With a view toward remedying this evil, the American Society of Civil Engineers, the American Society for Testing Materials, etc., have prepared standard specifications for asphaltic materials in various kinds of road construction. These specifications are very well drawn up and serve their purpose admirably, provided they are correctly and intelligently used. It is in this regard that experience can help us. In cases where these specifications have been used, nearly all the failures have been due to the fact that the specifications were followed blindly, without an understanding of their value and their interpretation; merely, as it were, substituting in a formula without considering the units involved. A knowledge of the value and interpretation of each test must be possessed by every engineer who makes or who uses specifications before he can hope to obtain success in the employment of these very desirable materials. This is, in fact, the chief lesson of experience. Then, knowing these facts, and keeping in mind the lessons of service tests of similar materials under similar conditions, the engineer can employ specifications with the assurance that his work will be a success.

INTERPRETATION OF TESTS.

The value and interpretation of the more important tests required of asphaltic materials will be discussed. The method of actually making these tests will not be taken up, since these can better be obtained from the aforementioned standard specifications.

The individual tests usually specified may be divided into three groups: (1) They may directly indicate the suitability of the material specified for a given use; (2) they may serve as a means of identifying the material or its source, and (3) they may serve to control uniformity in the preparation or manufacture of a material.

Viscosity.—The most important test belonging to the first group is the determination of the consistency or viscosity. The consistency or viscosity of a material is its degree of firmness as determined by measuring its internal friction. Requiring this test in the specifications is of value since a physical test can generally be better understood than any corresponding chemical one. This test, however,

TABLE II. TABULAR DESCRIPTION OF OHIO CONCRETE ROAD CONSTRUCTION—REPORTED FROM EXAMINATION MADE IN JANUARY 1914

Number.	Surface treatment	Joint filler	Joints, spacing, ft.	Kind of traffic	Max. grade, per cent	Method of curing	Defects: Longitudinal cracks	Defects: Transverse cracks	Defects: Holes, No.	Defects: Holes, depth, in.	Defects: Holes, area, sq. ft.	Condition of joints	Condition of surface treatment	Remarks
1	Tarvia and silica pea gravel	Poured tar	[illegible]	Heavy	4.5	Sawdust and water	*	*	1	....	1	Slightly worn	10% peeled	4-in. concrete on slag macadam foundation
2	Dollarway silica pea gravel	Elastite	30	Medium	1.7	Daily watering	*	1	0	.....	.	Three worn	2 blocks peeled slightly, remainder good.	
3	Carbovia and pea gravel	Poured tar	[illegible]	Medium	[illegible]	Daily watering	0	0	0	.....	..	O. K.	1% peeled in patches	
4	Tarvia and pea gravel	[illegible]	[illegible]	Medium	[illegible]	Daily watering	0	0	0	..		Good.		
5		Poured tar	[illegible]	Light	[illegible]	Straw and water..	0	7	6	1-4	355	Several badly worn.		
6	Tarvia and river sand	Elastite	30	Very light	5.0	Clay and water	0	0	1	3	1		7% peeled, dirty surface	Wavy surface.
7	Tarvia and bank sand	Poured tar	30	Medium	[illegible]	Sand and water	0	0	0	.....	.	O. K.		Rough surface
8	Tarvia and silica pea gravel	Poured tar	30	Heavy	[illegible]	Clay and water.	0	0	0	..	..	Uneven.	81 small patches peeled, remainder good.	
9	Tarvia and screenings from boulders	Elastite	[illegible]	Medium	[illegible]	Daily watering	0	0	1	3½	7	2 badly worn.	½% peeled, remainder good.	2-course concrete.
10	Carbovia and sand	Tar paper	[illegible]	Medium	[illegible]	Daily watering	0	0	0	.	...			Artificial drainage.
11	Bitusia and sand	Poured tar and elastite	[illegible]	Heavy	[illegible]	Daily watering	[illegible]	0	0	.	...	5 badly worn.	No peeling, but greatly worn.	
12		Tar paper [illegible]	[illegible]	Medium	[illegible]	Daily watering	65	0	7	⅜-1¾	20	Slightly worn		Road located on reservoir bank.
13	[illegible] and sand	[illegible] joints	[illegible] to 90	Heavy	[illegible]	Daily watering	121	48		.....	.		Practically all peeled	Concrete worn considerably ¾ ft.
14		[illegible] joints	30 to 90	Heavy	[illegible]	Daily watering	32	132		......	.	Some wear; joints imperfectly placed.		One 90-ft. section has 4 transverse cracks.
15	[illegible]	Poured tar	30 to [illegible]	Medium	7.21	Sand and water	7	30	8	½-2	10	Several worn.	8% peeled, remainder badly worn.	Concrete disintegrating in several places
16	Asphaltic oil and sand	Poured asphalt	30	Very heavy	[illegible]	Daily watering	0	0	0	...	...	5 badly worn.	50% peeled	Has successfully withstood a 10-foot cover of water with strong current
17	Bituvia and pea gravel	Elastite	30	Medium	0.45	Straw and water..	0	0	0	....	...	Very good	½ section peeled.	
18		Elastite	20	Heavy	0.22	Daily watering ...	0	1	0	......	...	51 joints, grout slopped over.		Transverse crack on 60-foot section.
19		Poured tar	[illegible]	Medium	[illegible]	Straw and water	0	1	10	½-2½	24	6 badly worn.		Concrete reinforced.
20	Tarvia and fine gravel	Poured tar	30	Medium	[illegible]	Straw and water	0	1	0	......	...	O. K.	Very good	
21		Tar paper	30	Heavy	[illegible]	Water & sawdust	0	0	0	......	...			
22		Tar paper	30	Heavy	1.5	Daily watering	0	0	0	......	...			
23	Tarvia and unscreened gravel	Poured tar	30	Heavy	1.5	Straw and water	17	1	11	½-¾	30	Several badly worn.	[illegible]% peeled	
24	Tarvia and pea gravel	Poured tar	30	Medium	6.0	Straw and water	0	0	1	¾	½	Fair.	[illegible]0% peeled	Has successfully withstood a 10-foot cover of water with strong current.
25	Bituvia and pea gravel	Elastite	30	Medium	0.85	Daily watering	0	0	0	....	...	Very good	Shows signs of peeling.	
26	Tarvia and sand	Poured tar	30	Heavy	[illegible]	Daily watering..	0	0	1	3	¼	O. K.	200 sq. ft. peeled	
27	Bituvia and silica pebbles	Poured tar	30	Medium	5.0	Daily watering	0	0	0	......	...	O. K.	½% peeled in patches.	
28	Tarvia and coarse sand	Poured tar	30	Medium	0.83	Sand and water ..	0	0	0	......	...			
29	Tarvia and pea gravel	Elastite	30	Medium	1.35	Sand and water...	0	0	0	......	...		2% peeled.	

*Several.

can be of maximum value only when applied to a specific type of asphaltic material and when considered in connection with other tests, which by themselves may not directly indicate suitability. Thus for one type of bituminous concrete pavement the proper penetration at 25° C. for a California asphalt may be between 7 and 9 mm., while the proper penetration limits for a fluxed Bermudez asphalt, to be used in exactly the same type of pavement and under the same conditions, may be from 14 to 16 mm. Again, experience has shown that a harder asphalt cement is needed for a sheet asphalt pavement of sand, powdered limestone, and asphalt cement, than for a pavement containing crushed stone. The amount of traffic and the climate are also important factors to be considered in fixing penetration limits for any asphaltic material.

Volatilization.—Another test serving to indicate the suitability of a material for any special purpose is the volatilization test. A determination is made of the loss in weight of the material by volatilization, and of the consistency of the residue. It is of considerable value when applied to road oils. The loss in weight thus found is a fair indication of the loss by volatilization suffered by the material in the course of time when applied to the road, and also the character of the residue is similar to that eventually left in the road. The residue should be of a sticky nature. If the residue of these oils are not sticky and adhesive they will produce an undesirable surface in wet weather. A paraffin oil will produce a greasy residue, and is consequently worthless for road work, whereas an asphaltic oil may be successfully used for this purpose.

Identification Tests.—The more important tests serving as a means of identifying the source of a material or even the material itself, are the specific gravity, melting point, solubility, in carbon disulphid, and fixed carbon determinations. The specific gravity of an asphaltic material is one of the most important characteristics used to determine its identity. This is especially true if its specific gravity is considered in connection with the consistency of the material, and sometimes its solubility in carbon disulphide. Thus a bituminous material with a specific gravity of 0.99 and a penetration of 7 mm. at 25° C. is a blown product. Fluid consistency and a high specific gravity of 1.25 in a tar serves to identify it as a coal tar, and the identity is strengthened if its solubility in carbon disulphide is as low as 75 per cent. As applied to oil and oil products, the specific gravity roughly indicates the amount of heavy hydrocarbons which give body to the material. As a rule, paraffin oils have the lowest specific gravity and are of no value for road work, whereas asphaltic oils have the highest specific gravity and are most desirable for this purpose. In regard to solid bitumens as originally found, it must be remembered that the specific gravity will depend largely upon the per cent of mineral matter. Thus the specific gravity of Trinidad asphalt (containing 37 per cent of mineral matter) is 1.40, while Gilsonite, an extremely pure native bitumen, has specific gravity of 1.04.

The melting point of a bitumen is directly related to its hardness and brittleness, but the relations for all classes are not the same. The method of employing the material must also be considered in connection with its melting point. If the penetration method is used, the melting point should not be too high, because the highly heated material would be solidified by the cold stone before it had penetrated to any extent. In a bituminous concrete pavement, the melting point may be as high as climatic conditions will allow.

The determination of the solubility in carbon disulphide of an asphaltic material enables the amount and character of the contained bitumen to be ascertained. The percentage of bitumen thus obtained does not alone determine the value of an asphalt for paving purposes. Two asphalts might contain the same amounts of bitumen and yet possess entirely different powers of resistance to the destructive action of the elements. One might possess stability, the other not. The character of the bitumen does, however, indicate, by the amount of mineral matter present, whether an asphalt has been used. Likewise, an absence of mineral matter indicates that the material has been destructively distilled during its preparation.

The amount of fixed carbon in asphaltic material also serves to identify the source of a material. Thus high fixed carbon in most asphaltic cements produced from Mexican petroleums serves to differentiate them from asphalt cements of the same consistency produced from Californian petroleums. Consequently, if the amount of fixed carbon found in a California asphalt cement were as high as the amount usually present in Mexican asphalts (about 16 per cent), the indication would be that it has been injured by overheating.

Method of Manufacture Tests.—The tests that serve to control the preparation and manufacture of a bituminous material are the flash point, distillation, and the solubility in naphtha tests. The temperature at which the material flashes is an indication of the treatment to which it has been subjected. Crude oils have a low flash point, while more highly distilled products have correspondingly higher flash points. The flash point of a material must be considered in connection with its proposed use. If the material, especially a heavy road oil, is to be heated before application, the flash point should be high enough to prevent its ignition while being heated.

The distillation test is a valuable one, especially for tars. No tar containing water should be employed as a permanent binder.

The extent of the solubility in naphtha is an indication of the amount of body-forming hydro-carbons which give mechanical stability to the material. No oil containing less than 4 per cent of bitumen insouble in naphtha will prove of service other than as a dust preventative.

Service Tests.—To the valuable data that may be obtained in the manner described above must be added the lessons of experience and service tests of various asphaltic materials under various conditions and in different types of roads and pavements.

The need of alleviating the dust nuisance has involved the use of asphaltic oils for that purpose. Experience has shown that the oils used should show some degree of adhesiveness when rubbed between the fingers. Their residues after the evaporation test should be decidedly sticky. If oily, they will act more as lubricants than as dust binders.

It must be emphasized again that paraffin oils are worthless for this purpose. Engineers throughout the country are finding that the use of palliatives for dust prevention is but a makeshift at the most, and that the best practice is tending toward the construction of bituminous surfaces or carpet coats wherever any degree of satisfaction and permanency is desired.

CONDITIONS CONTROLLING USE.

Bituminous surfaces are used principally on macadam and gravel roads, on bituminous and cement concrete pavements, and to some extent on brick and stone pavements. The character of the traffic to which the road is to be subjected should be considered in the construction of a surface. Experience has shown that if the road is subjected to light motor and light team traffic only, with the motor vehicles predominating, an asphaltic oil of such viscosity that it requires heating to at least 250° F. before application forms a bituminous surface that withstands the traffic and thoroughly preserves the road for a period of time depending upon the quality of the material and workmanship and upon the quantity of traffic. The failure of many bituminous surfaces can be traced to improper preparation of the broken stone surface. The adhesion of the oils to the stone is a very essential point to consider. Most engineers are in favor of applying the bituminous material under pressure since the adhesion to the stone is much more satisfactory in this case. In order to get the best adhesion of asphaltic oils, the stone surface should be a little moist rather than extremely dry, but in either case, every particle of dust and dirt should be removed from the stone by sweeping. With proper construction and under normal traffic conditions, retreatment is necessary every one or two years. In choosing the bituminous material for a surface which will have to be renewed often, experience has shown that care must be taken to select a material which will set up sufficiently to allow the application of repeated layers without producing a plastic, easily moved blanket surface. Properly constructed, bituminous surfaces are quiet, practically dustless, and comfortable to use.

With the advent of the motor car, it was found that the water-bound macadam pavement did not stand the automobile traffic and that bituminous macadam could be laid to withstand this traffic at a cost equal to water-bound macadam, when the maintenance of the latter is considered. The bituminous material used may be either asphaltic cement or refined coal tar. The former is preferable because it possesses greater stability under atmospheric action. Here again, numerous instances can be found where unsuitable materials have been employed through the failure of the engineer to understand the physical and chemical properties of the different types of asphaltic materials, and the different methods of treating them. For example, many a good asphalt has been ruined by overheating. Again, asphalts of too little ductility have been chosen. The slight displacement of the mineral aggregate of a pavement due to the shocks of hoofs or wheels makes it necessary that the asphalt cementing material should yield and stretch a little and not break. On the other hand, the greater the ductility, the more susceptible is the asphaltic material to changes in temperature. Once more the knowledge of the relationship among the various properties of a material enables the engineer to intelligently set the limits in his specifications.

A series of experimental sections was constructed on a road subjected to mixed traffic of about 100 horse drawn vehicles and from 250 to 300 motor cars per day. These sections were constructed with asphalt, refined water gas and coal tars, and combinations of refined coal tars and asphalts, in the form of bituminous concrete pavements, using the same material for the cement and the seal coat. Upon examination after three years, it was found that for this class of traffic, or for horse drawn traffic exclusively, the seal coat should consist of an asphalt, as being more durable and economical than the other materials. Experience in another locality has shown that an excess of flux or of the volatile constituents in asphalt cements has been a cause of failure. Pavements constructed with such materials are wavy and distorted by heavy traffic.

SHEET ASPHALT.

One of the most widely used pavements is sheet asphalt. Trinidad asphalt was for many years the only available supply of such material, but in recent times many other sources have been disclosed. In addition to the native asphalts, like those obtained from Trinidad and Bermudez, asphalts are obtained by refining asphaltic oils. The results of experience and service tests in this kind of pavement alone furnish the engineer with a collection of valuable data. The selection of both the aggregate and the asphalt cement requires a careful understanding of the climate, traffic and other conditions. Proper drainage, in this, as in all other pavements, is a primary requisite. Furthermore, certain bitumens require more skill in handling than others. Again, some require particular fluxing agents and result in failures when improper ones are used. The main requisite of a good sheet asphalt pavement is inherent stability. This can only be obtained if the mineral aggregate is properly selected and if the bitumen possesses cementing value. Experience has shown that the heavier the traffic to which the pavement will be subjected, the finer must be the particles composing the mineral aggregate. In addition, the consistency of the asphalt cement must be of the right degree, that is, the newly laid pavement should not be too hard. The marking up of a pavement by the caulks on horses' shoes does not necessarily indicate that the pavement has been improperly constructed. If the pavement were too hard at the start it would have a shorter life than if a softer asphalt cement had been used. Sheet asphalt pavements properly constructed with native asphalts, either from Trinidad or Bermudez, have given the highest degree of satisfaction. The Barber Asphalt Paving Company recently took a census of the old asphalt pavements. In New York City there are 184,290 sq. yds. of Trinidad sheet asphalt 20 years old or older. A part of this is on Fifth Avenue and is subjected to the very heaviest traffic. The remainder is also subjected to heavy travel, yet all the pavements are in very good condition at the present time. Washington, D. C., has more than 1,000,000 sq. yds. of Trinidad sheet asphalt pavement laid between 1877 and 1901. The average cost of maintenance is 1.8c per square yard per year.

WATER WORKS

Method Employed in Repairing Leaks in Flexible Jointed Water Main in 40 Ft. of Water, Galveston Harbor, Texas.

About two years ago a water main of 8-in. cast iron pipe with lead calked flexible joints was laid across Galveston Channel, which is the waterway lying along the wharf front of Galveston. It extended from the wharf front, where it connected with the city water main, to the pile dike on the north side of the channel, a distance of 1,400 ft., thence it was laid in a shallow trench along the dredge spoil bank to the Government Immigration Station. The portion across the channel was laid in a trench 40 to 100 ft. wide, dredged to a depth of 41 ft. across the entire width of the Galveston Channel, which was then about 30 ft. deep for 500 ft. of its 1,400 ft. width. Cast iron pipe was used with flexible joints of the ordinary ball-and-socket type, which allow a deviation from a straight line of about 12°. The ordinary lead calking was used with yarn packing. The greater portion of the line was laid by calking up and lowering one pipe at a time, the line of pipe being carried on an inclined trough from the barge nearby to the bottom of the trench and the barge being fleeted forward as the line lengthened. Soon after the laying was completed and water had been turned into the main, leaks began to develop due partly to the method of laying and to the uneven character of the bottom, which caused such a great deflection at each joint that the lead calking was squeezed out, and to the fact that the line was caught by a ship's anchor before completion. Efforts were made to recalk the leaky joints by the aid of a diver, but these efforts were unsuccessful, as it was found the pipe was deflected at these points to such an extent as to close the calking space on two sides. After considerable thought and investigation, a method of stopping the leaks was evolved which has proven entirely successful. The method adopted is here described from information taken from an article by Mr. N. T. Blockburn, Junior Engineer, in Professional Memoirs for July-August, 1914.

The method was simply the placing of a wooden box form around each joint and filling this form with neat Portland cement grout. Each form was supported on three 3-in. pipe, 14 ft. long, driven down through holes in the forms into the underlying clay so as to form a pile foundation and prevent any settlement at the joint due to extra weight.

The form is shown in detail by the three drawings herewith, Figs. 1-3. It is 3 ft. 4 ins. square by 2 ft. deep, hinged at one edge

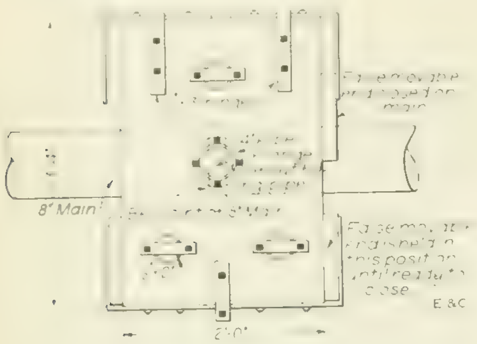


Fig. 1. Plan of Form Used in Repairing Leaky Flexible Joints in Submerged Water Main Under Galveston Harbor, Galveston, Texas.

so that it opens diagonally. In the top is a 4-in. hole with a pipe flange for connecting a 4-in. pipe extending to the top of the water, as shown on the sketch, and through which the grout was poured from the barge into the form. To the front of the lower half was fastened a chain which helped close it and held it closed afterwards, one link being let over a hook attached to the upper half and screwed up until the form closed tight. In either side, where the main passed through, a square hole 18 ins. by 18 ins. was left in order that the form would close no matter if the joining pipes were at their greatest possible angle, either vertical or horizontal. A movable section consisting of two pieces, 2 ins. by 12 ins. by 3 ft., each cut out in the form of an 8-in. semi-circle to take half of the main, closed around the main, lightly covering and at the same time entirely closing the 18-in. square holes. These false ends work outside of the form and are held in position by iron straps under which they can move in any direction against its side. Three holes large enough to take 3-in. pipe were bored through the top and bottom of the form for the piling. They were so bored that two piles would go on one side of the main and one on the other. After the pipe piles were driven down through the form flush with the top, a ½-in. iron plate was laid over the top of the pile and bolted to the form to carry the weight until the cement set around the piling.

The trench in which the main lies had been partly refilled with clay thrown into it by a dredge and by the natural deposit of sand, mud, and silt carried in by the cross currents and tides. This filling was partly removed with a 20-in. suction dredge, but fear of disturbing the main kept the dredge from working closer than about 4 ft. of its top. After the dredging was done the water was cut off the main, air pressure put on and leaks were located by the air bubbles coming to the surface of the water. All leaks of any consequence were marked by dropping a weight into the hole blown through the mud over a leak by the air and carrying a line ashore from the weight. It was not safe to use buoys for marking the leaks, as they were likely to be carried away by ships and all leaks had to be located and marked before repairs were commenced, as the air pressure had to be taken off the line and kept off until the cement was thoroughly set.

The plant used consisted of a derrick barge and a barge with an 8-in. belt-driven sand pump. A diver was in constant attendance.

The method of placing the form and filling with grout was as follows: The barges were anchored at the leak and the overlying sand and mud first pumped off the pipe. Then to the flange coupling on the form was connected a 45-ft. length of 4-in. pipe. Three rope lines were fastened to the front of the lower jaw of the form, one to the end of the closing chain and one near each side. The diver then took all three of these lines down around under the main and back up on the barge, where a man was stationed at each line. Then as the form was lowered away by the derrick with a line from the 4-in. pipe, the men took in on these lines and the lower half of the form, which dropped open when the form was picked up, was guided into place under the main. By lowering the upper half the form was closed. The piling were then set in the holes provided in the form and were driven flush with the top and the iron straps bolted over them. A jet was then put into the form through the 18-in. square openings and any mud in the form driven out and the joint thoroughly washed off. A piece of raveled, loose rope yarn was then tied securely around the leak to keep the cement from entering the main. The false forms over the ends of the form were then driven into place around the pipe and the form was ready for cement. The cement was mixed with salt water to a thickness that would just pour through a funnel into the 4-in. pipe leading down into the form. It was found necessary to pour it slowly in order to give it time to settle. Displaced water went out of the form through the holes around the piling in the top of the form. When the form was filled, the nuts on the bolts in the flange union fastened to the form were taken off and the 4-in. filling pipe removed, and the job was finished.

On the last leaks which were closed, pouring the cement through the pipe was abandoned owing to too much lost time in waiting for it to settle. The pipe, however, was still used to lower the form and to hold the form in position until the piling were driven. After this it was taken off and the cement, which was mixed as thick as possible, was lowered down to the diver in buckets and poured into the form through the hole at which the pipe had connected.

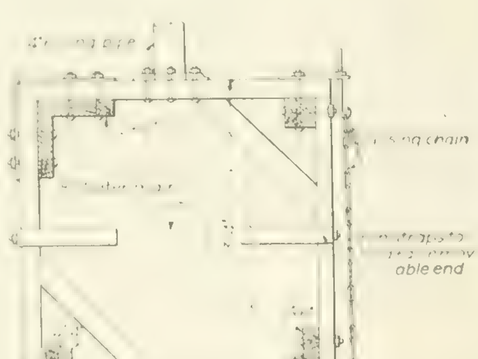


Fig. 2. Side Elevation of Form for Repairing Leaky Flexible Joints in Submerged Water Main in Galveston, Texas.

Where the soft mud and silt was so bad that it could not be kept out of the form, a 4-in. centrifugal pump with a flexible suction end was used to clean out the form after it was in place and all closed, the mud being pumped out through the cement filling hole while a jet alongside stirred it up. In the work of closing these leaks it was found necessary to have the pipe and inside of the form absolutely clean, so that the cement would adhere to the pipe. It was found necessary to take the form off the first leak and do the work all over again, as mud had been pocketed in the grout.

When repairs were completed, all cement was allowed to set a week. About 40 lbs. of air pressure was then put on the main and kept on for over an hour. During this time not a single air bubble could be seen and

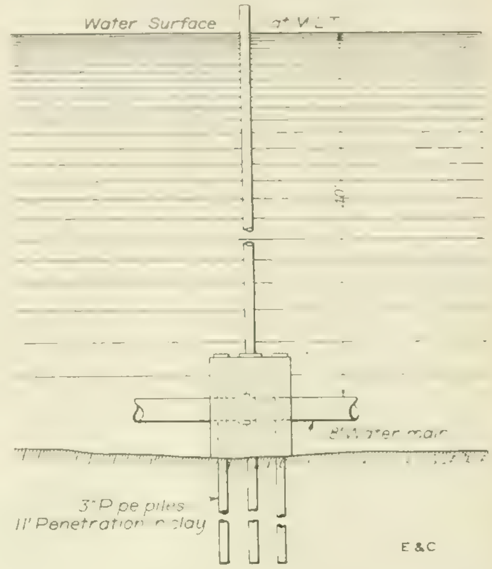


Fig. 3. Sketch Showing Form in Place for Repairing Leaky Flexible Joints in Submerged Water Main, at Galveston, Texas, by Encasing Them with Neat Portland Cement Grout.

the water meter showed the leaks had been stopped.

Four leaks were repaired and the entire work was executed in about four weeks, including the time of assembling plant, dredging, building forms, etc. By actual time a form was lowered and fastened around the pipe in 45 minutes. To close the false end gates of the form required 50 minutes. To drive the pile required from 1 to 1½ hours, depending on how hard the driving was. To mix the cement and fill the form required 1 hour and 15 minutes. A good deal of credit is due the diver for the rapidity with which the form was placed. In this connection it is interesting to note that during the entire work the diver went down without a suit, simply placing the diving helmet over his head.

With the proper plant after the main is clear, one leak a day can be repaired. The total cost of the repair work, closing four leaks, was $2,300.

The work was done by the United States Engineer Department, under the direction of Lieut. Col. C. S. Riché, Corps of Engineers. The plan of closing the leaks was devised, the form was designed, and work superintended by Mr. O. R. Scott, U. S. Inspector. The force consisted of one foreman, one hoisting engineer, six laborers and one diver, Mr. Albert Majors of Galveston.

Method and Cost of Installing Water Service Connections at New Orleans.

The installation of water service connections at New Orleans is by force account under the supervision of an assistant engineer at the head of a sub-department. The following notes on the methods and cost of conducting this work are taken from a recently issued report by Messrs. Rudolph Hering, George W. Fuller and Harrison P. Eddy on the work and organization of the Sewerage and Water Board of New Orleans.

The work of installing house connections consists in inserting a corporation cock into the street main of the distribution system, the laying of a lead pipe therefrom to the property line, and the connection therewith of a meter, which has been properly tested and which is enclosed in a suitable meter box. This sub-department handles all house connections smaller than 2 ins. in diameter. Those connections larger than this are installed by the water maintenance sub-department, which also does all repair work other than the repairing and retesting of meters. The latter work is handled by a crew at the main water pumping station, under the direction of the mechanical engineer in charge of the operation and maintenance of the pumping stations and power plants for water and sewerage.

The cost of building these house connections, including overhead allowance for the office staff and assistant engineer in charge of this sub-department, but without overhead allowance for any other officers, for the first five months of 1914 is given in Table I.

The records are kept with unusual care, not only as regards the description and location of every structure put under ground, but also as to the cost data of this work and the checking systems employed to account for all material purchased by the city. Information is also kept to show the relative efficiency of different gang foremen.

This organization consists of the assistant engineer in charge, who has in his office one chief clerk, one application clerk, one cost data clerk and one stenographer who jointly serves four sub-departments. There is a general foreman in the field, who directs the work of five gang foremen, each of whom has a driver, plumber, plumber's helper, tapper, and six to eight laborers to dig the ditches.

The routine procedure, upon receipt from a plumber of an application for a water house connection, is that the application card, duly setting forth what is wanted for a given property, and duly signed by the owner thereof, is sent to the record office, to see that no mistake has been made in filing an application for a connection to a property that has already been so furnished. Then the information needed in the field is copied onto a card, and each day the general foreman comes to the main office, receives transcripts from the various application cards, which he allots to the different gang foremen. Each of the latter works in a district that minimizes distances in traveling, as far as practicable. He also requisitions from the Washington Avenue Material Yard and has delivered to his driver for use on his various jobs ample quantities of the various materials needed therefor. Each evening, on returning to the yard, each gang foreman turns over to the office, for delivery to the main office, a card on which he fills out exactly the amount of each kind of material involved in building each house connection, with an accurate description of the location of the corporation cock and meter box duly referenced to an easily identified property line or street intersection. He also records the condition in which he leaves the excavation and the paving, with a description of the latter. Another man, called the "follow-up man," visits these trenches on the following day to see that they are in a safe and proper condition, and also to verify the report of the gang foreman.

The restoration of paving is done by the paving sub-department, although mention should be made of the fact that ordinarily house connections are "jacked" through from the street main to curb line beneath asphalt paving or other expensive paving laid on a concrete base, thus avoiding the expense and inconvenience of disturbing the latter. This operation consists in forcing, with the aid of a jack, a jointed pipe with a sharp end on it from the curb to the excavation made above the street pipe, where the corporation cock is tapped into the main. As this jointed iron pipe is jacked back through the hole through which it has been forced the lead pipe is attached to the iron pipe and pulled through the opening made by the iron pipe. In dirt streets or streets paved with large stone block, where there is no concrete base, it is cheaper to dig a trench than to resort to the operation above described.

TABLE I.—COST OF INSTALLING METERS AND SERVICES AT NEW ORLEANS.

Month.	⅝" to 1½" services.	⅝" to 1½" meters.	Material.	Labor.	Office and supervision
January	576	583	$ 8,813.12	$ 1,975.40	$ 655
February	536	549	8,059.00	1,903.05	655
March	783	804	11,688.29	2,451.30	673
April	654	659	9,953.46	2,098.95	655
May	874	889	13,310.65	2,714.95	682
Total	3,423	3,484	$51,824.52	$11,043.65	$3,320

Each week an inventory is made of all material on hand on each wagon at the end of that week's work and sent in to the office. A comparison is then made of the materials used in actual construction plus those on the wagon at the end of the week with the materials originally furnished. In this way misuse or loss of materials is rigidly guarded against.

Each week the data are available for showing the cost of labor and material used by each gang foreman in building the water house connections which he has installed during the week. These records are checked over and embodied as monthly records in the report of the head of this sub-department. This report shows a comparison in the efficiency of the several gang foremen, and records the average cost of building the house connections, a vast majority of which are ⅝ in. in diameter. In our judgment, this is as far as it is worth while to carry the cost data, as the length of leads, from corporation cock to property line, while averaging about 25 ft., vary too much to make it of value to secure the cost data for each individual connection, and the actual cost of material on each individual connection is recorded on the back of each job report, but actual cost of labor on each job is not reported on the job card.

Our attention has been called to the possibility of building at one time all water house connections needed in a single block. We do not consider this practicable unless arrangements can be made by the various property owners, after due conference with their plumbers, to decide just where, at the curb line, they want to have their meter box located. If this detail were not all worked out adequately in advance, which we think impracticable, misplaced services would result and cause much inconvenience and expense to the property owners. This would result in the laying of many connections which would never be used because of arrangements made by property owners to connect several premises with a single lead and meter, which arrangement, under the present system, results in a material saving in expense, both to the board and the water taker.

It has also been suggested that economies might result if the water house connection sub-department were united with the sewer house connection sub-department, and also if the two types of house connections were built in a single trench. So far as uniting these two sub-departments is concerned, we think that this will never be practicable or wise. All men now employed in these two sub-departments are busily engaged on two classes of work which differ quite essentially in character.

We do not favor an attempt to build the two house service lines in the same trench. Their alignment and depth are materially different. The street sewers have an average depth of about 7 ft., whereas the water pipes are not more than 3 ft. below the surface. The sewer house connections which must be laid to grade usually run obliquely from the nearest Y-branch to the property line, whereas the lead water pipes run at right angles to the street water mains, which are placed on the opposite side of the street from the sewer. It is highly undesirable to place a meter and meter box upon the newly backfilled material in the trench in which a sewer house connection has been built. We do not believe it is generally practicable to build water house connections on a bench adjoining the trench of a sewer house connection under the local conditions.

SKILLED LABOR—WHITE.

1 Foreman at $3.33 per day.
1 Plumber at $3 per day.
1 Tapper at $3 per day.
1 Helper at $2 per day.
1 Driver at $2 per day.

ORDINARY LABOR—NEGROES.

4 Laborers at $1.75 per day.
4 Laborers at $2 per day.

Data on Quantity of Water Used in Lawn Sprinkling.

Contributed by H. R. Ferris, Victoria, B. C.

Our company maintains an estate comprising (closely) 30 acres in lawns and 1.5 acres in shrubs. The area is made up of approximately 20 acres in long narrow boulevards cut at frequent intervals by streets and private driveways, and of 11.5 acres divided into small parks varying in size from 0.05 acres to 1.5 acres. About one-half the total area has a sub-soil of light sandy or gravelly loam, and the rest a sub-soil of stiff white clay, the whole covered with from 8 to 18 ins. of rich, black loam. The water is delivered by means of a ¾-in. hose, through sprinklers of various designs, the kind of sprinkler used depending on the size of the space watered, slope of the ground, etc.

The water is metered and a very careful daily check is kept to prevent waste. The season has been an unusually dry one for this locality, as the precipitation records in Table I will show, but enough water has been used to keep the grass green and healthy.

TABLE I.—WATER CONSUMPTION IN LAWN SPRINKLING AT VICTORIA, B. C.

Month.	Cu. ft. used — Total.	Cu. ft. used — Per acre.	Precipitation, ins.
April	15,000	476	1.04*
May	335,000	10,635	0.18†
June	205,000	6,508	1.67‡

*Light, ineffective showers. †No showers, hot, dry winds. ‡Two moderate showers, hot, dry winds.

In April we used 15,000 cu. ft., sprinkling only during the last two weeks. In May an unusually dry and rather hot month, we used 335,000 cu. ft., watering the entire area at least once daily and sometimes oftener. In June we had two light showers of about 36 and 12 hours duration each, which lowered the quantity of water used to 205,000 cu. ft.

Making Meter Repairs at Cincinnati.

In the latter part of October, 1913, the Cincinnati water department established a meter repair department. Procedure in the new department is here described from information taken from the annual report of Mr. J. A. Hiller, general superintendent.

A new meter testing table has been installed which permits of six meters, from ⅝ in. to 1 in. in size, being tested at the same time, this work being handled by one man, who can likewise assist in meter repairs when not engaged in his special work. As the city does not own any of the meters in use, it has been left optional with the consumer to have his meter repaired by the city or by the manufacturer. But with the present equipment the department is prepared to handle all meters of ⅝-in., ¾-in. and 1-in., exclusive of the piston type, and as over 94 per cent of the meters in service are 1-in. or less, the department is in a position to take care of practically this entire number. As soon as a meter is removed from the premises for test and examination, and it is found in need of repairs, the owner is notified at once, and on learning that repairs are made by this department at the cost of labor and material only, he gladly avails himself of the opportunity to have this work done by the city, thus eliminating the profit of the middleman or manufacturer. Under former conditions meters were withheld from the premises an unreasonable length of time, owing to the fact that in many instances they were shipped to factories located in distant cities which exacted payment in advance of repairs being made. Now these conditions have been remedied, as repairs can be made by the city and the meter returned to the premises without needless delay.

Table I shows the number of meters repaired by this department, different sizes and average cost of each.

TABLE I.—COST OF REPAIRING AND TESTING WATER METERS AT CINCINNATI IN 1913.

Size, ins.	No. repaired.	Total cost.	Average cost.
⅝	223	$487.41	$2.18½
¾	16	39.15	2.44¾
1	28	85.53	3.05¾

Average cost of meters repaired, not including testing charges: ⅝-in., $1.18; ¾-in., $1.45; 1-in., $2.06.

By the introduction of the meter repair department the consumer is protected from exorbitant bills he formerly paid and thus is added another feature that will contribute to the popularizing of meters.

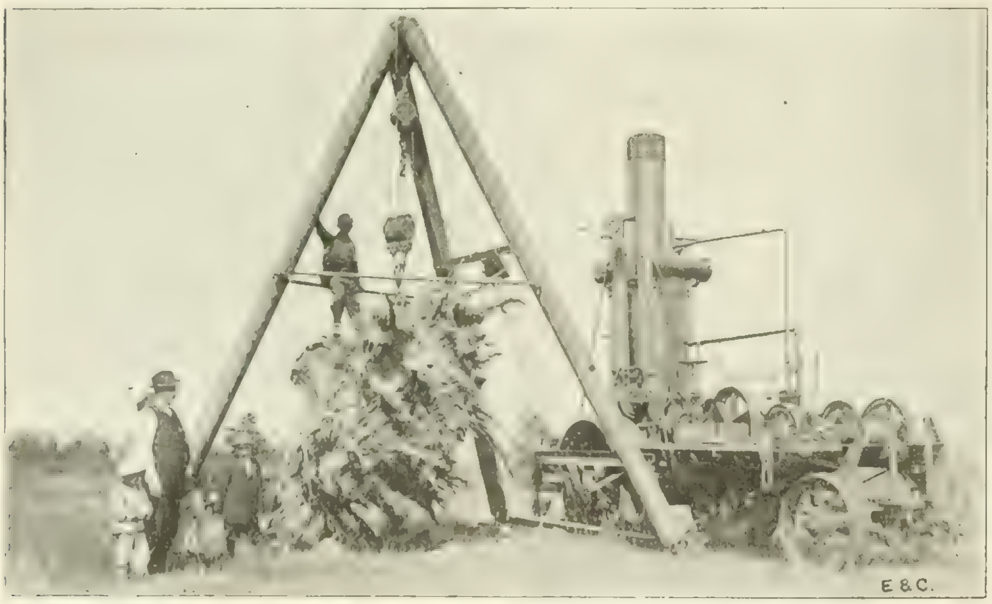
An auto truck will also be used in the future for delivering meters, as in this manner a great saving of time is effected, especially when long distances have to be traversed in reaching outlying districts.

CONSTRUCTION PLANT
MACHINES DEVICES MATERIALS

A Powerful Steam Land-Clearing Machine.

(Contributed.)

Development of the vast areas of cut-over land, both in the north and in the south, is proceeding at a pace which demands improved appliances and machinery for pulling stumps and grubbing. The conversion of the wild tracts left in the wake of the lumberman, to fields ready for tillage, is becoming recognized as a problem in reclamation, to be solved by the application of engineering principles. The demand has clearly been for a power machine capable of pulling the smaller growth and grubs, and the largest stumps as well, at a speed that would enable the land to be brought under cultivation in the shortest space of time and at minimum cost.



Pioneer Land Clearing Machine.

The engine shown in the accompanying illustration, the Pioneer land-clearing machine, was designed with these requisites in mind, and with the further idea that such a machine ought to be able to travel readily from place to place, under its own power, to reach various jobs, or different parts of the same job. The difficulty in designing a light, portable machine of this kind was to obtain sufficient power and strength for the heavy work without exceeding the weight and size of the ordinary traction engine. This was accomplished by the use of a large steel tripod, which is swung from the side of the engine in such a way that while pulling the stump the tripod rests solidly on the ground, entirely free from rigid connection with the machine; while for moving from stump to stump the tripod is tilted and raised so that it hangs entirely from the engine and is clear and free of the ground.

The machine is steam power, of 20 HP. capacity. It has a vertical boiler and a horizontal engine of a very rugged type, mounted on a substantial frame of structural steel. The engine is geared back through the medium of a suitable clutch, to the traction mechanism; while forward the power is applied to two drums mounted on the front end of the frame, geared down to give a very powerful pull. An auxiliary drum mounted on the gallows frame at the sides raises and lowers the tripod.

The wire rope which winds upon one of the drums leads to a pair of blocks suspended from the apex of the tripod, and by varying the number and arrangement of the sheaves practically any desired power could be obtained. However, it has been found that a pull about 45 tons, which is easily within the capacity of the machine using a pair of 5-sheave blocks, is ample for pulling stumps up to 4 ft. diameter. After the stump is pulled it is but the work of a few moments to lift the tripod, locate over the next stump, and drop the tripod into position again.

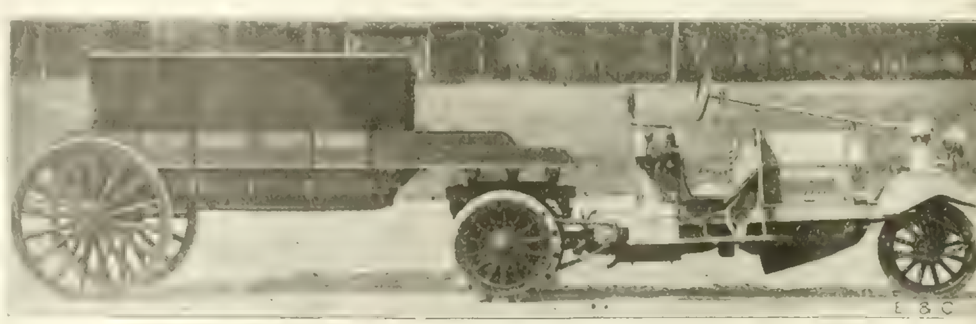
However, it is not necessary to use the tripod except for that small percentage of the stumps which are of the largest size. All the lighter growth, grubs and smaller stumps are pulled by the ropes running forward direct off the drums. Working in this manner it is best to use plow steel wire ropes, ⅝ in. diameter, and about 250 ft. long. The engine is located at a spot where it can be anchored to a stump in the rear, and where a big stump ahead will serve to attach two head blocks for the purpose of leading the ropes fairly onto the drums. Through these blocks the two ropes work simultaneously, covering a circle at one setting, amounting with the above length of rope to an area of over three acres. The drums give a direct pull of about seven tons, which suffices for all stumps up to 15 ins. diameter; and by doubling back through a single block, which involves but little loss of time, everything up to 20 ins. diameter is pulled. The comparatively light rope used enables it to be handled at maximum speed, a thing very essential where the grubbing is heavy.

The tripod also affords a means of piling the stumps. For this operation the rope is run through a block at the top of the tripod, and the tripod guyed to a stump in the rear. The stumps can then be skidded in, a higher speed on the drums being provided for this purpose, and they are readily dragged up into piles of considerable height.

This machine is sold by the Pioneer Land Clearing Machine Co., 1220 First National Bank Bldg., Chicago. It is adapted to the use of parties developing tracts of land on a considerable scale; or for individuals located within the cut-over districts either north or south, it affords an opportunity for profitable contracting, as there is any amount of work to be had at profitable prices.

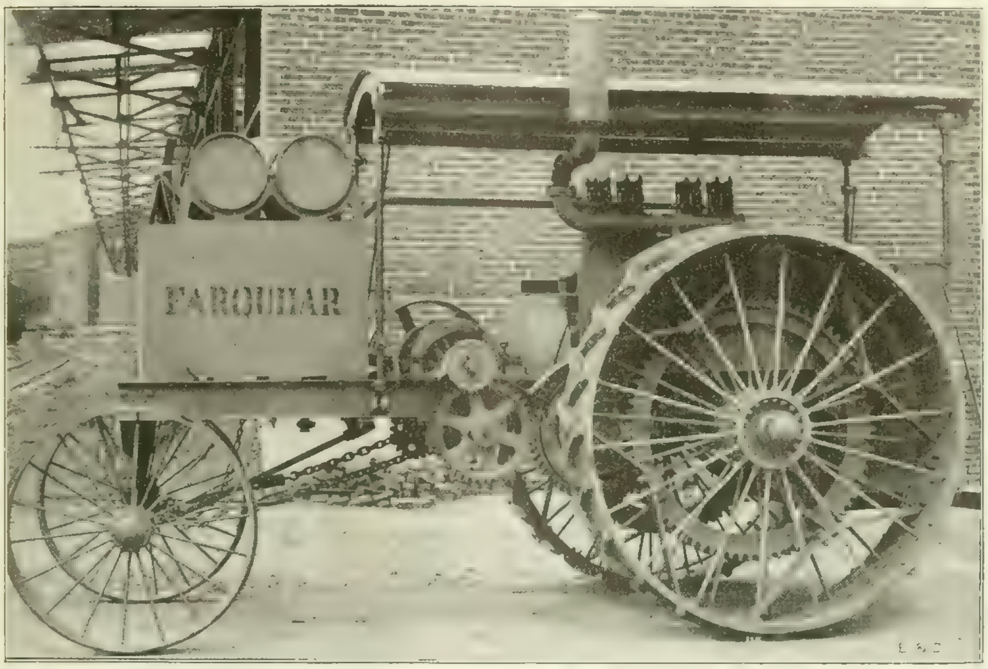
The Knox Trailer Tractor.

The motor tractor illustrated is unique in that it provides the necessary power for any type of wagon. The idea in mind in designing this tractor was to provide a flexible engine which could be quickly coupled to any type of wagon and therefore could be operated continuously, avoiding delays incident on loading operations. By an arrangement of a fifth wheel and kingpin on the rear of the tractor the rear wheels are made to serve for the front wheels of the trailer. The important advantage of this arrangement is that the stiff rear springs required by an ordinary motor truck to support loads in excess of 6 tons are not necessary. A ten or fifteen ton body may be used upon comparatively light rear



A Tractor and Bottom Dumping Trailer.

truck springs—a load beyond the capacity of springs of most self-contained trucks. Additional advantages of this arrangement are that steel tires may be used under the rear of the load and a bottom dumping trailer may be used. The illustration shows a Watson bottom dumping trailer in use by the city of Springfield, Mass., hauled by a tractor manufactured by the Knox Motor Co. of Springfield, Mass.



A General Purpose Gasoline Road Tractor.

A General Purpose Gasoline Tractor.

(Contributed.)

The gasoline tractor illustrated has a business like look that is interesting. The first impression is that of accessibility of parts and simple construction. Each part shows its purpose clearly; no attempt having been made to box in mechanism and produce a fine appearing machine.

Briefly stated the various details of construction are as follows: The 30-HP. motor is of heavy duty construction throughout. There are four cylinders with 6-in. bore and 8-in. stroke. Both cylinders and cylinder heads are cast in pairs from charcoal iron. Valves are of the poppet type overhead construction operated through rocker arms and push rods. They are drop forged from a solid piece and located in the head surrounded by water. Crank shaft is vanadium steel hammer forged, machine and oil treated, and ground. The fly wheel flange is forged solid to the shaft. There are three crank bearings 2¾ ins. in diameter. Length of the center bearing is 6 ins. and of each of the end bearings 7½ ins. The crank pins are 2⅞ ins. in diameter and 5 ins. long. Speed from 500 to 550 revolutions a minute. Ignition is of the jump spark type with dry cells for starting and a Remy three-magnet ball-bearing R. F. heavy duty type magneto. A Kingston 2-in. carburetor regulates the fuel control. It has a throttling ball governor connected to it. This carburetor is suitable for kerosene as well as gasoline.

Cooling is by the water system. The water is circulated by pump having a capacity of 16 gallons a minute. Water tank in the front on engine has a capacity of 250 gallons. As shown in the illustrations there are two gasoline tanks set on top of the water reservoir. Each tank has a capacity of 30 gallons. Oiling is by the splash system with pump delivering 1¼ gallons per minute.

Bevel gear drive transmission is used. This operates through friction clutches giving the necessary forward and backward motions. There is a bevel pinion fastened to the end of the crank shaft which drives two bevel gears. These gears are situated on opposite sides and bolted fast to friction drums on either side of the center of motor. Gears are made of nickel steel, machine cut, heat treated, and ground. They run in oil bath continually and with proper care should last as long as the motor. Only one lever is necessary to operate the transmission for both forward and backward motions. Regular belt pulley is 32 ins. in diameter with 5-in. face. It makes from 250 to 275 revolutions a minute. Master pinions and master gears are each 4½ ins. wide. Face of the engine pinion and differential 4 ins. each. Rear axle is made of I-beam sectional cast steel, stationary and rigid. Spindles are machined to fit brass bushings in hubs of drivers.

Rear drivers are 84 ins. in diameter, 20-in. rims. High or flat cleats furnished as desired. Front wheels are 48 ins. in diameter with 9-in. rims. Spokes are round and set into a cast-iron hub which is brass bushed. Edges of rear wheels are reinforced with 2½x¾-in. bands. Fifth wheels and front cross piece on the frame are of solid steel castings. The cap on top of the axle is also of steel. The draw bar extends the full width of engine, permitting straight and oblique hitching. Approximate weight of engine complete ready for work, eight tons. Road speed 2 3/10 miles per hour.

The tractor is manufactured by the A. B. Farquhar Co., Ltd., York, Pa.

The Jeffery Quadruple Drive Motor Truck.

The illustration shows a newly developed type of motor truck manufactured by the T. B. Jeffery Co. of Kenosha, Wis., successfully negotiating plowed ground and a ditch with a load of hay. The novel feature of this truck is its four-wheel shaft drive by which, it is claimed, superior tractive ability is secured. Other important features are an arrangement of the steering gear by which all four wheels are turned, permitting much shorter turns to be made; the height of the carburetor above the ground, permitting operation of the truck in water 34 ins. deep; and both front and rear wheel brakes, operated singly or together.

The 1½-ton truck has an overload capacity of 20 per cent. The wheel base is 125 ins.; the chassis frame being 38 ins. wide and 191 ins. long. Wheels are 36 ins. in diameter, tired with 5-in. solid rubber tires. The tread is 56 ins. A 32-40-HP., four-cylinder motor having a three-point suspension and equipped with Bosch duplex ignition, magneto and battery, is used. Gasoline tank capacity is 25 gals., with 5 gals. in reserve. The oil capacity is 3 gals. Springs are semi-elliptical. There are four speeds forward and a reverse; the normal number of revolutions per minute on the fourth speed being 1,000. The height of the body platform above the ground is 45 ins. when empty and 41 ins. loaded. The ground clearance is 15¼ ins. under the axles

Jeffery Quadruple Drive Motor Truck.

and 24 ins. under the transmission. The weight of the chassis is 4,800 lbs. All parts are standardized and interchangeable. The price of the chassis, including minor equipment, is quoted at $2,750, f. o. b. Kenosha, Wis.

For hauling trailers a draw-bar is provided that has a coil spring cushion and is rigidly cross braced to the frame. This type of draw bar relieves the starting shock. The truck was developed to meet the rather severe service requirements of the U. S. Quartermaster Department and a number of them have been purchased and are in use by that department.

A Hand Operated Ratchet Pipe Cutter.

The Strickler ratchet pipe cutter cuts cast iron, steel and wrought iron pipe from ¾ in. to 30 ins. in diameter. The cutter is hand operated and can be used equally well in the trench or shop. The cutting blades make a channel cut around the pipe the same as a lathe cut. Burrs and ragged edges are avoided and neither filing nor reaming is required. The cutter is made in eight sizes and each size cuts a range of pipe sizes. The illustration is of the No. 5 cutter which cuts pipe from 8 to 12 ins. in diameter.

The cutter opens wide and is quickly placed on the pipe. It centers easily and is fastened by tightening one swing bolt. The ratchet head, which holds the handle, is equipped with

The No. 5 Strickler Ratchet Pipe Cutter for Cutting 8 to 12 In. Cast Iron, Steel or Wrought Iron Pipe.

dogs engaging the teeth on the body of the machine. The body holds the star feed cutting blade. As the machine rotates around the pipe the feeder sends the cutting tools in automatically. One man can cut 8-in. pipe with this cutter and two men can cut 20-in. pipe. After clamping the machine around the pipe it is necessary only to pump the handle to make the cut. The cutter here illustrated weighs about 90 lbs. It requires only 3 ins. of clearance. These cutters are made and sold by W. W. Strickler & Brothers, Columbus, O.

Quick Dumping Hydraulic Hoist for Packard Trucks.

A dumping motor truck body must discharge from the sides or end. The operating mechanism of the truck prevents bottom dumping. To avoid excessive height of body above the ground and to provide at the same time a steep discharge angle is the problem

Hydraulic Dumping Hoist on Packard Truck.

with which designers have to contend. One solution of this problem that provides a good dumping angle is by tipping the whole body, using a hoist or elevating device at one end.

The hydraulic hoist illustrated can be used on a 3, 4, 5 or 6 ton Packard chassis, and gives a 45° dumping angle for dumping sand, gravel, coal, brick, etc., and a 55° dumping angle for dumping asphalt and similar material; the body being raised, lowered or held stationary between these angles and a horizontal position.

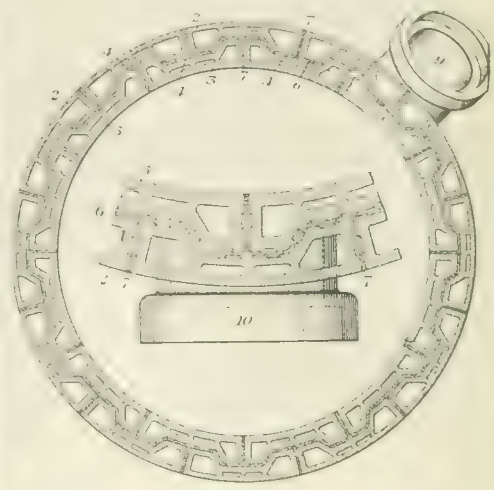
As may be seen from the illustration, the hoist is mounted in a semi-vertical position, rigidly braced behind the driver's seat. All working parts are encased and run in oil, the pump being located in an accessible position at the base of the hoisting cylinder. The weight of the hoist complete is about 600 lbs. Dumping is accomplished by the driver without shifting his position at the wheel.

It is claimed that the 55° dumping angle and tapering steel body constructed of ½ in. of sheet asbestos between two shells of heavy sheet steel makes a superior equipment for handling hot asphalt. At the present time the Barber Asphalt Paving Company is using six of these trucks for that purpose on paving work in Chicago. One of these trucks seen by the writer is now hauling hot bituminous concrete 8 miles in 55 minutes, delivering the hot asphalt in excellent condition.

A Hollow Lock Joint Vitrified Clay Tile for Sewers and Culverts.

The Natco lock joint sewer tile is illustrated herewith. The view shows the blocks assembled to form the circular conduit and within this circle is shown a detail of the construction at the lateral connection. This product is manufactured and marketed by the National Fire Proofing Co. of Pittsburgh, Pa.

As will be noted in the illustration, the complete section is made up of blocks of two forms known as the outer and inner blocks. The joints lap from 6 to 9 ins., and are made with cement. In moulding these blocks they are made double and are broken apart by a chisel and hammer preparatory to their use. In excavating a trench to receive this form of conduit it is necessary to form the trench bottom to template to provide a supporting surfact for the outer section. The radius of this template is, of course, that of the outer surface of the conduit. When the outer blocks have been laid so that they extend about one-quarter of a block above the horizontal diameter of the conduit the inner blocks are placed to the same height. A center, 1 in. thick, is



Cross Section of "Natco" Lock Joint Sewer and Culvert Tile and Detail at Lateral Connection.

then placed within the conduit and the arch blocks are placed. The center is then moved to the next section of blocks. If the soil traversed is too soft to hold in excavation the shape and size of the conduit, the tile can be assembled outside the trench in sections 6 to 8 ft. long and can be lowered into the trench in half sections 24 hours after assembling.

That this type of construction is strong is indicated by a test recently made at Columbus, O., on a 48-in. sewer. The test was made by piling 10 tons of cement in bags on top of the sewer. The load was 1,000 lbs. per square foot and the pressure per lineal foot was 3,090 lbs. No signs of fracture were apparent.

The advantages claimed for this hollow, lock joint, vitrified clay tile are that the blocks being in large units are quickly laid at a correspondingly low cost for labor; the inner surface is smooth; the blocks are salt-glazed; and the construction is applicable to sizes over 36 ins. in diameter, the upper limit of size for the vitrified pipe in ordinary sewerage practice.

A 150-Ton Floating Crane.—There has recently been placed in commission at the United States Navy Yard, Boston, Mass., a 150-ton floating crane for Government service. The crane is electrically operated and is carried on a steel pontoon which is 70 ft. wide by 125 ft long. The normal draft of the pontoon is 7 ft. and its moulded depth is 14 ft. The crane is of the cantilever bridge type, with an overhanging arm at each end, the trolley of which travels the full length of the bridge. The overhang of each arm is 64 ft. 3½ ins., from end to center of block, and the equipment is so arranged that the crane can operate from a height of 77 ft. above water level to 18 ft. below water level. The hoisting equipment consists of two main hoists which are capable of operating simultaneously, and an auxiliary hoist of 15 long tons capacity. Each main hoist has a capacity of 75 long tons. The contract price for the floating crane completely equipped was $294,397.92, the contractors being Wellman-Seaver-Morgan, of Cleveland, Ohio.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLII. CHICAGO, ILL., AUGUST 19, 1914. Number 8.

The Earliest Recorded Estimate of the "Development Cost" of a Railway.

Development cost, or the deficit in fair return that comes as a sequel to "interest during construction," has become a cost item of great importance to appraisers. There are still some engineers who seem to look upon this cost item as if it were a new discovery, because they themselves have not been in the habit of estimating it. Therefore it is of more than ordinary interest to find that 70 years ago a definite estimate was made of the probable development cost of a railway.

Mr. F. S. Burroughs, Chief Engineer of the Public Service Commission of Washington, has called our attention to a part of the history of the Northern Pacific Railroad wherein probable development cost, as estimated by Asa Whitney, is given.

In 1845 Whitney made a 1,500-mile trip up the Missouri River, and then went to Washington, D. C., with a scheme for constructing a railroad from Lake Michigan to the Pacific Coast. He asked for certain land grants as means of securing funds not only with which to build the road but to operate the road for ten years. It is clear that Whitney realized that the road could not pay even operating expenses during that period.

On page 64 of his History of the Northern Pacific Railroad, Eugene V. Smalley says:

> Whitney estimated the length of the road at 2,030 miles, and the cost of construction at $40,600,000, to which he added $20,000,000 for repairs and operation until the road should pay expenses, making a total of $60,600,000.

Thus it seems evident that Whitney estimated the development cost at 50 per cent, even without including the interest lost on the investment during the ten-year period.

The two receiverships that the Northern Pacific went through prove that a very heavy development cost was incurred, in spite of assistance from the government in the form of land grants.

Water Works Data of Exceptional Value.

The most important compilation of water works data made in recent years is that by the committee of the American Water Works Association, on tabulation of water rates and other information of interest to water companies. This report was presented at the latest annual convention of the Association, and is published in full in the recently issued quarterly journal of the Association for June. We understand that this report, which occupies slightly over 100 pages of the journal, will shortly be issue separately in pamphlet form. The tabulations are much too extensive to permit of their publication in this paper, but we desire to call attention to their scope so that the reader may fairly judge of their potential interest to himself.

The first tabulation pertains to the flat and meter rates in vogue in over 300 American cities. This tabulation also gives data on ownership of works, number of consumers and meters, and states whether supply is treated or not. The second tabulation pertains to the distribution system and gives, for the same cities covered by the first tabulation, the mileage of mains, sizes of mains, the number of fire hydrants and drinking fountains, the average number of hydrants and taps per mile of main in each of the cities, number of taps in service, the percentage of the total population supplied and the amount of "free water" supplied. The third table gives information relative to electrolytic action on pipes and service lines, and indicates the provisions in force in the various cities as to additions to the distribution system. The tabulation on metered service gives the number of meters in service, the percentage of taps metered, data on ownership and maintenance of meters, frequency of meter readings, cost deposit required, if any, rate of interest paid, if any, on required deposit, and states whether metering is optional or not. The next tabulation pertains to the supply itself and gives very full information as to the source and treatment of the supply, system of supply and operation, average monthly, daily and hourly rates of consumption, and pressure carried at pumping station, and in the business and residence districts. The tabulation on public fire service gives information relative to the percentage of fires where steamers are used, nature and extent of separate high pressure fire system, annual revenue per hydrant, maintenance of hydrants and valves, size of hydrant connections, description of fire hydrants used, and whether hydrants conform to national standard or not. The information pertaining to private fire service covers the following points: charges for service for automatic sprinkler connections, hydrant rentals, maximum size of fire service connections, and meterage of private fire lines. The concluding tabulation pertains to service connections and tells by whom these are installed, how paid for, price per foot, by whom maintained, charge for tap and size of tap.

The reader will readily appreciate the fact that a wealth of information is contained in the report under discussion. This information is of the widest interest to the operators of water utilities and will be immensely valuable for comparative purposes. In introducing the report the committee wisely cautioned users of the data presented against using any segregated item of information without due regard to the local operating conditions. This relates, of course, with special emphasis to the matter of water rates.

The report gives a very clear presentation of operation conditions and methods in representative cities. A copy of it should be secured and preserved by all water department officials and water supply engineers.

The Work of the Pennsylvania Highway Department.

At its inception the present highway department of the state of Pennsylvania had laid out for it by law a definite task to perform. A system of trunk roads comprising nearly 9,000 miles of highway was designated to be improved, minor roads of local importance were to be built where requested, and numerous administrative duties performed, among them the collection and expending of the automobile tax and the publication of accurate county maps.

Briefly stated, main traveled roads leading to the state line and connecting cities, boroughs and townships are by law made state roads, to be constructed and maintained wholly by the state. Other roads improved upon application of the political unit in which they lie are to be built and maintained jointly by the state and political unit. This unit may be a township, in which case the township pays one-half of the cost; or the county and township may act together, in which case each pays one-fourth of the cost. In either case the state pays one-half of the cost, the total amount of aid granted being dependent upon the mileage of road in the unit.

Thrusting aside details and considering for a moment the idea upon which this classification is based, the bigness of the plan is apparent. Modern conditions demand that certain interurban roads be maintained in a higher degree of improvement than is necessary to meet the demands of traffic originating along their line. That the cost of this higher type of service be met entirely by the local political unit is manifestly unfair. The construction and maintenance of these roads—either wholly or in part—is distinctly a function of the state. The construction and maintenance of distinctly local roads is just as clearly the function of the local unit, the extent to which the state participates in meeting the cost of the work depending on their value to the state as a whole. The value of adopting a definite plan of improvement will be more apparent in future years. This is usually the case with large undertakings which have in view future welfare.

The undertaking of this work was begun in a small way. First appropriations were small to accomplish more than a part of the preliminary work necessary. Adjustments with various other state departments affected by the highway law were necessary and new details of organization had to be tried out in actual practice. The growth in importance and amount of the work accomplished has, however, been steady and a basis has probably now been reached upon which the carrying out of the work planned can be accomplished. Slow growth should, undoubtedly, promote efficiency in the personnel of the department.

One of the most interesting features of the organization perfected is the arrangement for the supervision of township road work. The state is divided into districts, each in charge of a superintendent, who supervises repair, maintenance and construction. This work is under a separate bureau of the department and covers practically the entire state, involving all classes and types of road building and maintenance.

Perhaps the most characteristic feature of the work of the department is one which is typical of the people of the state—thoroughness. Whatever things are undertaken are accomplished in a thorough manner. This keynote will be found running through the publications of the department and its effect is seen in the work completed.

Our Iron and Steel Exports to Various Countries.

It is interesting to note the volume and destination of our iron and steel exports. Published statements usually have given the volume of such exports but generally have not given the destination of these exports. The Bureau of Foreign and Domestic Commerce has recently published its tables of exports by countries, which extend through the fiscal year ending June 30, 1913. These data show that our iron and steel exports are far from being uniformly distributed. The particular kind of development going on at the time and the cost of transportation are important factors which influence the volume of exports and make it difficult for those exporting materials to keep closely in touch with the various markets.

The total exports of iron and steel in the fiscal year 1912 amounted to 2,509,797 gross tons, which increased to 3,019,142 tons in the fiscal year 1913, although recent exports have been somewhat less.

About three-fourths of our 1913 exports of 102,201 tons of iron and steel scrap went to Canada, while practically all of the remainder was sent to Italy, the latter having developed recently a steel industry, largely on the basis of working up scrap. Our pig-iron exports, which in 1913 amounted to 287,022 tons, have been chiefly to Canada, the latter taking 222,-184 tons. England took only about one tenth as much as Canada, while Italy imported from this country about 20,000 tons and Austria-Hungary about 8,100 tons.

Our total exports of ingots, blooms, billets, sheet bars, etc., amounted to 235,000 tons in 1913. Of this tonnage between 150,000 and 175,000 tons went to England and most of the remainder to Canada, the latter country using practically all our exports of wire rods during the past four years, the totals varying from about 20,000 tons in 1910 to 18,000 tons in 1913.

Our rail exports have steadily increased since 1909. In that year they were 234,138 tons; in 1910 they were 369,578 tons; in 1911, 391,428 tons; in 1912, 417,547 tons; and in 1913, 452,-545 tons. The demand of individual countries for steel rails has varied widely, and it is not possible to build up a regular trade with any particular country. The rail exports to South America in 1909 amounted to 50,000 tons; in 1910 to 130,000 tons; in 1911 to 96,000 tons; in 1912 to 131,000 tons; and in 1913 to 94,000 tons. The various countries of South America, however, varied widely as to their imports of rails from this country. In 1909, 1910 and 1911, Argentina used more than one-half of the total exports to South America, while Brazil was a moderately close second. In 1912 the exports to Argentina decreased, while those to Chile exceeded those either to Brazil or Argentina. In 1913 the exports to Brazil were 32,000 tons; those to Argentina were 25,000 tons; to Chile, 14,000 tons; and to other South American countries, 23,000 tons. Our rail exports to Mexico were 80,000 tons in 1909, but decreased each succeeding year until they reached 20,000 tons in 1913. In 1910 our rail exports to Cuba were 40,000 tons, although this tonnage has slightly decreased since that time. Our exports to Canada varied from 25,000 to 40,000 tons in 1909, 1910 and 1911, but in 1912 they jumped to 118,726 tons and in 1913 to 138,439 tons. Present developments in Canada, however, do not promise a continuance of such a large rail trade to that country. Outside of the American continent our rail exports have not been large. In 1913, 15 per cent of the total went to Oceania, about 10 per cent to Japan, about 3 per cent to China and 1 per cent to Africa. Europe imported from this country only 3,045 tons.

In 1913 our exports of steel plates were 263,000 tons, and of black sheets, 133,000 tons. Canada took 75 per cent of the plates and 81 per cent of the sheets exported. Japan and Australia each imported from this country 6 per cent of the total tonnage of plates, while Chile took 8 per cent of the sheets and Scotland 4 per cent. The total exports of galvanized sheets in 1913 amounted to 115,000 tons, of which Canada took 32 per cent, Mexico, Argentina and Australia each took 7 per cent, the Philippine Islands used 6 per cent, and Japan came next with 5 per cent. The galvanized sheets were exported to 58 countries, Europe taking about 5 per cent of the total.

This summary of some of the iron and steel products exported will suffice to show the difficulty of keeping in touch with the fluctuating demands of various countries. For some products it is possible to build up a somewhat constant trade with certain countries, while for other products the demand is only temporary.

BUILDINGS

Design Features of the Commonwealth Pier No. 1, East Boston, Mass.

(Staff Article.)

The Commonwealth of Massachusetts is to construct a pier in East Boston to be known as "Commonwealth Pier No. 1," which will possess some interesting features. The main building is 157 ft. 8 ins. wide by 882 ft. 0 in. long, between outside column centers, and is surrounded by a concrete platform. The structure is two stories high and rests mainly on a wood pile foundation. The first floor contains two lines of tracks near the center of the building, which run practically the entire length of the pier. The remaining space on this floor is used as a wharf and for storage, except a small portion at the shore end, which is divided into several rooms. The second floor contains a large examination room for ship passengers at the shore end, this room being surrounded by small offices. Adjacent to this there is a large room for steerage passengers, and adjoining this latter room there are accommodations for first and second cabin passengers. At the outer end of this floor there is a large open space surrounded by wide gallery. The outer end panel of the structure is only one story high, the flat roof of which forms a lookout balcony. The project also includes a grain conveying gallery and supports. This gallery is located at one side of the pier and is supported on the steelwork of the main structure. It connects with the grain elevator of the Boston & Albany R. R. The lowest bid received for the entire project was $658,857.

FOUNDATIONS.

The work to be done under the contract for the foundations of the pier consists of the construction of a timber bulkhead, the filling of the area back of it, the construction in front of the bulkhead of a pile and timber wharf with a concrete floor slab, and in connection with latter the construction of concrete foundations, supported on piles, for a steel frame building. The contract also provides for building sewers, drains, pipes and conduits. Another contract provides for the excavation alongside of the wharf to a depth of 40 ft. below mean low water.

The estimated quantities of materials required under the contract for the foundations are given in Table I.

Bulkhead.—The bulkhead, which extends diagonally from a point in the fifth panel from the shore on the east side to a point in the seventh panel on the west side consists of creosoted pine piles, yellow pine stringers and 6-in. yellow pine grooved and splined sheeting. The 6-in. sheeting is of short-leaf yellow pine, creosoted with 16 lbs. of dead oil of coal tar per cubic foot. The sheets are grooved and have a width of 8 ins. The splines are of spruce and are 1½x3 ins. x 10 ft. long. They are fastened to the sheet piling with 6-in. spikes driven not more than 2 ft. apart. Figure 1 shows a plan and elevations of the bulkhead and gives details of its construction. It is required that the filling inside of the bulkhead be placed in layers in such a manner that, if there is any tendency for it to flow, the direction of flow will be away from the bulkhead. The portion of the building inside of the bulkhead rests on an earth fill.

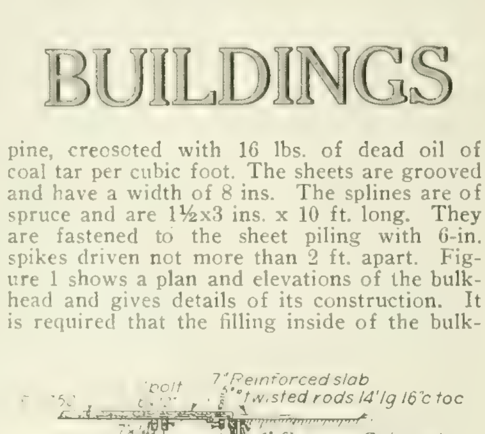


Fig. 1. Plan and Elevations of Bulkhead of Commonwealth Pier No. 1, East Boston, Mass.

Wharf Construction.—The wharf, which extends in front of the bulkhead, rests on wooden piles, which are capped with timber caps and which support a reinforced concrete deck.

The foundations for the support of the steel frame building are constructed within the area covered by the wharf. Cast-iron belay posts are set along the outer edge of the platform. In the center of the wharf there is a track pit containing two tracks, which terminate near the outer end of the platform.

The foundations supporting the columns of the steel building consist of clumps of creosoted pine piles, cut off at elevation 12.5. After cutting the piles to grade their heads were again creosoted. Upon these pile foundations are built concrete piers to receive the steel columns. The tops of the piers contain core holes for the anchor bolts of the columns. Slots are moulded in these piers to receive the timber caps of the pile wharf. After the caps are in place they are grouted to insure a uniform bearing. The concrete used in these piers is a 1:2:4 mix.

TABLE I.—ESTIMATED QUANTITIES OF MATERIAL IN FOUNDATION OF PIER.

Item.	Quantity.
Excavation, cu. yds.	[illegible]
Ordinary refill, cu. yds.	20,000
Riprap, tons	100
Track ballast, cu. yds	200
Plain concrete, 1:2:4, cu. yds.	2,000
Plain concrete, 1:3:6, cu. yds.	253
Reinforced concrete, 1:2:4, cu. yds.	3,090
Brick manholes, number	8
Granolithic surfacing, sq. yds.	2,200
Spruce or non-treated pine piles, number	740
Creosoted pine piles (5,415 piles), lin. ft.	282,975
Oak fender piles, number	280
Oak bulkhead piles, number	170
Hard pine lumber, ft. B. M.	700,000
Square oak lumber, ft. B. M.	4,500
Rough oak braces, lin. ft.	45,000
Railroad ties, number	200
Reinforcing steel, lbs.	492,007
Miscellaneous, cast-iron, lbs.	15,000
Cast-iron belay posts, number	17
Wrought iron bolts and washers, lbs.	245,000

The contract also includes sewers, piping, conduits, etc.

Figure 2 (a) shows a half cross-section of the pile substructure taken through the center of a line of column foundations; and Fig. 2 (b) shows the framing for a typical pile bent between columns, of which there are two between each set of columns. The substructure framing is symmetrical about the center line of the track pit, except that for the platforms which extend beyond the building. The platform for the half not shown in Fig. 2 (a) extends 6 ft. 8 ins. beyond the center of the outside line of columns, instead of 10 ft. 8 ins., as given for the half shown in the drawing.

The piles supporting the wharf are creosoted short-leaf yellow pine, except the fender piles along the outer edge of the wharf, which are oak. It is specified that the pine piles shall be at least 14 ins. in diameter 3 ft.

from the butt, and that the diameter at the tip shall be at least 7 ins. These piles are to be creosoted with 16 lbs. of creosote oil per cubic foot of timber.

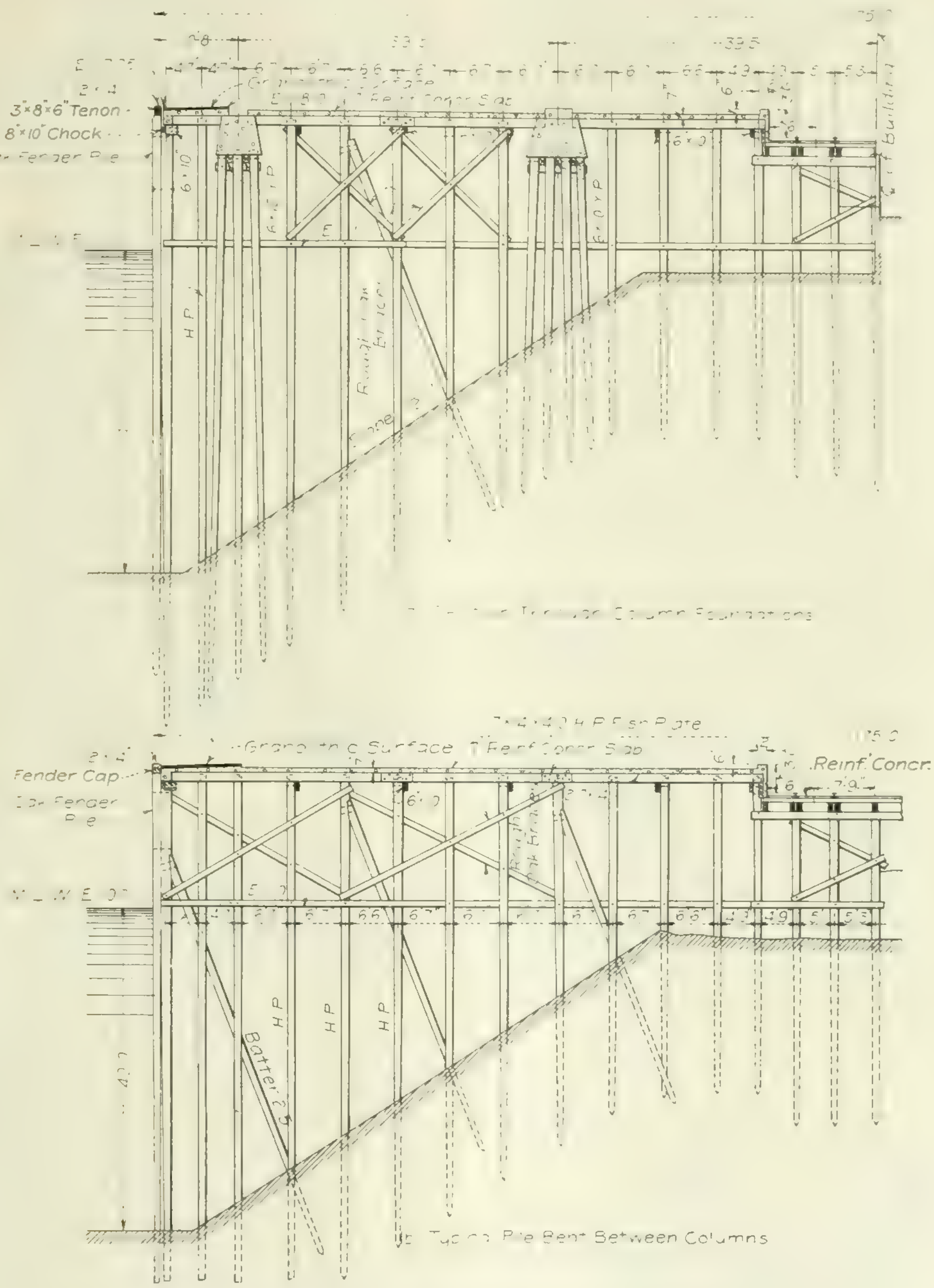


Fig. 2. Half Section Through Column Foundations and Elevation of Typical Pile Bent Between Columns of Commonwealth Pier No. 1.

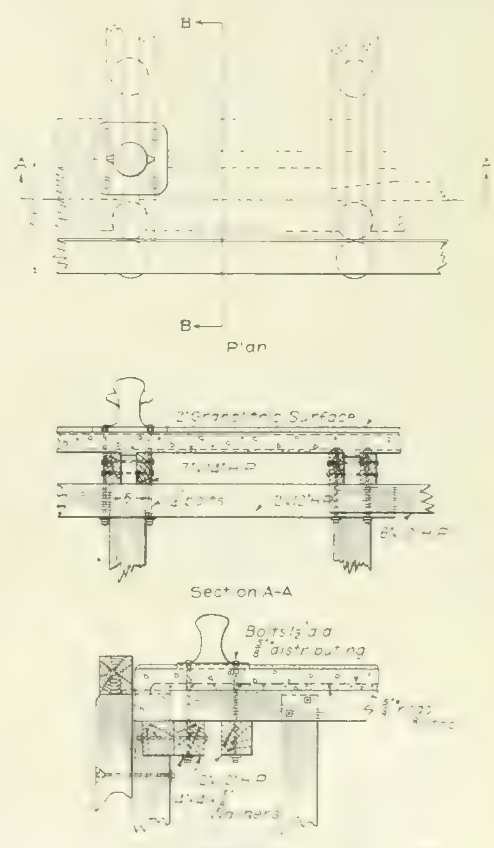


Fig. 3. Plan and Sections of Wharf Platform of Commonwealth Pier No. 1. Showing Details of Construction.

The oak piles for the fenders on the two sides and outer end (except at the corners) are required to be white or red oak, of which not less than 20 per cent shall be white oak. These piles shall be not less than 14 ins. in diameter, 5 ft. from the butt and not less than 6 ins. in diameter at the tip (including bark in each case). The corner fender piles are white oak, of the same dimensions as the other oak piles, and each pile is bolted to the framing of the platform with three 1¼-in. screw bolts, the heads of which are countersunk to a depth of 5 ins. The bolts are to be of double-refined iron.

The lengths of the piles are indicated approximately in Fig. 2, but it is specified that the creosoted pine piles shall penetrate at least 15 ft. into the original clay bottom—no pile to have a length less than 40 ft. It is required that the oak piles penetrate at least 10 ft. into the original clay bottom.

The caps for the piles are hard pine, each timber being 7x14 ins., with the butt joint on the piles. They are fastened to the piles with two 1-in. screw bolts. At the butt joints each cap is bolted with one bolt, and is further secured by bolting a splice timber, of the same section as the cap and 4 ft. long, with four 1-in. screw bolts. The fender piles are capped with 12x14-in. hard pine caps, secured to the piles with mortise and tenon joints and oak tree-nails. The fender caps are spliced between the piles with a ship splice 2 ft. long, secured by four ¾-in. screw bolts with large flat heads.

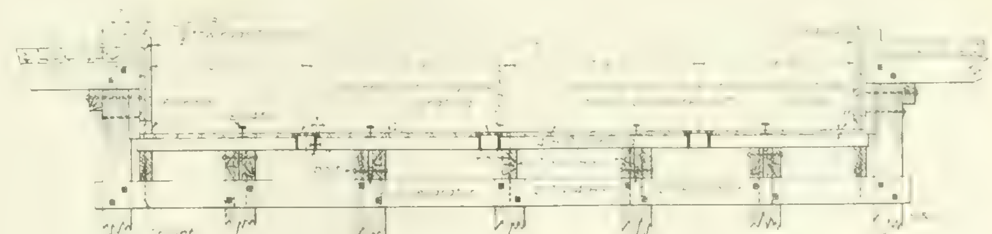


Fig. 4. Cross-Section of Track Pit of Commonwealth Pier No. 1, Showing Construction Details.

The piles are braced longitudinally with 6x10-in hard pine timbers, which are secured to the piles with 1-in. screw bolts placed just under the girder caps. The outer row of bearing piles have in addition 8x10-in. hard pine chocks tightly fitted between them and bolted to the longitudinal stringers with ¾-in. screw bolts, three to each chock.

The main piles in the bents are braced with spur piles (see Fig. 2), which are fitted to the main piles and are fastened with two 1¼-in. screw bolts, and with oak braces and girders bolted to the piles with 1-in. screw bolts.

The outer corners of the wharf are strengthened with timber platforms and braces bolted to the piles. The outer face of each corner has fender piles of white oak spaced about 2 ft. apart, with solid square oak timber chocks between them.

Figure 3 shows a plan, a cross-section and a longitudinal section of a portion of the outer platform. This drawing shows the location of one of the belay posts and gives construction details of the platform. The reinforced concrete platforms have a 2-in. granolithic surfacing.

Track Pit.—The track floor is supported on 7x14-in. hard pine stringers resting on the girder caps of the bents under the track pit. Under each rail there is a double 7x14-in. stringer, the timbers of which are bolted together with 1-in. screw bolts having cast-iron separators 2 ins. thick between them. These bolts are spaced 3½ ft. apart. The timbers forming the double stringers break joints, the butts falling on the bents. Each stringer is bolted to the girder caps with one 1-in. bolt 20 ins. long.

hard pine timber floor 4 ins. thick, each extending the full width of the track pit except where it butts against the column foundations, is spiked to the stringers with ⅜-in. square wrought-iron ship spikes 8 ins. long. The spikes are driven through ⅜-in. holes, which are previously bored in the planks.

After the tracks are laid on the timber floor the whole track pit is covered with a 3-in. concrete slab, reinforced with steel mesh. On each side of the track pit there is built a thin reinforced concrete wall, in the top of which, at intervals, are set angles to form rests for bridges across the track pit. The concrete for the floor and sides is a 1:2:4 mix.

Figure 4 shows a cross-section of the track pit at the scuppers, and gives details of its construction.

Substantial bumping posts are placed at the ends of the tracks, an elevation and a cross-section of which are shown in Fig. 5.

Floor Construction of Pier.—The floor construction of the pier within the building consists of a 7-in. reinforced concrete slab resting directly upon the girder caps, upon which is placed a 3-in. layer of cinder concrete. The wearing surface consists of 3-in. yellow pine planks nailed to 3x4-in. sleepers embedded in the cinder concrete. (The pine wearing surface and the cinder concrete fill is included in the contract for the superstructure.) Figure 6 shows a plan and sections of the floor for the pier and gives details of its construction. At the inner end of the pier the reinforcing rods extend about 10 ft. over the bulkhead, to serve as anchors for the concrete slab. (The bulkhead slab is to be built later under a separate contract.) The concrete for the main floor slab consists of a 1:2:4 mix.

Foundations for Grain Conveyor Gallery.—A grain gallery about 460 ft. long connects with the conveying galleries from the Boston & Albany R. R. elevator on the shore. This gallery connects to the steelwork of the pier at a point near the inner end of the pier and continues from the latter point a distance of over 700 ft. along one side of the pier. From the elevator to the pier the gallery is supported on concrete piers having pile foundations. The portion of the gallery which extends along the pier is carried by the steel superstructure of the pier (see Fig. 7). The foundations for the columns of the grain gallery are of two types, one of which consists of six piles cut off at elevation 14.0 and capped with a concrete pier 2.4 ft. high, and the other, of nine piles cut off at elevation 8.5 and capped with a concrete pier 7.4 ft. high. For the 2.4-ft. pier the four 1¾-in. anchor bolts extend through the concrete and are anchored into 12x12-in. blocks placed between the piles and bolted to them; while for the 7.4-ft. piers the anchor bolts extends almost through the concrete and project 2 ft. above the piers.

[illegible]

The work included under the contract for the steel superstructure consists of furnishing and erecting in place, on the foundations previously prepared, the steel frame and other structural steelwork for the building on the pier, including a grain gallery and its supports. There are about 2,200 tons of steel in the superstructure.

Building.—Figure 7 shows a half cross-section of the steel superstructure and gives the principal dimensions and the sizes of the members. It will be noted that the columns in the bent are spaced 39 ft. 5 ins. apart, the distance between the center lines of outside columns being 157 ft. 8 ins. The building contains 42 panels at 21 ft. each, its total length, between center lines of outside columns being 882 ft. Beginning with the third bent from the outer end of the pier the middle line of columns "C" is omitted for a distance of 252 ft. (12 panels). For this distance the second floor framing is omitted between the column lines "B" and "D."

The second floor framing consists of 60-in. steel girders between columns, 12x16-in. yellow pine stringers, 3-in. yellow pine tongue-and-groove planks, and a ⅞-in. maple flooring. A thickness of asphalt felt is laid between the 3-in. planks and the maple flooring. This floor is designed for a live load of 150 lbs. per square foot. A 2-in. granolithic wearing surface is placed in the toilet rooms.

The roof consists of 2-in. spruce tongue-and-groove planks, nailed to 2x3-in. nailing strips, resting on steel purlins, and a 5-ply asphalt and slag roofing. The roof was designed for a live load of 30 lbs. per square foot.

The outer end panel of the structure is only one story high, the roof of which forms a balcony. The balcony has a reinforced concrete roof slab supported on steel I-beams.

The outside walls of the shore end of the structure, which here rest on an earth fill, are composed of 8-in. terra cotta blocks, plastered on both sides. The remaining portion of the sides is covered with corrugated iron. The shore end of the building also has a terra cotta wall. In the perishable goods room and in part of the third-class waiting room, the corrugated iron siding is backed with 8-in. terra cotta blocks up to the ceilings of these rooms.

Grain Gallery.—The grain gallery, which extends for a distance of about 714 ft. along one side of the pier, has its floor at elevation 97.0, which is about 79 ft. above the first floor of the pier. This gallery is carried on steel bents, the outside columns of which are in line with the outside line of columns of the building, while the inside line of the gallery columns are carried by the roof trusses (see Fig. 7). This gallery encloses a 36-in belt conveyor, which runs at a speed of about 900 ft. per minute. Grain is unloaded from the belt conveyor into ships through a long steel spout, which is carried by a cantilever A-frame. The latter runs on a track, which is carried by brackets on the outside columns of the gallery bents, this track being at about elevation 89. A guide track on the roof of the conveyor gallery holds the A-frame in position and provides for the horizontal thrust caused by supporting the steel spout at the outer end of the A-frame. The vertical load on the track due to the A-frame is about 5 tons, while the horizontal thrust at the roof of the gallery is about 7 tons. Two of these traveling frames and steel spouts are provided for loading grain into ships.

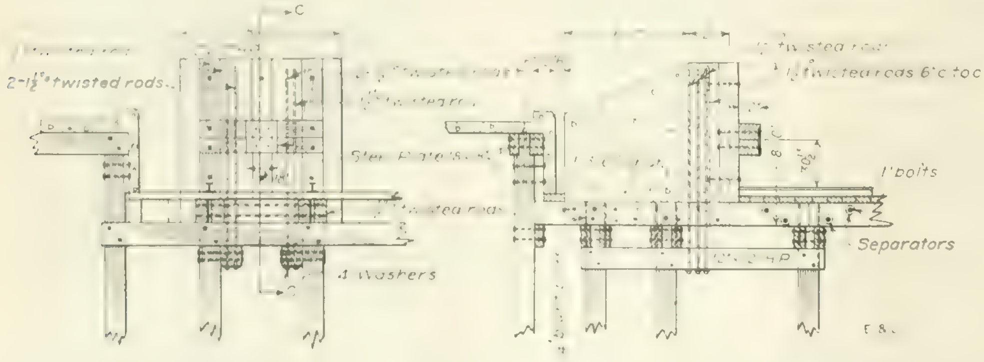


Fig. 5. Elevation and Cross-Section of Bumping Post for Commonwealth Pier No. 1.

The building is equipped with a complete automatic sprinkler system and fire hose pro-

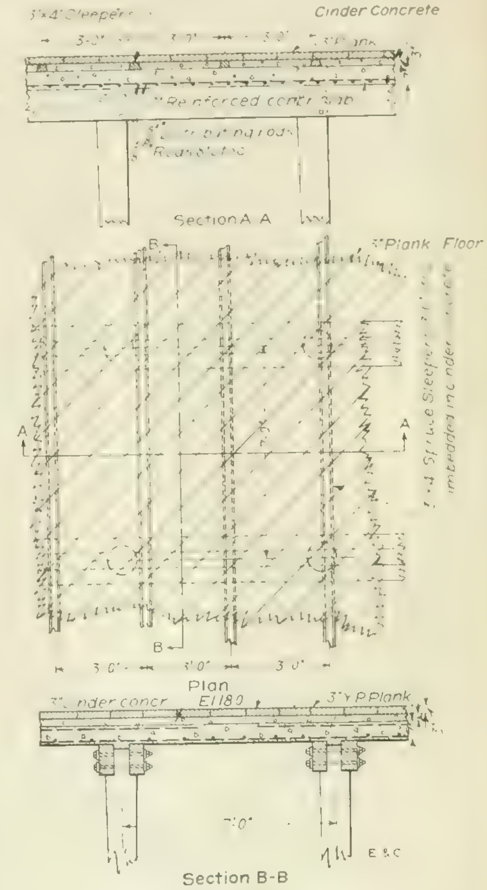


Fig. 6. Plan and Sections of First Floor of Commonwealth Pier No. 1, Showing Details of Construction.

tection. Two electric passenger elevators are provided, each elevator having an area of 80 sq. ft. and a clear head room of 9 ft. The capacity of the motors is sufficient to operate the elevators at a speed of 150 ft. per minute when lifting a superimposed load of 6,500 lbs.

Proposed System of Reinforcement for the Roof Timbers of Westminster Hall, England.

An interesting report on the condition of the roof timbers of Westminster Hall, in England, with suggestions for maintaining the stability of the roof, has recently been prepared by Mr. F. Baines, one of the architects in the H. M. Office of Works. This building dates from the year 1097, and the original walls still remain, the width between walls being 67 ft. 6 ins. It is believed that the original roof was constructed with intermediate supports in the form of pillars. The present roof was practically completed in 1399-1400. Many minor repairs have been made from time to time, but a thorough examination of the hall was not made until its care was assumed by the Ancient Monuments Branch in March, 1912, the present report being the outcome of this examination. This following article contains extracts from the report prepared by Mr. Baines.

The hammer beam roof contains thirteen trusses, of which eight have been selected for particular reasons and the causes of decay given as follows:

(a) Dry rot.
(b) Incipient surface decay.
(c) Decay due to the attacks of the xestobium tessellatum.
(d) Decay due to the attacks of a smaller anobild beetle.
(e) The ravages of the goat moth.

Many investigations have been made to find a remedy in the form of a liquid which would preserve the wood from further decay, but none appear to be practicable or effective although 18 different solutions have been considered. The report gives the results of a detailed examination of the main members of the trusses and shows that the defects due to decay and twisting are so serious that collapse is likely to occur at an early date if the trusses are not reinforced. Although the method of piecing the timbers with modern oak has been considered, this method is not advised. The roof timbers, exclusive of the roof sheathing, contain about 480,000 ft. B. M., which shows that any considerable cutting away and reparation would be very expensive.

WALLS AND FOUNDATIONS.

An investigation was first made to determine the stability of the walls of the hall, this factor being very important inasmuch as a system of steel reinforcement might be designed which would exert an outward thrust on the walls. Holes were sunk at the external faces of the walls of the east and west sides to ascertain the character and depth of the foundations and subsoil. It was found that the foundations rest on a solid bed of alluvial, gravelly sand, which is lower on the east side than on the west by about 6 ins. The lowest course of the wall footings consists of rough boulders of Kentish rag, with a good ballast filling between them. The information gathered showed that, provided the walls are monolithic throughout, there is no reason to expect that they will not be able to withstand any dead loads or thrusting stresses which they may be called upon to bear.

SYSTEM OF REINFORCEMENT RECOMMENDED.

The method recommended to maintain the roof in its integrity consists of a scheme of steel reinforcement, designed to affect the appearance of the structure as little as possible and to afford adequate support to each truss member. The particulars of this scheme will be developed later. Unless some such scheme is adopted it will not only be impossible to render the roof safe by piecing the timbers, but it might be necessary in certain cases to take some of the trusses down and entirely renew many of their members. To cut and piece the hammer beams would be a grave constructional error should they ever be called upon to carry unrelieved the present stresses caused by the roof. By means of some system of steel reinforcement every vestige of the original wood which is unaffected by decay can be retained, and only members, such as some of the purlins, will have to be wholly renewed. Such a system would prevent the complete collapse of the roof even though the timbers should continue to decay.

In all cases, however, where the decay has developed into large cavities and fissures in the wood, the defective timbers will have to be cut and pieced, so as to provide a solid bearing for the reinforcement on each side of the members. This piecing up in oak will have to be done simultaneously with the work of reinforcing the members with steel. Before this can be attempted a carefully designed center or supporting scaffold will have to be built up from the floor of the hall to carry fully every member of the trusses treated and also the intervening bays.

FACTORS TO BE CONSIDERED.

It is proposed to construct the center of steel members of ordinary commercial sections, and the major portion of it will be erected so that it can be rolled on wheels from one position to another. This procedure will be adopted for that portion which clears the truss under the hammer beam and rises about half-way up the arch formed by the two curved struts from the lower ends of the hammer post to the center of the great curved rib and collar beam. Above these levels the center will have to be taken down and rebuilt as its positions are altered, and it may be advisable so to arrange that the upper stages of the center, where they reach up to support the upper collar and higher portions of the upper principal, together with the ridge, should be constructed of timber erected from the steel center at the level of the middle of the hammer post.

The weight to be carried by the steel center for two trusses, one bay and the portion of two other bays will be approximately 115 tons, made up as follows:

	Weight tons.
Two trusses	[illegible]
One bay	[illegible]
Portion of two other bays	[illegible]
Total	115

Wind pressure, calculated at 20 lbs. per square foot over the whole roof surface, may increase this by another 60 tons for three bays, giving a total of 175 tons to be carried by the center in the worst case. Should it be found necessary, however, to center up at one time four trusses and three bays, this load will be about doubled. The center, therefore, together with its own weight will be carrying a very heavy load at the base, which the present floor slabs would not sustain safely. There is, however, about 18 ins. to 2 ft. below the slabs a layer of lime concrete, which rests upon made ground superimposed over Thames mud and blue clay. These subsoils are not in themselves satisfactory, but they are contained by the great walls of the hall, which go down to good sandy gravel for their foundation. The concrete should therefore safely carry the weight of the center and the superimposed load of the truss.

The difficulties of fixing the center are very great. At Eltham, where a similar but much smaller timber center has been constructed, the condition of the roof was such as to allow of the lifting of the truss by wedging up under the ends of the main collar beam, where it joins with the principal rafters. In the case of Westminster Hall roof, however, this method is quite impossible, for the joints between the main collar beams and the upper principals and hammer posts are so poor, and in some instances so much affected by decay, that if any attempt were made to carry and lift the roof at this point many of the trusses would collapse. A further complication arises from the fact that at Westminster Hall the principal rafters are in two sections, and any attempt to carry the roof at the main collar (the foot of the upper principals) would seri-

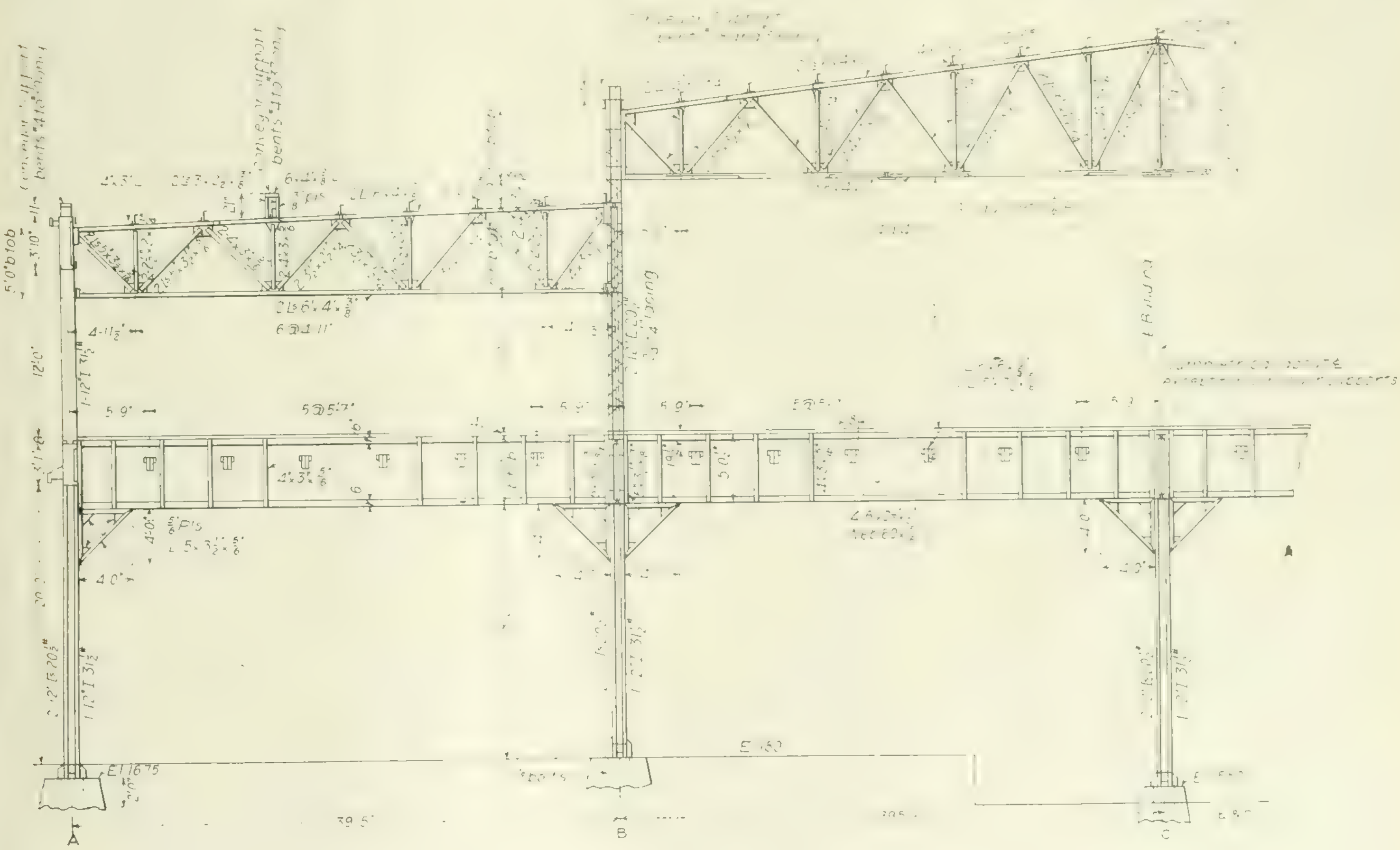


Fig. 7. Cross Section of Steel Superstructure of Commonwealth Pier No. 1, East Boston, Showing Type of Construction.

ously endanger the stability of the lower principals. The decay generally applies to almost every other structural point in the truss—namely, to the hammer beams (the timbers of which are mostly decayed at their wall ends and some also at their free ends), to the lower principals (which are decayed at their wall ends) and to the purlins, which have in some cases rotted throughout their whole length. The steel center must therefore be designed to clip and carry each individual member of the truss freely and by itself at points where the timber is sound. This has added greatly to the difficulty of designing the center and will necessitate the greatest care and foresight in actually carrying out the work.

A further point which will have to be observed carefully with regard to the centers is that they must retain the trusses absolutely in their present positions. No wedging up or drawing together must be attempted, and each truss must be strictly maintained in its existing vertical plane. Should one of the trusses, when freed from the strut supports of its purlins while the latter are being reinforced, rock either towards the north or south, the gravest danger of the collapse of the adjacent trusses would occur.

As the trusses must not be jacked or lifted up in any way, it will be necessary to make provision for drawing together the various sections of the reinforcement in the most careful manner before letting the truss sink back on to its supports after the reinforcement is fixed. This point is an important one, as it will be seen that unless the steel reinforcement is tight and true everywhere on the truss it will, when actually carrying its load, deflect under such load on the removal of the center, and possibly open many joints in the timber. This difficulty was overcome in part at Eltham by slightly lifting the truss and wedging up, which has taken a proportion of the spring out of the principal rafters, so that the reinforcement will be fixed on the principals in a more favorable situation than the actual position taken up by the latter before the centering was fixed. The deflection due to the load when the reinforced trusses at Eltham are put back on to the walls cannot be so great as the amount of the distortion which existed previously in the roof, and which has been relieved by the wedging up. Although it is intended to reinforce only two trusses at one time and to complete that section of reinforcement throughout, yet to accomplish this the three double ranges of purlins must be reinforced and connected up to the steel reinforcement of the two trusses for three bays. This will mean that two other trusses (one on each side of the two actually reinforced) must be tampered with and indeed freed from the strut supports of the purlins for a time. After which the reinforced purlins must be temporarily fixed to the unstrengthened adjacent trusses. In the present condition of the roof this will be a highly dangerous operation unless these trusses, as well as the two trusses dealt with and reinforced, are also centered and supported in the meantime. It may be deemed advisable, therefore, to arrange to center four trusses and three bays in order to reinforce at one time two trusses and three bays only.

In considering the proposed scheme for the steel reinforcement to the trusses, etc., the following main points must be considered:

(1) [illegible]

(2) [illegible]

(3) It should be designed so that too great a [illegible]

(4) It [illegible] against the possibility of collapse in any of the trusses, should further dangerous decay take place.

The problem would therefore appear to resolve itself into one of adding to the existing timber work an entire truss of steel, which would be able to support the whole of the present roof together with the weight of the steelwork itself, so as to bring the total weight of the strengthened roof to a safe and satisfactory bearing on the walls. The only alternative to this would be to strengthen by individual steel reinforcement each decayed member of the truss, and by inserting plates and bolts to make up the deficiencies in the strength of the pieced-up structure.

If this latter method, however, were carried out a very large proportion of the old timber would have to be taken out, while the steel straps which would be necessary to bind the work together could not be kept in the background, and would most seriously affect the general appearance of the structure. In addition, the scheme, though very costly, would not be a full and effective means of providing for the safety of the roof in the future, should the decay continue.

This alternative was therefor abandoned at the commencement of the consideration of this problem, and careful thought was given to the question of selecting the best form of steel reinforcing truss which would efficiently support the roof under the conditions laid down, and would interfere in the least degree with the general appearance of the timbers.

DETAILS OF SYSTEM OF REINFORCEMENT.

The system suggested above consists of a complete steel truss which follows the outlines of the principal rafters on the top and the outlines of the great arched rib on the underside, and finishes at the bottom by horizontal members at the level of the wall plate. The truss is braced throughout by a complete system of single triangulation with a few exceptions.

Considerable difficulty was experienced in dealing with the junction of the upper principal rafter, queen post and upper collar. These three members do not meet in a point, and in order to make the error in fixing as small as possible, the steel member which follows the upper collar is placed at the top of the timber, and the vertical steel ties are placed on the outsides of the queen posts, instead of in their center. This partly overcomes the difficulty, while the heavier reinforcing steel plates on the upper principal rafters are placed so that the secondary stresses may be satisfactorily resisted. In deciding on this form of truss, and on the sections to be employed very careful consideration has been given to the absolute necessity for doing everything that is possible to keep the steel reinforcement out of sight, and where this could not be done to construct the steel reinforcement in a simple and unobtrusive form.

It has also been necessary to give the closest attention to the question of building the reinforcement into position. The consideration of the facilities for the erection of the steel reinforcement on the site is a most important one, as a scheme might well be designed which it would be impossible to carry out without taking down a large proportion of the roof. In the present scheme, however, the work can be executed without interfering unduly with the existing timbers. It will, of course, be necessary to lift down on to the staging some of the members of the roof to allow of the introduction of the steel reinforcement. The most important of these, however, are the collar beams and purlins. With regard to the first, many of the collar beams are in such a dangerous condition that it would be absolutely necessary to lift one of the two timbers of this member out to repair the other, and so one would have to be removed in any case. Even if this were not true, the vast gain to the roof by getting the reinforcement to the main collar beam out of sight, by inserting it between the two timbers of this member, should make this method of erection an acceptable one. As regards the purlins, these are in such an extremely defective and dangerous condition that without some scheme of reinforcement most of them would have to be renewed.

The members which it will be necessary to take down in order to insert the reinforcement are therefore not such as could ordinarily be allowed to remain if the scheme of steel reinforcement were not followed. So much of the roof will be left in position, and indeed must actually be retained in position to allow the steel reinforcement to be applied, that the present proposal does, in effect, demand the retention of the majority of the members of the trusses undisturbed.

In providing for the insertion of this scheme of steelwork, the existing reinforcing timbers and steel tie-rods, etc., can all be removed, leaving the roof in this respect almost entirely as it was originally built.

The difficulties of erection and fixing will be great, and these will only become apparent as the work is continued; they should not prove insuperable, however, if experienced erectors, who have had some knowledge of such exceptional work, are employed in the execution.

The calculated loading includes the total dead load of the roof, together with the wind pressure, to which has been added about 550 lbs. per linear foot of the truss to cover the additional weight of the steel reinforcement in each bay. In the calculations for the steel reinforcement, however, the allowance for wind pressure has been increased. No snow load has been taken in the calculation, as it is unlikely that any serious weight of snow would remain on this roof at such a steep angle. The addition, however, of 6 ins. of snow covering the whole of the roof would only increase the tresses approximately to the extent of about 7½ per cent, and the safe stresses allowed for on the whole of the steelwork are taken in the calculation to give a factor of safety of 3¾ generally. Thus the addition of 7½ per cent to the stresses would leave a very satisfactory margin for the possible future depreciation of the steel by oxidation, etc. This is well within the usual accepted safe stresses on steel.

In selecting suitable sections for the truss members of the steel reinforcement, the limited space available, and the necessity for keeping the steelwork as much out of sight as possible have been very fully considered. The result is that the steelwork, which includes forged rods, heavy turned pins, large gusset pieces cut to very irregular sizes, etc., is not at all of a simple nature, and in the working out of the details a vast amount of calculation and consideration will have to be given to the designing of the connections. The complicated and costly nature of the steelwork, however, is necessary to meet the difficult and unusual conditions imposed by the problem. Whenever bolts are necessary in the joining of the steel truss they will have to be of the best turned steel.

In order to keep the section of the members which act as struts to neat and unobtrusive proportions, particularly in the case of the principal rafters, it was necessary to calculate the timber as a bracing which would prevent the buckling of the steel plates. Thus these plates on each side of the principal rafters will be bolted together through the thickness of the timber, and every portion of hollow and decayed wood must be made good by new pieces of oak. These must be secured, not by simple butt jointing, but by good scarfing joints, so that the timber itself will hold the steel reinforcing plates rigidly in their relative positions. This condition being attained, the steel alone will then be fully capable of resisting the compression without any further assistance from the timber.

In all the bearings, in order to bring the load well on to the walls and not upon their edges, a steel girder has been introduced, held down by an anchor bolt at the back and bearing upon a cast-iron bed plate, and thus on to the wall.

In order to carry out the erection, which will be a matter of extreme danger and difficulty, the most expeditious manner, as well as the safest, will be to erect it from a traveling steel stage or center, which has been previously described. On this stage at two levels—just under the hammer beams and just beneath the main collar beams—close boarded platforms will be fixed, and these would be the two main working stages for getting at the roof. Smaller platforms would be necessary to reach other parts of the roof for taking templates and for general purposes could be constructed in steel and timber from these platforms.

In order to get the steelwork made in the manufacturer's yard before delivery, the most careful and accurate full-size templates would have to be taken at the hall. These templates would be of an unusual size, and would have to be sent to the manufacturers in order that the steelwork would fit true to its positions when delivered on the site.

For the reinforcement of the purlins it is proposed to use steel joists with a wide top flange so that the steel member shall have stiffening in two directions and thus resist the tendency to sag in the plane of the covering of the roof, which is so obviously an existing defect.

It will be necessary to take down the existing timber purlins (re-using every portion unaffected by decay), and cut them in half for their full length, after which the halves will be bolted to the web of the steel reinforcing purlin girders. The reinforced purlin will then be put back in the same position, and will efficiently support the common rafters without sagging. This operation will necessitate the fixing of temporary purlins to carry the common rafters, as it is undesirable to take down the whole of the latter members; only such of them as are perished or seriously affected by decay would be removed.

A further important point which must be considered will be the necessity for the most accurate fixing of the steelwork throughout the truss. It may even be necessary to tie in the steel reinforcement across the width of the hall, so as to stress slightly the reinforcement throughout before it is let down upon the walls. When completed, and when the center is removed, the deflection under its full load should not then be sufficiently serious to cause fracture and opening of the timber joints.

The estimated cost of the proposed work is about $300,000.

Data for Use in Operating Compressed Air Hoisting Engines and Rock Drills.

The use of compressed air is an important factor in construction work, yet there are few available data which give those in charge of building operations usable information on this subject. The following data were taken from a paper describing the various uses of compressed air by Robert L. Streeter, in the Engineering Magazine.

The loss of power in transmitting air through pipes generally is not serious, and is not to be compared with the losses of power in the operation of compressing the air or in its re-expansion and final use. It is difficult to derive a satisfactory formula for the loss in pipes due to friction, as this loss depends upon so many factors, among which are: unit of time, volume of air, pressure of air, diameter of pipe, length of pipe and difference in pressure at the ends of the pipe. None of these factors can be allowed its absolute value, but each is subject to modifications due to its associates. Moreover, the actual diameter of the pipe, especially in the smaller sizes, is different from the nominal diameter, while the pipe may be straight or it may be crooked and have numerous elbows and valves.

FLOW OF AIR THROUGH A PIPE.

A common formula for the flow of compressed air through a pipe is,

$$Q = c\sqrt{\frac{pd^3}{wL}} \quad \ldots\ldots\ldots (1)$$

in which Q = volume in cubic feet per minute, p = difference in pressure in pounds per square inch causing the flow, d = diameter of pipe in inches, L = length of pipe in feet, w = density of the entering air in pounds per cubic feet, and c is a constant. The value of c was found to be 58 in the experiments made at the Mt. Cenis tunnel, these experiments being made with pipes 4, 8 and 14 ins. in diameter and 3,281 ft. in length. Table I shows the probable loss in pressure in 1,000 ft., of pipe for different volumes of air flowing at 80 lbs. gage pressure.

The volumes of the free air in cubic feet given in Table I are found by multiplying the value of Q, determined from equation (1), by $\frac{80 + 14.7}{14.7}$, or the number of atmospheres.

The presence of a globe valve in the pipe has the effect of increasing its length. For each globe valve the length to be added is given by the formula,

$$\frac{114 \times \text{diameter of pipe in feet}}{1 + (3.6 \div \text{diameter of pipe in inches})}$$

The reduction of pressure due to an elbow or a tee is two-thirds that caused by a globe valve.

TABLE I.—LOSS OF PRESSURE FOR DIFFERENT VOLUMES OF FREE AIR COMPRESSED TO GAGE PRESSURE OF 80 LBS. AND PASSING THROUGH PIPE EACH MINUTE.

Diameter of pipe in inches.	—Volume of free air in cubic feet compressed to 80 lbs.— 100.	200.	400.	800.	1,000.	1,500.	2,000.	3,000.	4,000.	5,000.
	Loss in pressure in lbs. per sq. in. for each 1,000 ft. of straight pipe.									
1½	5.8	....	....	....	....	....	....	....	....	....
2	1.05	1.5	....	....	....	....	....	....	....	....
2½	0.35	1.41	5.80	....	....	....	....	....	....	....
3	0.14	0.57	2.28	....	....	....	....	....	....	....
3½	....	0.20	1.05	4.16	6.4	....	....	....	....	....
4	....	0.14	0.54	2.12	3.27	7.60	....	....	....	....
5	....	....	0.18	0.68	1.08	2.43	....	....	....	....
6	....	....	....	0.28	0.43	1.00	1.7	3.91	7.10	10.7
8	....	....	....	0.07	0.10	0.24	0.42	0.93	1.68	2.59
10	....	....	....	....	....	0.08	0.14	0.30	0.55	0.84
12	....	....	....	....	....	....	....	0.12	0.22	0.34
14	....	....	....	....	....	....	....	....	0.10	0.16

VOLUME OF AIR REQUIRED TO OPERATE HOISTING ENGINES.

Any engine which will operate with steam will also operate with compressed air without any changes. Compressed air is used only in small units and principally in hoisting engines on construction work.

TABLE II.—VOLUME OF FREE AIR REQUIRED TO OPERATE HOISTING ENGINES, AIR COMPRESSED TO 60 LBS. GAGE PRESSURE.

Cyl. diam., ins.	Stroke, ins.	Revs. per min.	Nominal horse-power.	Actual horse-power.	Weight lifted, single rope.	Cu. ft. free air per min.
5	6	200	3	5.9	600	75
5	8	160	4	6.3	1,000	80
6¼	8	160	6	9.9	1,500	125
8¼	10	125	15	12.1	2,000	151
7	10	125	10	16.8	3,000	170
8½	12	110	20	18.9	5,000	238
10	12	110	25	26.2	6,000	330

The volume of air required in an engine depends on the cut-off, varying directly with it as does the horsepower. Hence for any given pressure the horsepower determines the quantity of air used. Too often air is used non-expansively in engines, and in this case the volume of air used is that swept through by the piston, with an additional 10 per cent for leakage.

Table II gives the volume of free air required to operate hoisting engines under working conditions, the air being delivered to the engine at 60 lbs. gage pressure. In compiling this table the hoisting engine is assumed to run only half of the time, while the compressor works continuously. If the engine runs less than half of the time, as it usually does, the air required will be proportionately less, and vice versa. The table is computed for maximum loads, which also vary widely in practice. The data are for single-cylinder engines. For double-cylinder engines of the same diameter and the same speed the horsepowers and the air consumption will be doubled.

VOLUME OF AIR REQUIRED TO OPERATE ROCK DRILLS.

In most types of rock drills either steam or air can be used in the same machine. Table III gives the volume of free air required to operate one rock drill of the size and at the pressure stated.

In order to determine the volume of free air required to operate a number of drills, the values given in Table III should be multiplied by the following factors:

Number of drills.	Multiplier.	Number of drills.	Multiplier.
1	1	12	8.1
2	1.8	15	9.5
3	2.7	20	11.7
4	3.4	25	13.7
5	4.1	30	15.8
6	4.8	40	21.4
7	5.4	50	25.5
8	6.0	60	29.4
9	6.5	70	33.2
10	7.1	..	....

The multipliers in the above table are for use at the sea level; for higher altitudes they should be slightly increased.

TABLE III.—VOLUME OF FREE AIR REQUIRED TO OPERATE ONE ROCK DRILL OF THE SIZE AND AT THE PRESSURE STATED.

Gage pressure, lbs. per sq. in.	—Cylinder diameter of drill in inches— 2	2¼	2½	2¾	3	3¼	3½	[illegible]¼	[illegible]½	3⅝	4¾
	—Cubic feet of free air required for one drill—										
60	50	60	68	82	90		[illegible]	[illegible]	108	113	140
70	56	68	77	93	102	108	110	113	124	129	147
80	63	76	86	104	114	120	[illegible]	127	131	143	164
90	70	84	95	115	126	[illegible]	[illegible]	141	152	159	182
100	77	92	104	126	138	146	149	154	166	174	199

Oxy-Acetylene Welds.—A paper on the strength of welds made by the oxy-acetylene process was presented at a joint meeting of naval architects and engineers recently held at Newcastle, England. The results of a series of repeated impact tests showed that the thicker the plate the less reliable was the weld and the greater the reduction in strength. Thick material also generally showed less improvement by annealing; in fact, under ordinary working conditions it was usually impracticable to anneal the material. As the result of a large number of tests and experiments upon autogenous welds, the conclusion was reached that, although there was a wide field of usefulness for oxy-acetylene and similar processes for joining or repairing steel plates, bars, etc., which were not required to resist very severe stresses, considerable caution should be exercised and due regard paid to all the conditions before making use of them. A very serious responsibility attended the use of autogenous welding in cases where the parts were likely to be subjected to considerable mechanical strain. Autogenous welds always represented a somewhat uncertain quantity, as there was no method, except by destructive tests, of ascertaining whether a weld was good or bad.

The establishment of a state highway commission in Florida is being urged by several county road commissioners of that state. A system of trunk lines through the state and state supervision of county road work are proposed.

WATER WORKS

Curtailment of Water Waste and Selection of Meters at Milwaukee Water Works.

The following notes on the curtailment of water waste and the selection of water meters in Milwaukee are taken from the latest annual reports of Mr. H. P. Bohman, Superintendent of Water Works, and Otto F. Poetsch, Superintendent of Meters:

During the past five years the average daily increases in the water consumption over the preceding year have been as follows:

	Daily increase, gals.
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]

At the rate of increase prior to 1913 the capacity of the present intake tunnel would be inadequate to meet the demand during the period of maximum consumption long before the new Linwood Ave. Intake now under construction, could be completed. This was fully realized by the department about two years ago and all efforts since then have been directed towards eliminating unnecessary waste.

Compulsory meterage of all unmetered services, including services supplying the public buildings, parks and grounds; sealing of automatic fire sprinkling systems; sealing of all private fire hydrants, rigid inspection of all premises where water is used for building construction purposes prior to the setting of the meters, and a general inspection of the distribution system, service connections, fire cisterns, etc., was the program mapped out.

Proof that the unmetered services and Automatic Fire Sprinkling Systems caused most of this excessive increase is forcefully shown by the insignificant increase in the pumpage for the year 1913 and the extraordinary increase in the revenue per 1,000,000 gals. pumped since all services are metered and the sealing of the automatic fire sprinkling systems.

The revenue per 1,000,000 gals. pumped for the past three years has been as follows:

[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]

As compared with 1912 the increased revenue per 1,000,000 gals. pumped amounts to $53,879.23, in the aggregate. No better argument can be advanced for the maintenance of a water waste department and the efficiency of water meters than the results shown above.

There are at present 59,233 water meters in service in Milwaukee. This includes all sizes from ⅝ in. to 12 in. of about 34 different types. When the city first started to meter its taps, which was about 35 years ago, there were few makes of meters on the market. At that time the meters were the property of the water department. Later on objection was found to this and a change was made, so that the meter was required to be furnished by the consumer and maintained at his expense. It is to be regretted that this change was made.

Water works, both municipal and private, throughout the country, generally, have found it advisable to own and supply the meters rather than have the consumer supply them as is done in Milwaukee.

It is vital to the success of the meter system that the water department shall determine the meter to be used and the manner of setting, the time for testing and repairs. This can better be accomplished when the meters are owned by the department, as the necessary red tape, delays and expenses, involved in sending notices to the owner or occupant and such trips to take off and reset the meters, are avoided. The department also can buy fittings in wholesale at prices lower than those usually charged by the plumbers. Meters purchased in large quantities can be bought at a better discount than is given a plumber, who must necessarily, buy them in smaller numbers.

Much time and expense would be saved, yearly, if the meters were the property of the department. As it is now, when a meter becomes out of order, it is necessary to take it off, insert a connection pipe and average the account during the time the meter is off. This averaging of the account, quite frequently, meets with objection, as it is impossible to determine if more water was used or less than ordinarily, during the time the meter is off. After the meter is repaired it has to be sent back to the premises, where taken off and reset. Two trips are required for each meter, that has to be repaired; one to take it off, the other to reset.

Were the meter the property of the water department, when a meter would be found out of order, it would be taken off and in its place a meter in perfect condition would immediately be set, in place of the by-pass, as at the present time. These defective meters would then be brought to the shop to be repaired and put in stock and used to replace others, when they become out of order. Fully one-half of the teaming expense could thereby be eliminated.

Competition in the meter business has brought out many makes and styles. The scramble for business has reduced the prices of meters, as well as brought on the market several makes of meters of cheaper construction. Some of these are not perfect in workmanship and trouble is experienced in repairing these meters, as the parts are not standard. Some manufacturers of meters change the style or model of their meters too often. This makes it necessary to add to our stock of repair parts. It has also been found impossible to get repair parts from the manufacturers for some of their models, making it necessary for the department to condemn the consumer's meter and requesting him to buy a new model.

The department, with no specifications as to the style and quality of meters, has, therefore, received meters of practically all makes. Experience has shown, that, sometimes, some makes of meters will pass splendid tests, when being tried in the shop but after they are in service a short time, they do not retain their accuracy on a par with other standard makes, requiring considerably more attention and causing innumerable troubles. Under the present arrangement, it is a difficult matter for those in charge to bar or condemn meters of such nature.

If the department controlled the purchase of the meters, this condition could not prevail. At the present time it is necessary to carry about $10,000 worth of repair stock on hand at all times, in order to repair the 34 different styles of meters in service. The department, by controlling the buying of meters, would confine itself to a few standard makes, judged by their merit and render it unnecessary to carry such a large amount of stock, as at the present time.

From the large number of meters coming in the shop for repairs it was found that by far the greater number were brought in on account of failure to register, caused by corrosion of the interior of the cast iron outer housing, which in time interfered with the operation of the train gear. The galvanized coating of the casing apparently afforded but little protection as meters brought into the shop after a few months of service frequently showed evidence of corrosion. As the water meter is the property of the consumer and must be maintained by him, meter repair bills are not looked upon with particular favor by the consumer. The department, therefore, decided to prohibit the use of cast iron meters of the sizes of ⅝ to 2 ins. in diameter, feeling that in so doing a large percentage of repairs would be eliminated. With this in view the rules were changed so that hereafter meters of the sizes of ⅝ to 2 ins. inclusive must be of the best bronze composition; meters above the size of 2 ins. to be of the best grade of grey iron. The difference in the first cost between a ⅝-in. bronze and an iron meter does not exceed $1. Bronze meters are cleaner and more sanitary than iron meters, are less apt to get out of repair and will serve to make water meters more popular. Unfortunately this change was not made at a time when there were but few meters in service.

In order to raise the standard of the meters being offered for installation, a resolution was passed by the common council, that meters must be of such a type or types, as shall be approved by the water department. The construction of meters to be as follows:

The outer casings of all meters of the sizes of ⅝-in. to 2 ins., inclusive, shall be made of a good grade of bronze composition and finished with a bronze coating to give a uniform appearance. Cast iron frost bottoms with interior surface brass lined will be permitted. The outer casings of all meters in excess of 2 ins. shall consist of a good grade of close grained gray iron and shall be painted or coated with a good water-resisting paint or coating. The measuring chambers and gear trains for all meters shall be made of good wearing, non-corrosive hard bronze. Discs and propeller wheels shall be made of the highest grade of vulcanized hard rubber.

Experience has proven that an all brass meter is a great deal more satisfactory than an iron casing meter and the registration of a brass meter is not impaired by rust, as was found in many of the iron casing meters, even though thoroughly galvanized. As most of the outer casings of meters in service are made of iron, it will be the policy of this department to replace the old iron parts with brass instead of iron, whenever found necessary, so that, eventually, the iron casing meter will become obsolete.

The different size services ranging from ⅝ in. to 12 ins. and the various uses of water makes necessary different styles of meters. The disc type of meter, which is the most modern and generally approved meter for ordinary domestic supplies, is made in sizes of ⅝-in. to 4-in. inclusive. It is proving successful on these services up to 2 ins. The 3-in. and 4-in. disc type meter should be discouraged.

In large buildings and manufacturing plants, where a 3-in. or larger service is required, a meter of current or compound type should be installed. The current type meter, which is made in sizes from 3 ins. and larger, is designed, primarily, to handle large volumes of water with a minimum loss of pressure. They are so constructed that the working parts can be inspected or taken out and repaired without removing the entire meter. They, also, retain their accuracy remarkably well and the expense of maintenance is considerably less than that of the old plunger type of which we have quite a number in service. It is the best suited meter, where large volumes of water are constantly used, such as in large manufacturing establishments, standpipes, etc.

In cases where small streams are likely to be used, the best meter would be a compound type. These meters are designed to handle large volumes of water and will also register on small flows accurately. They are made in sizes of 2 ins. and up. A number of these have been recently installed in the city of Milwaukee and have proven very satisfactory. The installation of compound meters should be encouraged in large flat buildings, office buildings, factories, etc., as they insure the maximum revenue, where there is a great fluctuation in the amount of water used.

The Pacific Coast Society of Engineering Contractors has been incorporated in California. The directors are L. G. Atkinson, G. F. Pennebaker, F. A. Hudson, F. C. Ayars, G. A. Rogers, L. G. Garnsey, W. W. Birer, E. T. Sherer, C. L. Hyde, W. S. Russell, W. M. Ledbetter. No capital stock.

Leaky Check Valve Between Public and Contaminated Industrial Water Supplies at Circleville, O., Causes Typhoid Outbreak.

Circleville, Ohio, is a city of 7,000 inhabitants. It is the county seat of Pickaway County and is situated on the Scioto River 25 miles downstream from the city of Columbus. The water supply of Circleville was installed in 1887 and is owned by the Circleville Water Supply Co. The supply is obtained from an infiltration gallery in the valley of Big Darby Creek, about 1½ miles from the city. The water from the gallery flows to a tight collecting well from which it is pumped to the distribution system. Thorough examinations at various times by the State Board of Health have indicated that the supply is of good quality from a sanitary standpoint and safe for domestic use.

The Circleville board of health was alarmed by the large number of cases of typhoid fever reported during January, 1914, and requested aid of the State Board of Health Investigations were made by the Divisions of Sanitary Engineering and Communicable Diseases and, based upon the findings of these divisions, an article relating to the outbreak was prepared by W. H. Dittoe and Frank G. Boudreau, directors, respectively, of the two divisions, and published in the Bulletin of the Ohio State Board of Health for July, 1914. The following information is taken from that article.

During the entire year of 1913 only 7 cases of typhoid were reported in Circleville. In January of 1914, 14 cases were reported and in February there were 25 additional cases. At the time of the investigation 43 cases had been reported.

The epidemic was characterized by a number of very mild cases, which occurred for the most part, during the beginning and end of the outbreak. A number of severe cases occurred during the height. Besides the two patients who died, five suffered from various complications.

There are a large number of wells in Circleville which are not being used for domestic purposes. A majority of them are of a type that easily admits of pollution, so it is wise to maintain the public confidence in the public water supply in order that they may not turn to private supplies of a doubtful nature. There are also a large number of cisterns in Circleville, but they are not used for drinking purposes. Following is a statement of the supplies used by those stricken with the fever:

Number of patients who used city water only	31
Number of patients who used well water only	1
Number of patients who used city and well water	11
Total	43

Plotting these cases on a map of the city showed that with one exception the cases were located south of a line dividing the city into northern and southern portions. There was no other peculiarity of grouping.

RESULTS OF THE INVESTIGATION

The facts to be taken into consideration are briefly these: Circleville is a city whose typhoid rate is exceptionally low. Only one outbreak of typhoid fever has occurred within the last five years, and that one comprised only seven cases. The present outbreak began on Jan. 9, comprised 43 cases and two deaths, and terminated on Feb. 21. There was no peculiar age grouping or occupation distribution, but the ratio of males to females was much higher than usual. The cases were practically restricted to the southern part of the city, and no relation to milk supplies could be found. Numerous investigations and sanitary analyses have shown the public water supply to be of good sanitary quality as a general rule.

No clue was found leading to any satisfactory explanation of the situation until Jan. 31, when the local board of health learned of the existence of a connection to the distributing system of the city through which water was furnished to the American Strawboard Co. for fire protection and industrial use. It was found that the company also maintains an intake in the Scioto River from which a regular supply for industrial use and fire protection is derived. The city connection has been maintained as a safeguard in case of a shut-down or failure of the private supply. The water from the city mains and from the private supply enter the same system of pipes connecting with the sprinkler system within the factory installed for fire protection. The Scioto River from which the private supply is obtained is seriously polluted, receiving the entire discharge of sewage from the city of Columbus at a point 25 miles above the intake. It is, therefore, evident that the introduction of such a supply into the water mains of the city would result disastrously.

A complete investigation of the connection between the distributing system of the city and the distributing system of the American Strawboard Co. revealed the existence of a gate valve owned by the Circleville Water Supply Co. and a gate valve and a check valve owned by and on the property of the American Strawboard Co. Normally both gate valves had been permitted to remain open and the check valve was depended upon to prevent passage of water from the American Strawboard Co. to the city mains. Upon examination it was found that the check valve was in a leaky condition and was permitting a rather large volume of polluted water to enter the mains. Inasmuch as the pressure maintained in the mains of the strawboard company frequently exceeds that on the distributing system of the city, the passage of polluted water into the mains must have been a rather regular occurrence. This check valve was removed and repaired and on Jan. 31, under an order of the local board of health, the gate valve of the strawboard company was tightly closed. Immediately following this, the mains in the southerly portion of the city were thoroughly flushed, this operation requiring several days.

Samples from a number of points on the distributing system in the southerly portion of the city were collected Feb. 2, after the gate valve had been tightly closed and the mains flushed to a considerable extent. The results of examination of these samples show that the water supply in the distributing system was of safe quality. Unfortunately no samples from the distributing system in the vicinity of the strawboard works were obtained prior to the closing of the gate valve.

The existence of this private connection through which sewage contaminated water could be introduced into the water mains supplying the southern portion of the city explains all the observed features of the outbreak. This theory is further substantiated by the fact that no more cases of typhoid fever were reported after the period from Feb. 12 to Feb. 21, which may be accepted as the longest period of incubation of typhoid fever. The dates of onset are only approximately correct, and it is quite possible that in some cases the real, preceded the supposed, onset by several days. Again it is possible that the infection persisted in the mains for several days after the contaminated river water ceased flowing into the mains. The outbreak was sudden in onset and decline, the distribution in the southern portion of Circleville showed no peculiar grouping, and all other possible sources other than the public water supply were eliminated. The conclusion, therefore, may be justly drawn that the outbreak of typhoid fever during January and February was due to a contamination of a portion of the public water supply by raw water from the Scioto River, through a private connection on the premises of the American Strawboard Co. This outbreak should prove a warning to municipalities to preserve the integrity of public water supplies, and to forbid private connections with contaminated sources except under exceptional circumstances and when under rigid supervision of the proper authorities.

Some Rules for Water Utility Operation in Oregon.

In May of this year the Railroad Commission of Oregon issued certain rules, orders and regulations relating to standards of quality, pressure and other service conditions of water and other utilities in the State. The following paragraphs, pertaining to water utilities, are quoted from that circular:

Testing Facilities. Each utility shall provide such laboratory, meter-testing shop and other facilities as may be necessary to make the tests required by these rules. All tests made by any utility under these rules shall be carried out in a manner and at such places as may be approved by the Commission, and the apparatus and equipment used for these tests shall be at all times available for the inspection or use of any member or authorized representative of the Commission.

Records of Tests and Meters.—A complete record of all tests of quality, service, or meter accuracy as made under these rules, shall be kept by each utility accessible to the public during business hours at the principal office in the town or city where the service is furnished, or at such other place as the Commission may designate. The record so kept shall contain complete information concerning each test, including the date and hour when the test was made, the name of the inspector conducting the test, the number of any meter tested and its capacity, the point at which pressure or other tests were made when not made at the regular testing laboratory of the utility, the results of the tests, and such other data as may hereinafter in these rules be specially required, or as the Commission may from time to time require, or as the utility making the test may deem desirable.

Whenever any service meter is tested, the original test record shall be preserved, indicating the information necessary for identifying the meter, the reason for making the test, the reading of the meter before being disturbed, and the accuracy of measurement, together with all data taken at the time of the test, in sufficiently complete form to permit the convenient checking of the methods employed and the calculations.

A record shall also be kept, numerically arranged, indicating approximately when each meter was purchased, its size, its identification, its various places of installation and removal, and the dates and general results of all tests.

Meter Testing.—Every meter hereafter installed for measuring water to any customer shall be tested and if necessary repaired and adjusted by the utility installing it before being placed in use; and every meter tested (except water meters installed underground) shall [illegible] be stencilled, giving the date of test, which tag, label or stencilled mark shall not be defaced or removed until a subsequent test shall have been made.

Meter Testing on Request of Customer.—Each utility shall, at any time when requested by a customer, test the accuracy of the meter in use by him free of charge, provided such meter has not been tested by the utility or by the Commission within the period of one year immediately preceding the request.

Any customer may at any time make application to the Commission for a test of his meter and shall deposit with the Commission a fee for said test as fixed in these rules. Such fee shall be returned to the customer by the Commission, and the amount thereof paid by the utility to the Commission, if the meter is found to be fast in excess of two per cent.

Adjustment of Bills for Meter Error.—If on test of any meter, for any cause, either on removal from or while in service, it shall be found fast beyond the limit above specified, the utility shall refund to the customer such percentage of the amount of the bills of the customer for the period of three months just previous to such test of the meter as the meter shall have been shown to be in error at the time of said test. If the meter is found not to register or to register less than 50 per cent of the actual consumption, an average bill may be rendered to the customer by the utility, subject to the approval of the Commission.

Deposits and Meter Rentals.—Any utility may require from any customer or prospective customer a deposit on account of current bills (1) in the case of customers whose bills are payable in advance, not to exceed an estimated 30 days' bill; (2) in the case of customers whose bills are not payable in advance, not to exceed the estimated 60 days' bill of such customer. Interest thereon, at the rate of six per cent per annum, payable annually or upon the return of the deposit, shall be paid by the utility to each customer making such deposit, for the time such deposit was held by the utility and the customer was served, unless such period of time be less than three months.

No utility may require from any customer or prospective customer a deposit to pay any part of the cost of installation, except under rules and regulations approved by the Commission and set out in the published schedules of the utility.

No rental shall be charged by any utility for any meter installed by it, which is used by the utility as the basis for the rendering of bills.

Interruptions of Service.—Each utility shall keep a record of all interruptions of service upon its entire system or major divisions thereof, including therein a statement as to the time, duration and cause of such interruptions. Such record shall be open at all times to the public inspection, and the Commission may at any time require from the utility a copy thereof.

Complaints.—Each utility shall make full and prompt investigation of all complaints made to it by its customers, either directly or through the Commission, and it shall keep a record of all complaints which shall show the name and address of complainant, the date and character of the complaint, and the adjustment or disposition made thereof. The information contained in such record shall be furnished the Commission upon its request.

Purity of Supply for Domestic Purposes.—Each utility delivering water for domestic purposes shall furnish a supply which shall at all times be free from injurious physical elements and disease-producing bacteria, and shall cause to be made such tests and take such precautions as will insure the constant purity of its supply. A record of all tests and reports pertinent to the water supply shall be kept.

Adequate Pressure Required.—Each utility shall always maintain sufficient pressure at the end of its services to supply adequately each customer connected, except customers who have specifically agreed to permit interruptions of service.

Meter Testing Equipment.—Each water utility supplying 300 or more customers through metered service shall install and maintain a meter tester of a type satisfactory to the Commission.

Installation Tests.—Each meter shall be tested for accuracy before its initial installation, and no meter shall remain in service longer than three years without being retested.

Allowable Limits of Variation of Meters.—No meter shall be installed or replaced in service if it registers more than 102 per cent or less than 94 per cent of the water passed on full capacity, or more than 102 per cent or less than 85 per cent on 1/16 capacity.

Fees for Tests.—The following fees shall be paid by any customer applying to the Commission to make test of his meter:

For each meter not exceeding 1-in. capacity..	$2
For all other meters..........................	5

Pressure Surveys.—Every water utility supplying over 300 customers shall have a recording pressure meter which shall be kept in continuous service on the system at some point approximating the elevation of the center of distribution of the district served, and shall indicate on the graphic records such elevation and causes of extreme variation in pressure. Such records shall be open for public inspection.

[illegible] 1, 1914.

The city of Athens, Ga., is about to try the commission form of government. The bill providing for it has passed both legislative branches and will without doubt be signed by the governor.

Chemical Standards for the Hygienic Purity of Public Water Supplies in Montana.

The public water supplies of Montana are taken from a variety of sources. The purpose of this article, which is taken from a paper by Prof. W. M. Cobleigh before the Illinois Water Supply Association, is to show the differences in the composition of the water from these sources, and with these data as a basis to draw conclusions concerning chemical standards of purity suitable to the various waters. The discussion is of great value in illustrating the difficulty of fixing standards suitable to general requirements and conditions.

In the mountainous portions of the State, city water supplies are taken from streams above human habitation as a rule. These represent the purest waters found in the State and the average of the analyses of 13 of these water supplies is tabulated below. These samples were collected during the fall and winter months when the organic content was the lowest

	Parts per million.
Solids	99.1
Free ammonia	.0152
Albuminoid ammonia	.028
Nitrogen as nitrites	None
Nitrogen as nitrates	.102
Cl.	Absent in all samples

Shortly after the above results were obtained the writer attempted to use them as a partial guide in passing opinions on the sanitary qualities of water supplies taken from the Yellowstone River at various points where there was some reason to suspect that the water was a menace to health. The results of analyses at these points on the river were much higher than could be accounted for by the amount of sewage known to be entering the river. Consequently, samples were then taken from the Yellowstone River above the town of Gardiner and above the mouth of the Gardiner River.

This is a point on the Yellowstone River above all sewage contamination and it was expected that the analyses would give results similar to other mountain streams. That this was not the case is shown by a typical analysis tabulated below:

	Parts per million.
Solids	150.00
Free ammonia	.25
Albuminoid ammonia	.09
Nitrogen as nitrites	.0045
Nitrogen as nitrates	.09
Chlorine	11.50
Oxygen consumed	1.4

This analysis was checked by taking other samples and it was learned that the Yellowstone River at points above sewage contamination contains more free ammonia and nitrites and at times more albuminoid ammonia than it does at the intakes of any of the cities using the water, all of which are located at varying distances below sewage outlets. In this connection it should be stated that the mountain streams flowing into the river below Gardiner contain no nitrites and only a small amount of free ammonia. The dilution, therefore, is a factor in decreasing nitrites and free ammonia.

The results so far point to the conclusion that the river water contains more free ammonia and nitrites during the winter months when most of the water in the river comes from Yellowstone Lake than it does in the spring and summer when the flow is increased by melting snow. The varying character of the water above sewage contamination and the fact that the water normally carries substances usually contributed by sewage makes it very difficult to give chemical data obtained at city intakes any diagnostic value. While it is evident in this case that the free ammonia and nitrites do not have their origin in sewage, still the exact source has not been traced so far as the writer knows. It is possible they are developed by chemical action in the geysers and springs of the Yellowstone National Park.

Water of still a different character is found in Northern Montana. The chief sources for city use are the Milk and Missouri Rivers. The water of these rivers are high in organic matter, which is not derived from city sewage. The organic content increases as the rivers flow to the eastern part of the State, where outcrops of lignite coal frequently occur. The effect of the coal is shown in the following analysis of a spring in a sandstone strata which is underlaid with lignite coal. The spring is far from any source of contamination:

	Parts per million.
Solids	1880.0
Free ammonia	.186
Albuminoid ammonia	.28
Nitrites	None
Nitrates	Trace
Oxygen consumed	6.35
Chlorine	5.1

Water of this character flows into both the Missouri and Milk Rivers at numerous places. It is evident that the chemical data on waters of this kind cannot be compared with the data reported on the pure mountain streams.

Another source of water for city use in Montana is the deep well. The following is a typical analysis. The well was drilled, cased and is 610 ft. deep. It is located far from any source of contamination.

	Parts per million.
Solids	1008.0
Free ammonia	.235
Albuminoid ammonia	.037
Nitrogen as nitrites	.008
Nitrogen as nitrates	Trace
Oxygen consumed	1.25
Chlorine	10.70

It is evident that there is a great variation in the organic content of the surface waters in Montana, known to be free from sewage contamination. This variation is so great that it is impossible to use a universal chemical standard of purity for all waters of the state. However, it has been possible by carefully studying the qualities of the waters from various sources to adopt local standards of purity, which are very necessary to use in connection with bacteriological data

Notes on the Maintenance and Inspection of Steam Boilers.

The appended notes on the maintenance and inspection of steam boilers are taken from a paper by Mr. H. A. Baumhart of the Hartford Boiler Inspection and Insurance Co. of Cleveland, O., before the latest annual meeting of the Ohio Society of Mechanical, Electrical and Steam Engineers:

Great progress has been made in steam boiler design and construction during the past twenty years. In fact, there are but few, if any, boiler manufacturers who would today construct a boiler for a given pressure without taking into consideration safe rules as adopted by some of the states and municipalities or as recommended by good steam engineering practice.

The subject of a proper retiring age for old steam boilers is one which comes frequently to light and is a most fruitful source of controversy. Of course there can be no question as to the propriety of condemning to forced retirement those boilers whose diseases of one sort or another have reached the chronic stage, and are no longer curable, but there is at once the basis for a deal of argument when an inspector approaches the owner of a boiler with the statement that it must be replaced because of old age, especially if it is known to have all the apparent qualifications, except youth, for many additional years of service. In the past many curious properties have been attributed to old boilers; one of the most interesting was the notion that they could not explode violently. It was supposed that an old boiler would merely rupture, allowing the pressure to be relieved much as if the safety valve had opened. This idea was definitely disapproved many years ago along with many other fallacies and much popular mystery concerning boiler explosions.

We all know that steel used in boiler construction will deteriorate with use. It undergoes a slow loss in strength and ductility. The process is hastened by the presence of a moderate excess of phosphorus. Deterioration of this character cannot be determined by the usual method of inspection. It is, therefore, difficult to state definitely at what time in service the boiler should be abandoned. It places a great responsibility upon the inspector. He must consider, in addition to the general appearance of the boiler, the conditions under which it has operated, and he must also make allowance for defects in the material which are not visible. Experience shows that boiler plate, subjected to the high temperature of the furnace, does deteriorate to the extent that after about 20 or 25 years' service, the boiler should (if we follow the motto "Safety First") be taken out of service. Because an inspector reports that he considers the boiler unsafe for further use, it does not imply that he can predict the day and the hour when it will explode; it does mean, however, that the factor of safety is too low, and to continue the boiler in service for any considerable length of time presents a hazard too dangerous to be undertaken.

No mysterious agency enters a boiler to cause an explosion. A steam boiler explodes or fails from one cause only, and that is the fact that the boiler or the part which fails could no longer resist the strain placed upon it. The applied pressure may be sufficient to rupture a sound boiler, or the boiler may have reached such a state of deterioration that the ordinary working pressure becomes an overpressure. In the majority of serious explosions, the primary cause was due to the lap seam form of construction.

Many accidents are due to overheating of the shell plate or tubes. The overheating may result from various causes. The common cause is due to a deposit of scale, sediment, oil, or a combination of oil and scale on the internal surfaces. The presence of oil or grease, although in moderate amounts on the internal surface of a shell plate or tube, is a dangerous condition, and, unless removed promptly, will result in a damaged or possibly an exploded boiler. Great care should be exercised to prevent oil entering a boiler.

By referring to the records of the Hartford Steam Boiler Inspection and Insurance Co., I find the data given in Table I on boiler explosions for the United States during 1911-13.

TABLE I.—DATA ON BOILER EXPLOSIONS IN UNITED STATES, 1911-13.

Year.	Boiler explosions.	Persons. Killed.	Injured.	Total.
1911	499	222	416	638
1912	537	278	392	670
1913	499	180	369	549

The annual property damage is estimated at $500,000. The above records include boiler accidents of every description, some of which were of minor importance. This record is not an alarming condition and, in fact, we are very much encouraged because of the great interest now taken by engineers everywhere insisting upon proper construction and inspection of steam boilers. In many localities compulsory inspection is required by law. The situation therefore as regards safety is gradually improving. We have not yet reached the stage of no boiler accidents. Such a condition will never exist. There will be boiler explosions as long as steam boilers are used as a unit of power. We have not yet reached the stage of perfection in the manufacture of material, neither have we reached perfection in the art of boiler design. We learn as much from the result of failure as from the result of success. Further experiences in the operation of steam boilers may teach us that some radical change in design or character of the material is required to make boilers that will not explode under constantly increasing pressure and temperature.

Another condition which must be considered in connection with the safe operation of steam boilers is their care and management. It frequently happens that a boiler explosion, or some other calamity resulting in loss of life, occurs at places where it is least expected. This proves that, in spite of the advanced precautions that science has provided, reliance must still be placed in human intelligence and faithfulness to a large degree and that sometimes these fail.

It is recommended that boiler owners and users provide every possible method of safeguarding human life and property by purchasing only such boilers as have been thoroughly inspected during course of construction and by seeing that they are also inspected during the life of the boilers. Fortunately in the State of Ohio such conditions now prevail. It also devolves upon the owner or user of a boiler to select only such persons as are fitted to care for them.

Economic Size of Sand Filter Beds.

While the slow sand filtration process unquestionably is on the wane in this country the following notes on the economic size of sand filter beds are useful in making estimates of cost for comparison with the cost of works for the utilization of the rapid sand process. The proper size of beds is a question of economical construction. The larger the beds the less the cost per acre. Covered beds, which are generally used, vary in size from 0.4 to 0.8 acres.

The following calculation from an article on the purification of public water supplies, by C. H. R. Fuller, published in the latest issue of Applied Science, is of assistance in determining the economical number and size of beds. The cost of a filter may be estimated as made up of two items, (1) a portion proportional to the area, which would include cost of bottom, filling small drains, covers, and end walls, assuming basins rectangular and placed side by side, and (2) a portion nearly independent of the size, such as cost of piping, valves, valve chamber, division walls, etc.

Let c = Cost of first portion per acre,
and C = the cost of the latter portion per filter.

If q = area of one filter
n = number of filters
A = Total net area required.

Then, assuming one filter in reserve

$$n = \frac{A}{q} + 1 \quad \ldots\ldots\ldots\ldots (1)$$

The total cost is

$$K = Cn + cnq \quad \ldots\ldots\ldots\ldots (2)$$

$$= C\left(\frac{A}{q} + 1\right) + cq\left(\frac{A}{q} + 1\right)$$

$$= \frac{CA}{q} + C + cA + cq \quad \ldots\ldots\ldots\ldots (3)$$

We then have $\dfrac{dK}{dq} = \dfrac{CA}{q^2} + c$

When for a minimum cost

$$q^2 = \frac{C}{c}A \quad \ldots\ldots\ldots\ldots (4)$$

i. e. the economical area of one filter is proportional to $\sqrt{A}$ and to $\sqrt{\dfrac{C}{c}}$.

The larger the value of "c," the smaller is "q." The values of $\dfrac{\text{"}C\text{"}}{c}$ will hardly be larger than 1/9 or less than 1/16, giving a value of "q" = $\frac{1}{4}\sqrt{A}$ to $\frac{1}{3}\sqrt{A}$. Thus, when $A = 9$ acres, the capacity $q = \frac{3}{4}$ to 1 acre giving 9 to 12 beds. Where $A = 1$ acre, the capacity would be ¼ to ⅓ acre giving 3 to 4 beds. Larger beds than 1 acre are undesirable on account of increased difficulty of operation.

Filter beds are usually rectangular and arranged side by side. It is usual to place them in two rows with a space between for sand washing, regulating houses, etc. The economical proportions of the beds is given by the following formula:

$$\frac{b}{l} = \frac{n+1}{2n}$$

where b = width, l = length, and n = number of beds in a row.

The Application of Geology to the Problems of the Municipal Engineer.

Few municipal engineers of today can fail, after much experience, to realize the importance of practical geology in the construction of their works.

The success of underground water supplies, whether attained by knowledge, trial and error, or by a stroke of luck, is admittedly dependent in each case upon the local geology. In works of sewerage, again, the cost may be largely influenced by the nature of the geological materials met with in the trenches and tunnels, producing as they may do difficulties and expense in excavation, timbering, dealing with ground-water, and modification in the design and construction of the permanent work. The same general principles apply to all deep excavations or foundations that may be carried out by the municipal engineer, whether in the construction of new roads with deep cuttings; sea, river, and retaining walls; bridges, reservoirs, sewage tanks or large buildings. The geology of building stones, road-metals, and similar materials, though in a lesser degree, is of importance in all those branches of municipal work in which they are constantly employed. The following discussion of applied geology in municipal engineering work is from a recent paper by Herbert Lapworth before the annual conference of the Institution of Municipal and County Engineers, held at Cheltenham, England, on June 24-27.

One of the most striking factors that has forced applied geology upon the attention of the engineer has been the number of disputes which have arisen between contractors and municipal authorities concerned almost solely with the "nature of the ground" encountered during construction of public works. A few of these cases have been taken to the courts, some to arbitration, while others have been privately settled. In most instances the cause of the dispute has been due to some misunderstanding as to the "nature of the ground," or, in other words, the geology of the excavations; and almost invariably the ultimate result has been costly.

Frequently this kind of trouble has been the result of providing intending contractors with inadequate or misleading descriptions of the strata, obtained from trial holes or borings. Another cause has been the enforcing under the contract of a type of construction unsuited to the kind of ground unexpectedly encountered in the trenches or foundations.

Quite outside the question of legal disputes, however, serious difficulties have arisen, and heavy expense has been incurred by locating works in bad ground, either through lack of previous geological investigations or by wrong deductions from haphazard exploration, when such troubles might have been avoided in the original location or largely anticipated by investigation with practical geological knowledge.

It is a remarkable fact that this branch of engineering, common as it is to all classes of the profession, and on which so much frequently depends, is so strangely ignored in our own engineering literature, and until recently so little considered in the scientific training of civil engineers. Thus we find in our engineering text-books the most minute mathematical investigations of structures, more or less without relation to the varying geological materials on or in which they may be placed.

As a minor example, retaining walls, the dimensions of which, other things being equal, depend entirely upon the geology, water-bearing nature, and dip of the materials to be supported, are subjected to detailed mathematical analysis which may, in the drawing office, be slavishly applied to practice by arithmetical and graphical calculations, without any knowledge of the ground in which the walls are to be built.

To a limited extent the design of tunnel and sewer sections, for example, in "good" and "bad" ground, are considered in text-books; but no account is given of the many types of material met with underground, under varying conditions; nor of the principles of the occurrence of underground water, the great variation in cost, and the difficulties of construction under different geological conditions. Under "earthwork" we find the usual tables of slopes, prismoidal formulæ, and the like, but practically nothing on the highly important subject of "slips," or treatment of soft, unstable, or "bad" ground.

It may be argued, and rightly, that these are matters of actual practice and experience; but this does not excuse the failure of these treatises to impress the student with the great variation in cost and difficulties of engineering construction in different deposits, and the fact that efficiency and economy in construction might be as much the result of geological knowledge and investigation as of skill and experience in calculation and design. In some degree this void in the engineering text-books is due to the extreme complexity and variation in the physical properties of the geological deposits, which render them incapable of mathematical analysis.

Another inconsistency is sometimes to be seen in the letting of large contracts, where we may have items in the schedule of quantities amounting to only a few shillings, yet the contractor may be wholly ignorant of the character of the ground in which the works are to be constructed, and may, in consequence underestimate by several thousand dollars the cost of excavation.

The causes which have contributed to the scant consideration of practical geology among engineers are not easy to determine. Probably the chief of these is the fact that, whatever geological difficulties do arise during construction, it is always possible to complete engineering works, whether economically or not, without geological knowledge, and the engineer is not likely to be blamed by his employers for what turns up unexpectedly below ground.

Secondly, there is a pessimism among certain engineers, some of whom, though they may be experienced in excavation work, have little time for geology, and see only a comparatively heterogeneous mass of deposits underground in no sort of order, and to be classified broadly into two types—"rock" and "muck." There is also the optimist, who expects the geologist by surface indications alone to prophesy precisely what will be met with in almost every yard of his proposed trench, and in consequence loses faith in the accuracy and practical application of the science.

Thirdly, we have scant consideration of the subject in the engineering and geological text-books. The former have already been discussed. The latter, while containing all the necessary elements for the study of the subject, rarely present the applied or practical side of the science, except on the very broadest lines, and include matter that is unnecessary to learn, and which has rarely any direct bearing upon engineering questions.

In the writer's opinion, every civil engineer should be familiar first with the elements of geology, i. e., the principles of stratification, and formation of rocks and deposits, and the various structures occurring in them as the result of formation, change, weathering, or earth movements. He should be able to recognize and describe correctly all the common solid rocks and unconsolidated deposits. He ought also to understand a geological map as he does a working drawing, and be able to plot approximate geological sections from it, as well as accurate geological records from borings, trial holes, trenches, and tunnels. A training in petrology is desirable for the road engineer; but expert knowledge in this branch of geology usually requires more time than the professional engineer can afford. A thoroughly practical geological training, however, cannot be acquired without considerable experience in the field.

Of equal importance is the subject of hydrogeology This includes the principles of the occurrence of underground water in different geological deposits, the laws of flow through porous and permeable materials, underground water-levels and their fluctuations The mathematical side of this subject has been considered in great detail by foreign scientists and engineers; but so far the quantitative results have been of little direct application to practice, owing to the extreme complexity of underground conditions. Nevertheless, the principles arrived at are of great value in clearing one's ideas, assisting one to anticipate or prophesy, and to deal more rationally with undeground-water problems where a certain amount of local information has already been secured. The value of this science is not only found in questions of water supply, but in the excavation of trenches, cuttings, deep foundations, tunnels, drainage, and the like.

It would be impossible in a paper of this kind to cover the whole of that region in municipal engineering on which practical geology has a direct bearing. The following are merely a few selected instances commonly occurring in practice.

PRELIMINARY GEOLOGICAL INVESTIGATIONS ON IMPORTANT WORKS.

In the majority of important public works it is desirable to ascertain by means of trial holes or borings the nature of the ground in which the works are to be constructed. The object of these investigations may be to determine first, the necessary dimensions of the permanent work and its approximate cost; secondly, the ground-water levels; thirdly, whether any saving may be effected by a change in location, if practicable; and lastly, to enable contractors to make a fair tender for the cost of construction. As regards the latter, it has given rise to discussion among engineers, whether data obtained from trial holes and borings should be shown to tendering contractors, because of the possibility of future claims or litigation. Information of this kind, however, allows a contractor to put in a fairer price, and avoids the necessity of his protecting himself, as he often does, by increased prices for unknown ground. Without such knowledge, a contractor's tender must usually be either unfair to the municipal body or the contractor himself; but there are pitfalls, of course, although the more practical geological knowledge there is brought to bear on the investigation, the less will be the risk.

As regards the selection of sites for boring, etc., it is essential that the number of holes should be sufficient to give reliable information, and the strata must be correctly described. Each hole should be sunk to the full contemplated depth of the excavation, and located on the center-line, or within the limits of, and not outside the work itself.

An interesting example of the result of sinking trial holes clear of the work occurred recently in this country in connection with a tunnel sewer. The trial holes, in order to save expense, were sunk in a railway cutting parallel to the sewer. These proved the ground to consist entirely of sandstone and conglomerate, whereas at least half the length of the tunnel was found to be in marl, the discrepancy being due to the cross dip of the strata. In the sandstone and conglomerate the tunnel was driven without timber, while in the marls close-timbering was required throughout, owing to falls from the sides and roof. An action was brought by the contractor against the public authority on the basis that misleading information had been supplied to him

In the solid rocks the selection of the sites for trial holes, etc., can often be much assisted by a knowledge of the surface geology. This is especially applicable to tunnel work and the like; but the majority of municipal works are founded close to the surface, and the nature of the ground is dependent more on the character and depth of the soils, sub-soils, and surface deposits of the districts. As the detailed structure and arrangement of these surface deposits cannot be foretold with any accuracy from surface indications, trial holes or borings are essential.

WATER SUPPLY.

It is perhaps in questions of water supply, and especially in schemes from underground sources, that a sound knowledge of geology and hydro-geology is essential. The selection of a site for a well or boring, which will yield a satisfactory amount of pure water requires intimate acquaintance with the local geology, the order, arrangement and structure and water-bearing characters of its various strata, both as regards its solid rocks and superficial deposits. Familiarity with hydro-geological principles will also assist the investigator in analyzing more rationally from data in local wells and springs the existing underground water resources of the district, and in considering questions of possible pollution from farms, fields, cesspools, cemeteries, and polluted streams. Similar principles apply largely to water supplies from springs.

Geological considerations are also highly important in the selection of sites for impounding and service reservoirs, dams, and embankments, and in the location and construction of lines of aqueduct, pipes, and tunnels; but these are matters, perhaps, more usually in the domain of the consultant, dealing with large schemes, than in the routine work of municipal engineers.

LINES OF SEWERS AND PIPES.

In the construction of lines and sewers we find many questions arising where applied geology may be of use in location, design, and construction. The most serious difficulties are generally encountered in the unconsolidated deposits of alluvial, glacial, and other recent origin. The beds of clay, silt, mud, running sand and peat, which so frequently occur in these formations, are usually capable of supporting only relatively low pressures, and require great skill and experience in the treatment of cuttings or excavations, being especially liable to slips, and to act as a semi-liquid when their existing stability is disturbed by any excavation within their mass. Occurring as they do in the low lying areas in valley bottoms, estuaries, and mud flats, they are liable to be saturated with ground-water, which further reduces their stability, and adds greatly to the difficulties of engineering construction.

In addition to the obviously water-logged ground formed of these deposits in low-lying areas, many lines of pipes and sewers may be constructed in water-bearing rocks and deposits below the saturation level. The opening out of a deep trench in porous or permeable material below the saturation level has the effect of draining the measures on both sides of the trench, and producing a lowering of the water-levels in the vicinity. This may often give rise to claims for damage to local wells and springs. Any trench excavated below river-level, in a valley bottom and in permeable strata, will tend to collect a large volume of water; and similarly where the saturation level rises from the valley bottom more or less parallel to the ground surface, a deep trench, although distant from the river, may require heavy pumping during construction. In all permeable surface deposits and permeable strata, such as chalk, sandstone, sands, and heavily fissured rocks, in which trenches are to be excavated, it may be often worth while to determine the underground water-levels in local wells, in order to see whether it is possible to fix the levels of the permanent work above them. Local wells have not infrequently been drained dry during the construction of a deep sewer, and sometimes permanently, so that it has been necessary to deepen them in order to restore their water supplies.

In considerations of this kind, hydro-geology has shown that the saturation level is subject to seasonal fluctuations; hence it may happen that a trench excavated in dry ground above the saturation level during one part of the year may be flooded during the winter and early spring seasons when the ground-water is high

Heavy pumping, and consequent lowering of the saturation level in certain sands, peat-bogs and marshes, may also seriously affect the foundations of adjacent buildings, producing cracks and settlement. Cases are on record where the saturation level has been lowered several feet by pumping, and accompanied at a distance of 200 ft. by a settlement in the building of several inches.

In hilly districts, sewers and lines of pipes may require to be constructed in steep side-long ground and scree slopes. Here there is a strong tendency for the surface material to slip into the trench, and great pressures may be brought to bear upon the timbering, and even upon the permanent work, which may be forced down hill and fracture. Such catastrophes have occurred both on lines of cast iron pipes and brick culverts in this country.

In certain types of ground again, such as those formed of steeply digging shales or clay masses, or surface materials upon a highly inclined rock surface, slips are liable to occur, and even the permanent work may require to be strengthened.

Another curious phenomenon is occasionally found in deep trenches wholly in clay deposits, which may appear to be perfectly consolidated and firm during excavation. The weight of the material on both sides of the trench may be sufficient to cause the plastic material to flow, with the result that the timbering must be crushed in and collapse, accompanied by an uplift of the bottom of the trench, and of the permanent work. The writer has known of a length of over 100 yds. of culvert invert, and side walls destroyed in this way. In some of the glacial clay deposits of Lancashire the trench bottoms have been observed to rise between the operations of trimming the trench bottom and bedding the pipes.

In ground of soft clays, mud, silt, running sand or peat, the difficulties of construction are, of course, at their worst. Special precautions must then be taken from the outset in timbering, strengthening the section of the permanent work, draining, and so dealing with the foundations below formation level, as to enable the ground to support the weight of the overlying structure.

The subject is too vast, however, to discuss here; but it may be taken for granted that in questions of this kind, the engineer armed with a sound knowledge of the local underground geology and its relation to the subsiding surface is likely to be more efficient, not only in dealing with lines of pipes and sewers, but with reservoirs, filters, bacteria beds, and other works.

ROADS AND ROAD M[illegible]

In the construction of new roads practical geology may be a very important factor in location, where the choice of a route may be guided by the geology of the district to be traversed. Thus areas liable to slips, or formed of very soft or very hard material, or requiring heavy retaining walls, may sometimes be avoided by deviation. In this country, however, the routes of the few new roads that are now being, or likely to be, constructed, are more or less fixed within rigid limits. Still, even here, practical geology and hydro-geology, when combined with engineering experience, are valuable aids in the excavation and drainage both of the cuttings and the roadbed itself.

DRAINAGE AND IRRIGATION

Proposed Standard Specifications and Recommended Practice for Drain Tile and Tile Drain Construction.

Standard tests and specifications for drain tile have been under investigation since early in 1911 by a special committee of the American Society for Testing Materials. In this study very elaborate experiments have been conducted at Iowa State College and the University of Wisconsin. In its recent report for 1914 the committee announces the virtual completion of its strength tests at Iowa State College and based on them reports: Proposed Standard Specifications for Strength Tests of Drain Tile; Proposed Standard Specifications for Quality of Drain Tile; Proposed Recommended Practice in Design and Construction of Tile Drains.

It also reports further that considerable progress has already been made on a special experimental investigation of absorption, and freezing and thawing tests, and their relation to strength and durability, which the University of Wisconsin is making for the committee. Professor Withey, of that institution, is making the tests, which are being applied both to clay and to concrete tile, of different grades, materials and processes. The completion of these investigations, the collating and study of their results, and the preparation of proposed standard specifications for absorption and other durability tests, and for corresponding requirements for quality of drain tile, to be inserted in their appropriate places in the proposed Standard Specifications for Strength Tests of Drain Tile and for Quality of Drain Tile, constitute the main part of the program for the work of the committee for 1914-15.

The two sets of specifications and the rules of recommended practice follow:

PROPOSED STANDARD SPECIFICATIONS FOR STRENGTH TESTS OF DRAIN TILE.

1. The specimens shall be unbroken, full-size tile. They shall be carefully selected so as to represent fairly the quality of the tile.

2. A standard test shall comprise five individual tests. The result for each specimen and the average of the five shall be given in the report of the test.

3. The materials of the tile shells shall, at the time of testing, be in a thoroughly wet condition, such as may be obtained by covering with sacks kept wet for 8 hours.

4. No test specimen shall be exposed to water or air temperatures lower than 40° F. from the beginning of artificial wetting until tested. Frozen tile shall be completely thawed before artificial wetting begins.

5. Each specimen shall, if practicable, be weighed on a reliable scale just prior to testing.

6. The load shall be applied by any machine or hand method which will apply the load continuously, or in uniform increments not exceeding 0.05 of the total load necessary to break the tile. The tile shall not be allowed to stand any considerable time under load. All solid parts of the bearing frames or bearing blocks shall be so rigid that the distribution of the load shall not be appreciably affected by the deflection of any part. All bearings and the test specimens shall be so accurately centered as to insure in every direction a symmetrical distribution of the loading on each side of the center of the tile.

7. The inspector, in specifying test requirements for drain tile, shall prescribe in advance one of the three following kinds of bearings: sand bearings; hydraulic bearings; three-point bearings.

8. The test results shall be reported in terms of the ordinary supporting strength. This term shall be defined to mean the supporting strength of a tile when the load is applied with such a distribution as to produce a maximum bending moment of $0.20 \frac{RW}{12}$, where W = the ordinary supporting strength, and R = radius of middle line of tile shell, in inches. The ordinary supporting strength shall be obtained by multiplying the test breaking loads, by the following factors: For sand bearings, 1.00; for hydraulic bearings, 1.25; for three-point bearings, 1.50.

The ordinary supporting strength shall be reported in pounds per linear foot.

9. The modulus of rupture shall be calculated from the maximum bending moment prescribed in Section 8 of the formula

$$p = \frac{6M}{t^2}$$

where p = modulus of rupture in pounds per square inch, M = maximum bending moment in shell in inch-pounds per inch of length, calculated as prescribed in Section 8, and t = thickness of tile shell in inches.

Five-eighths of the weight of the tile per lineal foot for sand bearings, or three-fourths for hydraulic or three-point bearings, shall be added to W in computing the maximum bending moment, when such addition exceeds 5 per cent of W.

10. Where sand bearings are used, each specimen shall be accurately marked in quarters, with pencil or crayon lines, prior to the test. Specimens shall be carefully bedded, above and below, in sand, for one-fourth the circumference of the pipe, measured on the middle line of the pipe shell. The depth of bedding above and below the pipe at the thinnest points shall at each place be equal to one-fourth the diameter of the pipe, measured between the middle lines of the pipe walls.

The sand used shall be clean sand which will pass a No. 4 screen.

The top bearing frame shall not be allowed to come in contact with the pipe or with the test load. The upper surface of the sand in the top bearing shall be carefully struck level with a straight edge, and shall be carefully covered with a heavy, rigid, top bearing plate, with lower surface a true plane, made of heavy timbers or other rigid material, capable of uniformly distributing the test load without appreciable bending. The test load shall be applied at the exact center of this top bearing plate in such a way—either by the use of a spherical bearing, or by the use of two rollers at right angles—as to leave the bearing free to move in both directions. In case the test is made without the use of a machine, and by piling on weight, the weight may be piled directly on a platform resting on the top bearing plate, provided, however, that the weight is piled in such a way as to insure uniform distribution of the load over the top surface of the sand.

The frames of the top and bottom bearings shall be composed of timbers so heavy as to avoid appreciable bending by the side pressure of the sand. The frames shall be dressed on their interior surfaces. No frame shall come in contact with the pipe during the test. A strip of soft cloth may be attached to the inside of the upper frame on each side along the lower edge to prevent the escape of sand between the frame and the tile.

11. Where hydraulic bearings are used, each specimen shall be accurately marked in halves, with pencil or crayon lines, prior to the test.

A hydraulic bearing shall be composed of a wooden platen to which is attached, as hereinafter described, a section of rubber hose. The hose shall lie against the tile, and the pressure shall be applied to the hose through the platen.

The platen shall be built of yellow pine, and shall be at least 4 by 4 ins. in section, and at least length shall be the length of the pipe plus 8 ins. One-inch quarter rounds with their convex surfaces facing shall be firmly attached to each edge of one side. The straight portion of this face shall extend at least the length of the pipe, and the platen beyond this length may be cut to the arc of a circle.

Between the quarter rounds shall be laid a piece of 2½-in. hose, which shall be closed in a water-tight manner at each end by clamps. The hose shall contain a volume of water not less than one-half nor more than two-thirds its capacity, when completely distended. This hose may be attached to the platen at either end in any satisfactory manner which will not induce wrinkling when under test pressure.

The test load shall be applied at the exact center of the top bearing, in such a way as to leave the bearing free to move in the vertical plane of the axis of the pipe.

It is recommended that stops be screwed to the platen symmetrical with the point of application of the load, and at a distance apart not greater than the length of the tile plus ½ in. This will help center the load coming upon the pipe.

12. Where three-point bearings are used, each specimen shall be accurately marked in halves, with pencil or crayon lines, prior to the test.

The lower bearings shall consist of two wooden strips having a corner rounded to a radius of approximately ½ in. They shall be straight and shall be securely fastened to a rigid block in a position such that the bearing lines of a cylinder of 24-in diameter laid along the rounded edge of the strips shall be 2 ins. apart.

The upper bearing shall be a wooden block, straight and true from end to end.

The test load shall be applied through the upper bearing block in such a way as to leave the bearing free to move in a vertical plane passing between the lower bearings.

In testing a tile which is "out of straight," the lines of the bearings chosen shall be from those that appear to give most favorable conditions for fair bearings.

PROPOSED STANDARD SPECIFICATIONS FOR QUALITY DRAIN TILE

1. Specimens to be tested shall be selected by the inspector from the tile to be used on the work; these specimens to be selected at the factory, shipping destination, or at the trench location. The tile shall be measured, sounded and examined by inspection. Five specimens of each materially different class noted shall be selected for a test. If in the judgment of the inspector it is necessary either before or after the testing of the specimens, additional specimens may be selected, but in no case shall these additional specimens exceed 1 ft. in length for each 100 lin. ft. of tile to be laid. These additional specimens shall be furnished by the contractor free of charge at the point of selection, provided that, in case the specimens tested meet the specifications, not more than 1 per cent shall be required to be furnished free.

2. Each tile shall be of a cylindrical section, the size being designated by the interior diameter. The average diameter shall not be more than 3 per cent less than the specified diameter. The maximum and minimum diameters of the same tile or average diameters of adjoining tile shall not differ more than 80 per cent of the thickness of the wall.

3. The minimum length of tile shall not be less than 12 ins. In tile 12 ins. or above in diameter, up to 30 ins. in diameter, the length shall not be less than the diameter. Tile above 30 ins. in diameter need not have a greater length than 30 ins.

4. Tile designed to be straight shall not vary from a straight line more than 3 per cent of its length.

5. Tile shall be reasonably smooth on the inside, and free from cracks and checks extending into the body of the tile in such a manner as to appreciably decrease the strength. Tile stood on end and tapped with a light hammer when dry shall give a clear ring.

Tile shall be free from chips or broken pieces which shall decrease its strength or admit earth into the drain. The end shall be regular and smooth and admit of the making of a close joint when properly turned and pressed together.

6. In a standard test, if one or more specimens fall more than 25 per cent below the required strength as specified, the class of tile represented by the failing specimens shall be rejected, and other specimens tested to complete the standard test.

7. (a) Class No. 1B.—No. 1B tile are intended to be suitable for supporting the load in the [illegible] a grade line 5 ft. deep. They shall have minimum average ordinary supporting strengths calculated as prescribed in Section 8 of the proposed Standard Specifications for Strength Tests of Drain Tile, in accordance with the following table:

REQUIRED AVERAGE ORDINARY SUPPORTING STRENGTH FOR CLASS NO. 1B TILE.

Diameter of tile, ins.	Lbs. per linear foot.
10	600
[illegible]	[illegible]
[illegible]	[illegible]
16	900
18	1,000
[illegible]	[illegible]
22	1,100
[illegible]	[illegible]

[illegible] porting the load in the worst material in a [illegible]

The inner surface of the tile shall be free [illegible] from broken blisters, lumps or flakes which are thicker than 20 per cent of the thickness of the tile, or whose diameter is greater than [illegible] and such defects as are allowed shall not be of such nature as to appreciably weaken the tile when laid in the ditch.

The tile shall have minimum average ordinary supporting strengths (calculated as prescribed in Section 8 of the proposed Standard Specifications for Strength Tests of Drain Tile) in accordance with the following table:

REQUIRED AVERAGE ORDINARY SUPPORTING STRENGTH FOR CLASS NO. 1A TILE.

Diameter of tile, ins.	Lbs. per linear foot.
12	900
14	1,000
16	1,200
18	1,300
20	1,400
22	1,550
24	1,700
26	1,800
28	1,900
30	2,000
32	2,050
34	2,150
36	2,250

(3) Class No. 1 Extra A.—No. 1 Extra A tile shall be extra good, and are intended to be suitable for supporting the load in the worst material in a trench having a grade line 10 ft. deep. They shall be either vitrified, salt-glazed, clay tile, or thoroughly seasoned concrete tile, made of the best materials, by the most approved method.

The inner surface of the tile shall be free from defects. The outer surface shall be free from broken blisters, lumps or flakes which are thicker than 16 per cent of the thickness of the tile, or whose diameter is greater than 12 per cent of the inner diameter of the tile, and such defects as are allowed shall not appreciably weaken the tile when laid in the ditch.

The tile shall have minimum average ordinary supporting strengths (calculated as prescribed in Section 8 of the proposed Standard Specifications for Strength Tests of Drain Tile) in accordance with the following table:

REQUIRED AVERAGE ORDINARY SUPPORTING STRENGTH FOR CLASS NO. 1 EXTRA A TILE.

Diameter of tile, ins.	Lbs. per linear foot.
12	1,000
14	1,200
16	1,500
18	1,700
20	2,100
22	2,300
24	2,500
[illegible]	2,600
[illegible]	2,800
[illegible]	3,000
[illegible]	3,200
[illegible]	[illegible]
[illegible]	[illegible]

8. Tile not meeting the above specifications shall be rejected.

[illegible] RECOMMENDED PRACTICE IN [illegible] AND CONSTRUCTION OF TILE DRAINS.

The selection of a class of tile suited to a particular case requires a knowledge of the pressures to which the tile will be subjected. This in turn depends upon the character of the soil and the manner of laying the tile as well as upon the depth and width of ditch. The following is recommended as good practice in design and construction:

1. Three grades of work are recognized, namely, Ordinary, First Class and Concrete-Cradle. Generally the engineer will specify the grade of work required, but in some cases it may be advisable to allow the contractor a choice between using a superior method of laying, or a stronger tile.

2. In Ordinary tile laying the contractor shall shape the bottom of the ditch approximately to fit the lowest one-quarter of the outside circumference of the tile, taking pains to secure an extra firm bearing near the outer edges of the bearing area. In hard material he shall bed the tile in a thin layer of granular earth where, in the judgment of the engineer, it is necessary to secure a good bearing.

After the tile is bedded truly to line and grade, the contractor shall carefully place the earth around and over the tile by hand to the depth of at least 1 ft. over the tile, using shovels or other suitable tools to work the earth filling down the sides, and underneath the tile so far as practicable.

Whenever the ordinary supporting strength of the tile, as determined by actual tests, and calculated as prescribed in Section 8 of the proposed Standard Specifications for Strength Tests of Drain Tile, is 50 per cent or more in excess of the strength specified, the bottom of the ditch need not be shaped to fit more than the lower one-eighth of the outside circumference of the tile.

3. In First-Class tile laying in hard material, the contractor shall shape the bottom of the ditch approximately to fit the lowest one-quarter of the circumference of the tile, taking pains to secure an extra firm bearing near the outer edges of the bearing area. Upon the concave surface so prepared the contractor shall spread a layer, 1 to 2 ins. thick, of pulverized soil, or sand free from pebbles larger than ¼ in. diameter, and shall firmly bed each tile truly to line and grade thereon.

Where the bottom of the ditch is so wet and soft as to enable the thorough bedding of the lowest one-quarter circumference of the tile without the use of the layer of pulverized earth or sand, and still is firm enough to afford good, safe support to the tile and its load of ditch filling, the engineer may authorize the omission of the layer of granular material, but such authorization shall not excuse imperfect bedding.

The space between the tile and the bottom and sides of the ditch shall be filled with selected earth, thoroughly tamped as fast as placed, up to the level of the top of the tile. The side filling shall be carried up as rapidly on one side of the tile as on the other.

The tile shall then be covered by hand with earth to a depth of at least 1 ft. above the top of the tile.

No tile laying shall be considered as First-Class unless the laying and tamping of each tile are watched and directed by an inspector kept constantly on the work for that purpose.

4. Two grades of Concrete-Cradle tile laying shall be recognized, one for solid material and the other for yielding material.

(a) Solid Material.—Solid material shall be defined as that which is as solid as average, firm, clay sub-soil. Concrete-Cradles, Solid Soils, shall be made as follows:

The contractor shall shape the bottom of the

TABLE I.—MAXIMUM LOADS ON DRAIN TILE AND SEWER PIPE FROM ORDINARY DITCH-FILLING MATERIALS—ORDINARY SAND, 120 LBS. PER CU. FT.; THOROUGHLY WET CLAY, 120 LBS. PER CU. FT.

Loads in pounds per linear foot.

Height of fill above top of tile, ft.	Breadth of ditch a little below top of tile. 1 ft. Sand.	1 ft. Clay.	2 ft. Sand.	2 ft. Clay.	3 ft. Sand.	3 ft. Clay.	4 ft. Sand.	4 ft. Clay.	5 ft. Sand.	5 ft. Clay.
2	180	190	410	420	650	660	890	900	1,110	1,130
4	270	300	710	750	1,170	1,220	1,640	1,690	2,100	2,150
6	310	360	910	1,000	1,590	1,680	2,270	2,400	2,970	3,100
8	340	400	1,070	1,190	1,910	2,070	2,820	3,000	3,720	3,930
10	350	420	1,180	1,330	2,180	2,400	3,260	3,510	4,380	4,680
12	360	440	1,250	1,440	2,400	2,670	3,650	3,990	4,980	5,340
14	360	440	1,310	1,530	2,570	2,890	3,990	4,380	5,490	5,940
16	360	450	1,350	1,600	2,710	3,090	4,260	4,740	5,940	6,480
18	360	450	1,380	1,650	2,820	3,250	4,490	5,050	6,330	6,930
20	360	450	1,400	1,690	2,910	3,390	4,700	5,340	6,660	7,410
22	360	450	1,420	1,720	2,980	3,510	4,880	5,570	6,960	7,800
24	360	450	1,430	1,740	3,050	3,600	5,010	5,780	7,230	8,160
26	360	450	1,440	1,760	3,090	3,680	5,150	5,970	7,460	8,490
28	360	450	1,440	1,780	3,120	3,750	5,240	6,120	7,670	8,760
30	360	450	1,440	1,790	3,150	3,800	5,340	6,280	7,830	9,030
Infinity	360	450	1,450	1,820	3,270	4,090	5,820	7,280	9,090	11,370

ditch to fit approximately the lowest one-fourth of the circumference of the tile. Upon the concave surface so prepared there shall be spread at least 2 ins. of soft concrete, stiff enough to sustain the weight of the tile, and the tile shall be firmly bedded truly to line and grade thereon.

The space between the tile and the bottom and sides of the ditch shall then be thoroughly tamped or spaded full of soft concrete, up to a level one-quarter of the diameter of the tile above the mid-height. The thickness of the concrete at any point shall not be less than 2 ins.

Each joint shall be promptly cleaned on the inside of the tile, as soon as the concrete is in place for that joint.

The concrete used in this method of strengthening tile shall be made of 1 part Portland cement and 8 parts of gravel, or 1 Portland cement, 5 parts sand, and 8 parts broken stone. No pebbles or stone shall exceed in size 1 in. less than the thickness of the concrete.

(b) Yielding Materials.—Yielding materials shall be defined as including all material not solid, as defined above.

Concrete-cradles for yielding material shall be designed by the engineer to carry safely to the soil foundations both the vertical load on the tile from the ditch filling and a side thrust at the mid-height of the tile, such as would exist if the tile were cracked at the top, bottom and each side. The thickness of the concrete at the lowest part of the bottom of the tile shall be at least one-eighth, and on each side at the mid-height at least one-fifth the internal diameter of the tile, and the side concrete shall extend about one-quarter of the diameter above the mid-height of the tile. Each joint shall be promptly cleaned on the inside of the tile as soon as the concrete is in place for that joint.

The concrete used in this method of strengthening pipe shall be made of 1 part of standard Portland cement and 5 parts of good, coarse, clean gravel, or 1 part of standard Portland cement, 3 parts clean, coarse sand, and 5 parts broken stone. No pebbles or stone shall exceed 2½ ins. in greatest diameter, nor exceed 1 in. less than the thickness of the concrete.

5. Tile in trench shall not be subjected to freezing weather during construction without a sufficient depth of cover to prevent cracking.

6. In Table I are given the approximate values in pounds per linear foot of the ordinary maximum loads on drain tile and sewer pipe from common ditch-filling materials, as determined by tests* at Ames, Iowa, and Boston, Mass., and by study of the detailed data of about 90 actual tile drains and pipe sewers, part sound and part cracked.

It is recommended that for clay and all common material except sand and loam, the values under clay be used, and for sand and loam, the values under sand.

6. It is recommended that where tile are to be laid according to the description for the Ordinary method, a factor of safety of 1½, applied to the average strength, shall be used when the results of the tests are reported in terms of the ordinary supporting strength, calculated as prescribed in Section 8 of the proposed Standard Specifications for Strength Tests of Drain Tile, and loads estimated according to Table I.

7. Where the tile are to be laid in accordance with the method denominated First-Class, in consideration of the increased support furnished by the improved foundations, the nominal factor of safety to be employed shall be 1¼, applied to the average strength, and with loads estimated according to Table I.

8. In this case it is intended that the concrete-cradles shall furnish the strength necessary to carry the load from the ditch filling. It is recommended, however, that only Class No. 1 A or Class No. 1 Extra A tile shall be used in this case.

*For both tests and the data of actual drains and sewers, see the Report of Committee C-6 on the Investigations on Drain Tile, American Society for Testing Materials, published as Bulletin No. 36, Iowa Engineering Experiment Station.

Construction of Timber Logging Flumes.

The construction of lumber flumes is a task which the professional engineer has in the past been seldom called upon to direct but one which, nevertheless, involves considerable careful engineering and one which in the future promises to become more often an engineer's opportunity. We give here, therefore, from a recent Bulletin of the Forest Service, U. S. Department of Agriculture, some data on flume construction which is valuable for record.

The V-shaped wooden flume is held to be superior to the box or square-sided form, because it requires less water and, on the average, less repairs than the other type, is better adapted to act as a slide on steep grades, and offers fewer chances for jams. Concerning a third type, the "sectional" metal flume, semicircular in form, the prediction is made that it will eventually come into wide use. Such a flume is strong and light, and can be quickly taken apart and transported from one place to another to be set up again. When building flumes a good plan,

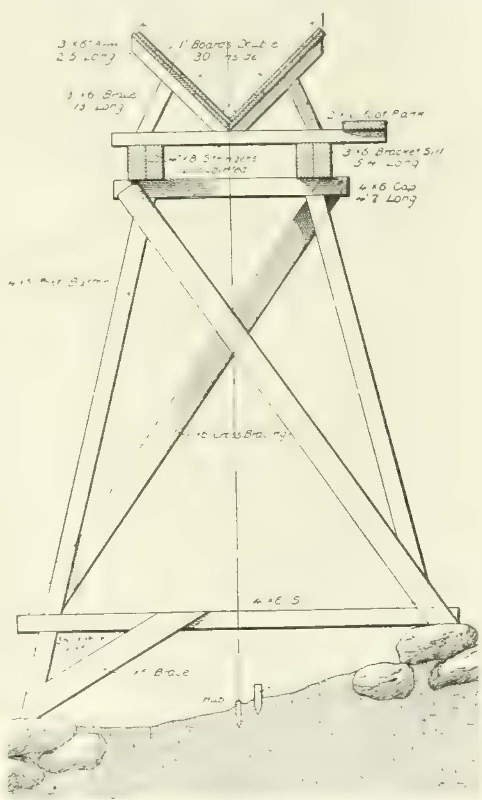


Fig. 1. End View of V-Shaped Flume Showing Method of Construction and Double Lined Box.

says the bulletin, is to erect a small sawmill at or near the upper end of the flume location to saw out the material needed for construction. Such material can be floated down the flume as fast as the latter is built and used for its further extension.

Flume lines should be surveyed with enough care to ensure evenness of grade. Grades should be kept below 15 per cent wherever possible, and the best results are obtained with grades between 2 and 10 per cent. A careful preliminary survey, followed by a location survey, using a transit and level, will make it possible to obtain a reliable profile map which will serve to show the prospective operator what the grading should be at different points along his line.

Abrupt curves in a flume should be avoided, for they are likely to cause jams. Curves should rarely be permitted to exceed 20 degrees. The longer the material to be handled in the flume, the less abrupt should the curves be. It may be necessary to blast out rocks and boulders, or projecting points of bluffs, or to trestle, or even tunnel, to eliminate abrupt curves or maintain an even grade.

Some flumes are built with only the lining or inside of the box of sawed lumber, the brackets or frames which support the sides of the V being made from round pole wood flattened on one side, and the sills, stringers, braces and trestling of small round timber or poles. Sawed material is recommended for flume construction, however, wherever it can be obtained at reasonable cost.

The "boxes" or sections of a flume vary in length from 6 to 20 ft. Sometimes the boxes are made of only one thickness of boards, but more often of two thicknesses with the joints broken by varying the width of the boards. Sometimes, also, a single thickness of boards is used, with battens spiked over the joints on the outside in the sections between the brackets. In still another form the battens are continuous. On curves the boxes should be shorter than on straightaways, and the bents, arms, and braces correspondingly closer spaced. In general, on curves of from 6° to 10° the boxes should be joined at least once in every 12 ft.; on curves exceeding 10° and less than 15° every 8 ft.; and on curves of more than 15° at least every 6 ft. Very abrupt curves also require increased bracing, in addition to shorter spacing of the arms and brackets. Flumes should also be strongly reinforced at points where extensive shipping is to be done or much material loaded into the flume over the sides.

SIZE OF FLUME.

The most advisable size of a flume for successfully transporting different classes of material depends on the grade, volume of water available, length of flume, etc. The class of material to be handled is always the principal factor to be considered, and upon this should depend largely the decision of what type or size of flume should be constructed. The capacity of a 24-in. V-shaped flume 10 miles in length, operated on a grade that was neither very flat nor exceptionally steep, with plenty of water, has been demonstrated as being capable of handling 25,000,000 ft. b. m. of railroad crossties and lumber per annum, under especially favorable conditions.

For handling small material, such as railroad crossties, cordwood, mining stulls or props, and loose lumber, a 30-in. V-shaped flume (inside measurement) will usually be found of sufficient capacity for most any requirement, provided the grade is neither too flat nor too steep. Where either of these exceptions obtain, the size of the flume should preferably be increased to 36 to 40 ins.

For a log flume nothing less than a 36-in. V-shaped flume should be built, and a 40 to 60-in. V-shaped would be preferable, even for medium-sized logs, if there is a sufficient volume of water available. To operate a log flume successfully and economically there should be a sufficient volume of water avail-

TABLE I.—WEIGHT OF WATER IN A 16-FOOT SECTION OF FLUME WHEN FILLED TO VARIOUS DEPTHS.

Slant depth. Inches.	Weight of water in a section of flume 16 feet long. Pounds.
20	1,390
22	1,680
24	2,000
26	2,350
28	2,710
30	3,110
32	3,560
34	4,010
36	4,490
38	5,010
40	5,540
42	6,110
44	6,740
46	7,300
48	7,990
50	8,670
52	9,360
54	10,100
56	10,900
58	11,700
60	12,500

able to fill the flume three-fourths full on all moderate grades. It will be apparent that if the grade of a large log flume were very steep, a very large body of water would be required. The larger, longer, and heavier the material to be handled, the larger the size of the V should be, the lesser degree of curvature is permissible, and the stronger must be the construction work.

Tables I to IV show the approximate amount of water in cubic feet required to fill a V-shaped flume on steady grades of different per cents and gives other data of value.

If a flume line is properly constructed on a favorable and steady grade where it can be operated with a plentiful supply of water, the actual cost of transportation alone is very slight after the flume is constructed. But the important fact should not be lost sight of that a flume works only in one direction and that all supplies intended to be used in lumbering operations have to be hauled to the head of the flume by animal or other power.

COST OF CONSTRUCTION.

The cost of constructing flumes will also vary a great deal with the conditions existing

TABLE II.—AMOUNT OF WATER REQUIRED TO FILL FLUMES TO VARIOUS DEPTHS AT DIFFERENT GRADES.

[illegible] depth, inches	Grade per cent: 1	2	3	4	5	6	7	8	9	10	11	12	14	16	18	20
	Flow—Cubic feet per second															
20	10	14	17	[illegible]	22	24	26	[illegible]	29	31	32	34	36	39	41	43
22	13	18	22	[illegible]	28	31	33	[illegible]	[illegible]	40	42	44	47	51	54	56
24	16	23	28	32	36	39	42	[illegible]	48	51	53	57	60	64	68	71
26	20	28	34	[illegible]	45	49	53	56	60	63	66	69	74	79	84	89
28	24	34	42	48	54	59	64	68	72	76	80	84	90	96	102	108
30	29	41	50	58	65	71	77	82	87	92	96	101	109	116	123	130
32	35	49	60	69	78	85	92	98	104	109	115	120	130	138	147	155
34	41	58	71	81	91	100	108	115	122	129	136	142	153	163	173	183
36	48	67	82	95	107	117	126	135	143	150	158	165	178	190	202	212
38	55	78	95	110	123	135	145	156	165	174	182	190	206	220	233	245
40	63	89	109	125	140	154	166	178	189	199	209	218	235	251	266	281
42	72	101	127	143	160	175	189	203	215	226	237	248	267	286	304	319
44	81	114	140	163	181	199	214	229	243	257	269	281	303	325	344	362
46	91	130	158	183	204	223	241	258	274	289	302	316	341	365	387	408
48	102	144	177	205	230	251	271	290	307	323	340	354	383	410	434	458
50	115	161	198	228	255	279	302	323	343	360	378	397	428	456	483	510
52	127	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	381	401	420	440	475	507	537	567
54	140	198	242	281	313	342	371	396	421	443	465	484	524	560	594	626
56	156	[illegible]	269	311	348	[illegible]	412	[illegible]	[illegible]	[illegible]	517	[illegible]	583	621	661	697
58	170	241	295	339	380	415	449	481	510	537	564	589	636	681	721	759
60	186	262	322	372	415	455	492	525	558	589	616	644	695	744	788	831

TABLE III.—VELOCITY OF WATER IN FLUMES WHEN FILLED TO VARIOUS DEPTHS AT DIFFERENT GRADES.

[illegible] depth, inches	Grade per cent: 1	2	3	4	5	6	7	8	9	10	11	12	14	16	18	20
	Velocity of water—Miles per hour.															
20	[illegible]	7	[illegible]	[illegible]	11	12	[illegible]	[illegible]	14	[illegible]	16	17	18	19	20	21
22	5	7	9	10	11	13	14	14	15	16	17	18	19	21	22	23
24	[illegible]	8	9	11	12	13	14	15	16	17	18	19	20	22	23	24
26	5	8	10	12	13	14	15	16	17	18	19	20	22	23	24	26
28	[illegible]	[illegible]	10	12	14	[illegible]	16	17	18	19	20	21	23	24	26	27
30	[illegible]	[illegible]	11	[illegible]	[illegible]	16	[illegible]	18	[illegible]	20	21	22	24	25	27	29
32	7	[illegible]	11	13	[illegible]	[illegible]	18	19	20	21	22	23	25	27	28	30
34	7	10	12	14	15	17	18	[illegible]	21	22	[illegible]	24	26	28	29	31
36	7	10	12	14	16	18	19	20	22	23	24	25	27	29	31	32
38	[illegible]	11	13	[illegible]	17	18	[illegible]	21	[illegible]	24	[illegible]	26	28	30	32	[illegible]
40	8	11	13	15	17	19	20	22	23	24	26	27	29	31	33	35
42	8	11	14	16	18	[illegible]	[illegible]	[illegible]	24	25	26	28	30	32	34	36
44	8	[illegible]	14	16	18	[illegible]	22	[illegible]	[illegible]	26	27	[illegible]	31	33	35	37
46	[illegible]	12	[illegible]	[illegible]	19	21	[illegible]	24	[illegible]	[illegible]	28	[illegible]	[illegible]	34	36	38
48	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	35	37	39
50	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	31	[illegible]	36	38	40
52	[illegible]	[illegible]	16	[illegible]	21	[illegible]	[illegible]	26	[illegible]	[illegible]	31	[illegible]	[illegible]	37	39	41
54	[illegible]	[illegible]	16	[illegible]	21	[illegible]	[illegible]	27	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	38	40	42
56	10	14	17	19	22	24	26	28	29	31	32	34	36	39	41	43
58	[illegible]	14	17	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	31	[illegible]	34	37	40	42	44
60	[illegible]	14	18	20	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	32	[illegible]	[illegible]	38	41	43	45

TABLE IV.—ESTIMATED APPROXIMATE AMOUNTS OF LUMBER REQUIRED FOR V-FLUMES.

	Ft. B. M. per mile. (1)	(2)	(3)	(4)	(5)
[illegible]	3,190		5,060	7,480	13,200
[illegible]	6,380	5,060	9,570	12,760	19,140
[illegible]	1,540	9,570	3,025	4,490	9,240
[illegible]	2,970	3,025	3,025	4,400	7,260
[illegible]	[1]4,125		[1]4,125	[1]5,500	[illegible]
[illegible]	21,120	[1]4,125	28,160	28,160	42,240
[illegible]	3,713	28,160	5,500	9,900	18,480
[illegible]	[illegible]	[illegible]	2,228	4,675	9,460
[illegible]	[illegible]	[illegible]	5,157	9,900	23,760
[illegible]	[illegible]	5,157	40,920	48,840	[illegible]
[illegible]	6,690	[illegible]	6,600	10,560	17,600
[illegible]	6,600	[illegible]	6,600	6,600	10,560
	[illegible]	127,010	119,970	153,175	275,990
Add for waste	[illegible]	7,990	[illegible]	6,825	9,010
Total	100,000	133,000	125,000	160,000	285,000

[1] [illegible] of each mile, on an average, will be too close to the ground to require the use of these braces.

[illegible] battered box, [illegible]

[illegible]

MATERIAL.

The cost of transporting material by flumes varies greatly according to the conditions under which the flume is being operated, the class of material, time of year, number of men necessary to operate the flume, and all the other factors that go to make up or reduce the expense of operation. Railroad crossties have been flumed a distance of 20 miles at an actual cost for operation alone of one-half cent per tie, or 15 cents [illegible] mately the same rate.

in the locality, the cost of lumber, cost of nails, and price of labor. In localities where it is possible to get a boiler, engine, and mill to the upper end of a proposed flume line cheaply and without being compelled to go to the expense of constructing a costly road, where there is plenty of timber easily accessible to the mill, which can be cheaply manufactured into lumber for purposes of construction, with low-priced labor, a flume can be constructed much more economically than in a locality where all these conditions were just the contrary. Rough lumber suitable for the construction of a flume can [illegible] construction work at a price varying from $7.50 to $10 for manufacture alone, exclusive of stumpage value.

So much depends upon the locality in which a flume is to be constructed, the price of labor, and the facilities for getting the necessary construction material cheaply, that it is impracticable to attempt any very close estimate on the total cost of any flume until all of the surrounding conditions are thoroughly understood. But in general, under favorable conditions, with a basis of $2.25 per diem for common labor and from $3.50 to $4 per diem for carpenters, not including board, suitably prepared lumber should be built into a flume for about $7.50 per thousand. This would be about the minimum figure, and the cost would be liable to range upward from this price to $12 or higher, according to the conditions and prices of labor.

The cost of the construction of the Bear Canyon flume in Montana, a 26-in. V 10 miles long, was approximately $2,000 per mile. The lumber cost $8.50 per thousand to manufacture and fit it for construction purposes, and it required about 100,000 ft. b. m. to the mile. The labor cost $800 per mile, and $350 per mile was expended for nails, iron for trusses, and for cost of surveying. This flume was constructed a number of years ago when the cost of material and labor was less than it is today.

A flume constructed from Dayton to Woodrock along the Tongue River, in the Bighorn National Forest, Wyo., is said to have cost approximately $3,500 per mile, in round figures, the cost of different sections varying from $2,500 to $7,500 per mile. This was a 30-in. V flume. There was considerable rock work on this line; Granite Canyon had to be passed through, where in some localities the flume was practically pinned to the sides of the canyon walls; there were several rock tunnels to be made through projecting rock; and there were necessarily some very high trestles to be constructed. Another difficult feature of the construction of this flume was that of building the line across several miles of very soft, spongy ground with an almost flat grade to contend with. This flume was also constructed several years ago when the prices of labor and material were not, in general, so high as at the present time.

Probably one of the best examples of modern V-shaped log-flume construction is a flume recently constructed on Rochat Creek, near St. Joe, Idaho. This flume, which is unusually large and strongly constructed for the purpose of handling large, heavy logs, and long timbers, is said to have cost approximately $8,000 per mile for the 5 miles of its length. This figure includes the cost of construction of a wagon road and telephone line equipment.

An Improved Intake for Irrigation Water from a Silt Laden Stream.

The diversion of irrigation water from streams carrying silt, sand and gravel calls for works which will retain the coarser materials and allow the fine silt to enter the canal system and be distributed over the land, where it serves a valuable purpose. A discussion of means for accomplishing this sand and silt separation and a description of an improved form of diversion works are given by A. L. Harris in Proceedings American Society of Civil Engineers, Vol. XL, p. 39, from which we abstract as follows:

The entrance of sand into the canals has been resisted by using the principle of skimming the less turbid water from near the surface. A forebay is walled off on two sides, in the form of a comparatively deep channel running directly in front of the row of intake gates; it is open at the upper end and given outlet at the lower end by large sluice-gates emptying through an opening in the dam. The intake gates draw water from this forebay at a level considerably above its bottom, and the sluice-gates, which have large capacity, draw from the very bottom of the forebay, which has a good slope toward them. In entering the forebay, the motion of th water is checked by the increased cross-section, and the heavier sand grains settle to the bottom. At intervals the sluice-gates are opened suddenly by a hydraulic piston and a powerful rush of water washes out the sand. During a flood the water eddies about and less sand is allowed to settle. With the gates placed in the usual way, if much water is drawn into the canals at these flood times, sand is carried in suspension with it. For this reason it is customary to build some kind of settling basin or sand-trap, within the first mile of the canal, where sand may be caught and sluiced back into the river. It would be better if all this separation of the sand could be done at the headworks. It is likely that the money expended on sand-traps or settling basins could, with equal advantage, be spent on the improvement of the intake sand-sluicing system.

To operate with good effect, the sand-sluicing gates must draw a current of water in front of every intake gate with enough velocity and agitation to scour thoroughly that part of the forebay. For this condition care should be taken that the capacity to enter the forebay, as compared with the discharge of the sluice-gates be not so large that the high velocity in sluicing occurs only close to the exit. On this account sluicing for the benefit of the forebay cannot be done in flood time, although the sluice-gates are often left slightly open during a flood to prevent the banking of sand against them. As the water must always have enough room in entering the

this gate, being controlled by the upper part, called the "flood gate," which closes down on the first as a sill. Gravel rolling along the river bottom during the flood passes on over the dam, finding no place to enter or accumulate. Whatever gravel and sand does find its way into the intake gates is caught in the settling chambers (one for each intake gate) where it can always be sluiced under uniform conditions from the bottom, through the sluicing conduit, and back to the river below the dam. It is planned to have the upper leaf of the inside gates rise and cut off the backwater of the canal simultaneously with the opening of the lower, or sluicing leaf. These gates, however, can be worked independently, and, in case of need, the water may be shut off from the sluicing chambers entirely. It is believed that an intake of this type can be built and operated as cheaply as the other. A suitable power operating mechanism, which can be uncoupled for hand operation, can readily be designed for the proposed arrangement, as it has already been done for the old. It is also believed that sluicing the collected materials from one chamber of moderate size at a time, where a shallow stream with sharp fall can be secured to agitate and cut the deposit, will be more certain and thorough. All the sluicing gates discharge into the same waste conduit, as only one or two settling chambers need be flushed out at the same time.

The most satisfactory sluicing and intake gates yet devised are of the common rectangu-

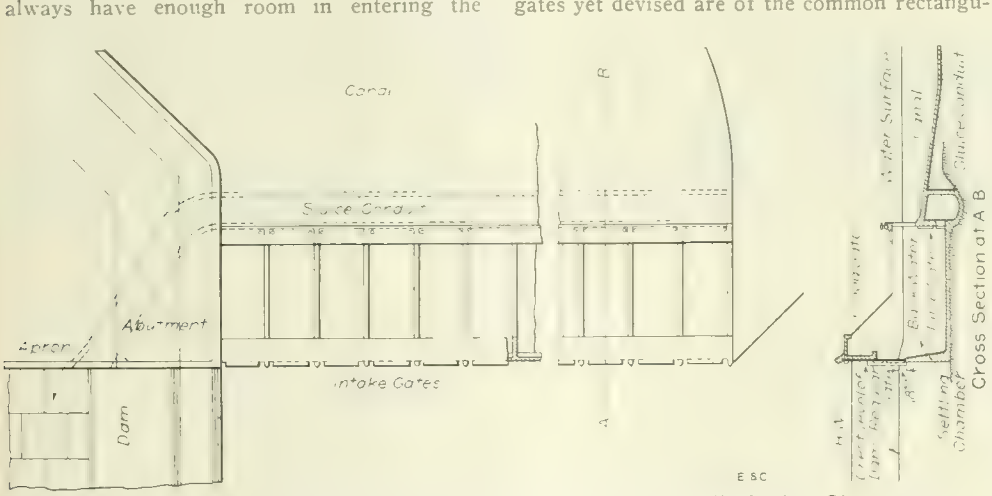


Fig. 1. Proposed Intake for Irrigation Water from Silt Laden Streams.

forebay to fill the maximum requirements of the canal, it follows that, to draw the level down and get a complete scouring at one operation the sluice-gates must have a discharge greater than the canal requirements, unless indeed some device for controlling the entrance area to the forebay can be provided. Such a device would have to be on the outer wall of the forebay, in a very exposed position in flood season, and would add one more complication to the gate system. In the best examples now built there are rather heavy and expensive sluice-gates, on account of the large quantities of water to be handled. The writer has recently designed diversion works for a canal, to be taken out of the Gila River, in which he attempts to improve the sand-sluicing equipment. A modified arrangement based on that design is shown by Fig. 1 which he proposes as an improved type of intake works suitable for silt-laden rivers. In this design the skimming principle is carefully preserved for use in flood time when most needed. For the ordinary low-water conditions, the water is clearer, and is taken from the river over a series of wide shallow crests which reduce the slope of the river bottom in the approaches to one so gentle that coarse materials will hardly be carried close to the gate entrances.

In flood times the forebay of the old type fills with gravel which is not easily sluiced out afterward. With the proposed arrangement, the lower part of the intake gate, called the "regular gate" is closed as soon as the flood rises 2 or 3 ft. above the crest of dam, and water for the canal is taken over the top of

lar sliding type, of cast iron or sheet steel, and operating by rising screw stems. They move on bronze sliding strips, and close on oak or metal sills at the bottom. Those at the Granite Reef Dam, built by the U. S. Reclamation Service, are of thin cast iron, of arched section on the compression side, the tension being taken by steel rods at the back and placed across the bow of the cast-iron shell. They are light, strong and durable. In this case the sluice-gates are operated by hydraulic pressure pumps, the transmission being by heavy chains running over sheaves. They are weighted with concrete for closing by gravity. At the Granite Reef Dam a very excellent feature of all the gates, which were designed by F. Teichman, is that on the pressure side, where sand, etc., is likely to bank against them, they present smooth fronts instead of a system of deep ribs, as often built in the past.

In designing gates for these situations, the question of the proper coefficient of starting friction to use for the bronze sliding ways comes up. The writer's experience has shown that this coefficient is much larger than published authorities known to him would indicate. In order to make a reliable determination of the coefficient under actual working conditions, the writer disconnected the raising mechanism of a 5x7-ft. cast-iron gate, having machine-bronze sliding way, in the power canal at Roosevelt, and in its place attached a long timber lever resting on a fulcrum made of a piece of round steel shafting held between flat steel plates. A platform for carrying weights was then suspended from a definite point on the lever arm, and the apparatus was used to weigh the starting resistance. The actual weight of this particular gate was known from the inspector's weight obtained at the time of its receipt from the manufacturers. The timber lever was weighed and its weight per lineal foot assumed to be constant. A correction for the weights of the lever arms was then made.

Statement of Conditions of Test—Water Pressure on Only One Side of Gate.
Size of gate opening—5 by 7 ft.
Area subjected to water pressure on the gate, measured on the center line of closing strips—38.7 sq. ft.
Depth of water at sill of gate—8 ft. 8 in.
Head on center of pressure area—6 ft. 0 in.
Total water pressure on gate—14,500 lbs.
Weight of gate—4,300 lbs.
Sliding strips—Machined bronze.

Two experiments were made as follows:

First Experiment.—To determine coefficient of starting friction after the gate had been closed tight for several weeks:

Result—
Frictional resistance.................. 9.062 lb.
Coefficient of starting friction........ 0.625 lb.

Second Experiment.—To determine coefficient of starting friction with the gate raised off the sill about ½ in. and water escaping under its lower edge:

Result—
Frictional resistance.................. [illegible] lb.
Coefficient of starting friction........ 0.62 lb.

The turbid condition of the water, perhaps, is the chief cause of the increased size of the coefficient.

In nearly all cases it would probably pay to use gasoline engine power for operating the gates. These engines are now so common that one suitable for the power required may always be found, and speed in opening and closing is essential. They should be designed in such a way, however, that hand power may be applied in case of necessity.

Trash-racks and booms, for protecting the gates from driftwood, have not been found necessary. Most of the driftwood comes down in flood time, and then it is nearly always carried in the middle of the stream and over the dam, as it collects in the stronger current in that part of the river. If the dam is placed below, or on, a sharp bend in the stream, which is not good policy, the drift, of course, will be thrown close to the outside shore and will need to be guarded against.

Gate Structures for Irrigation Canals.—Most of the gate structures in American irrigation canals a few years ago were of wood, but more recently concrete, both plain and reinforced, has come into common use. Wood has the advantage of cheapness and of easy handling and the disadvantage of rapid depreciation, while concrete which has the advantage of permanence is more costly. The kind of material used, as well as other features of gate structures, varies in different irrigated regions of the West. One section often uses features especially adapted to it for which other sections, that could use them equally well, are ignorant. The U. S. Department of Agriculture is endeavoring to bring together such designs for gate structures as are adapted to many localities so that each locality may profit by the practices of others, and has just issued a new bulletin (No. 115) entitled "Gate Structures for Irrigation Canals," intended to be of assistance to engineers and others with technical knowledge of the subject.

The purpose of all the gates considered in the new bulletin is the control of the flow of water in the ditches or canal systems. Headgates and floodgates regulate the water entering the system from the source of supply; check gates regulate the water while within the canal; sand and waste gates control the water which is to be turned out and wasted; and branch canal, lateral, and delivery gates regulate the water turned out to branches of the system or to users.

Small and medium-sized structures are for the most part described in the new bulletin, as it is believed that most of the problems confronting the engineer located in isolated regions of the West relate to this class. The bulletin should also be of value to directors of mutual water companies who are themselves irrigators and who are called upon to pass upon questions of construction and maintenance.

Since the bulletin is prepared for engineers and others who are familiar with gates and gate structures, it does not attempt to treat the subject fully, but merely gives examples of structures which serve the purpose for which they are intended better than many others in common use. Local conditions control many features of gate structures, and the descriptions given relate to existing structures in actual use, which it is believed will prove suggestive and can be readily adapted to other conditions by local engineers and ditch owners.

One of the important questions dealt with is that of the materials to be used for gate structures. Shall they be of wood or concrete? The best practice, according to the Department's investigator, seems to be to make a structure of combined wood and concrete, using concrete for the parts that are inaccessible and not easily replaced, and wood for the accessible parts which can be replaced easily. Local conditions affecting the relative prices of these materials will also help to determine which material should be used.

Because of the high cost of water rights, and the inability of settlers to make the payments required, there is much to be said in favor of the cheaper wooden structures for original construction with a view to their replacement with more permanent structures of concrete, as the wood decays. This will lessen the first cost and will bring the heavier cost after the lands have been put under cultivation. The use of wood has the further advantage that in case of mistakes in either the type of structure or the location it is not so costly. It frequently happens that structures are found to be placed too high or too low, or they are too small and not of the best type. A few year's experience in their operation will demonstrate these facts after which a more permanent structure of wood or concrete may be put in.

However, if the failure of a structure will result immediately in great damage to the canal system or to crops, only the most nearly permanent construction should be used at first. If the failure merely means the replacement of the structure itself, the cheaper construction may profitably be put in first.

The new bulletin contains 61 pages and numerous plates and figures illustrating structures that have already proved practicable. The new bulletin is strictly a professional paper and of little interest to the farmer in general, but the attention of the engineers and directors of farmers' canal companies in the irrigated sections of the West is invited to its contents. The bulletin may be had free as long as the Department's supply lasts by those who may find it profitable to use.

Some Costs of Grouting Dam Foundations.

Comparing the published figures of costs of grouting the foundations of the Estacada and Lahontan dams, Mr. S. H. Rippey, in Proceedings American Society of Civil Engineers, Vol. XL, p. 767, obtains the following:

COST OF DRILLING AND GROUTING AT ESTACADA AND LAHONTAN DAMS, PER LINEAR FOOT OF COMPLETED WORK.

Labor and materials.	Estacada Dam. Fisher.	Estacada Dam. Rands.	Lahontan Dam. Cole.
Labor, drilling	$0.58	$0.59	$0.93
Labor, grouting	0.18	0.18	0.29
Cement	0.12	0.12	0.31
Repairs and supplies	0.17	0.17	0.23
Plant	0.30	....	....
Plant depreciation	....	0.15	0.35
Power	0.05	....	0.03
Other items	....	....	0.94
	$1.40	$1.21	....
Salvage on plant, credit	0.17	....	....
Direct cost	$1.23	$1.21	....
Total field cost	....	....	$3.08
General plant, etc	0.32	0.45	0.12
Coffers and pumping	....	0.15	....
Engineering and superintendence	....	0.19	0.27
Clerical and office	....	....	0.10
Total cost per foot	$1.55	$2.00	$3.57

ROADS AND STREETS

Organization and Standards of the Pennsylvania State Highway Department.

(Staff Article.)

The present State Highway Department of Pennsylvania was organized under what is commonly known as the "Sproul Road Act" of the legislature of 1911. This act called for a reorganization of the existing highway department and further provided for taking over as state highway the roads comprising 296 specified routes which form connecting links between county seats and principal cities and towns, and in addition furnish trunk lines through the state. These roads were to be maintained and reconstructed by the department; existing contracts completed; township roads improved by state aid to the extent of $2,000,000, half paid by the state and half by county or township; and all highways in the state surveyed and mapped by counties. The licensing and regulation of automobiles was also placed under the supervision of the highway department.

To carry out the provisions of this law the state is divided into 15 districts, each in charge of an assistant engineer. The headquarters of the state highway department are at Harrisburg, and the executive officers consist of a state highway commissioner, Mr. E. M. Bigelow; two deputy state highway commissioners, Messrs. J. W. Hunter and E. A. Jones; chief engineer, S. D. Foster, and auditor, W. R. Main. The field work is under the direction of a bridge engineer, 15 assistant engineers, and 50 superintendents. The auditing department was organized with headquarters in the Capitol Bldg. at Harrisburg, under the direction of a certified public accountant. A maintenance division was likewise organized in charge of a maintenance engineer, under whose direction is carried on the maintenance of the state highway routes, aggregating 8,827 miles. The chart, Fig. 1, shows the organization and personnel of the department at present.

WORK ACCOMPLISHED.

The work imposed upon the department was varied and the funds provided were totally inadequate. Work has, however, progressed steadily in spite of this, although the maintenance of poorly improved roads, required by the law, has consumed a large portion of funds available. Maintenance of this type of road is both costly and difficult.

The total length of the 296 main state highway routes put in charge of the state highway department is 8,827.9 miles, in addition to which there are 306 additional miles known as alternate lines.

The average expenditure for maintenance of approximately 6,000 miles of these routes during the year 1912 was approximately $169.00

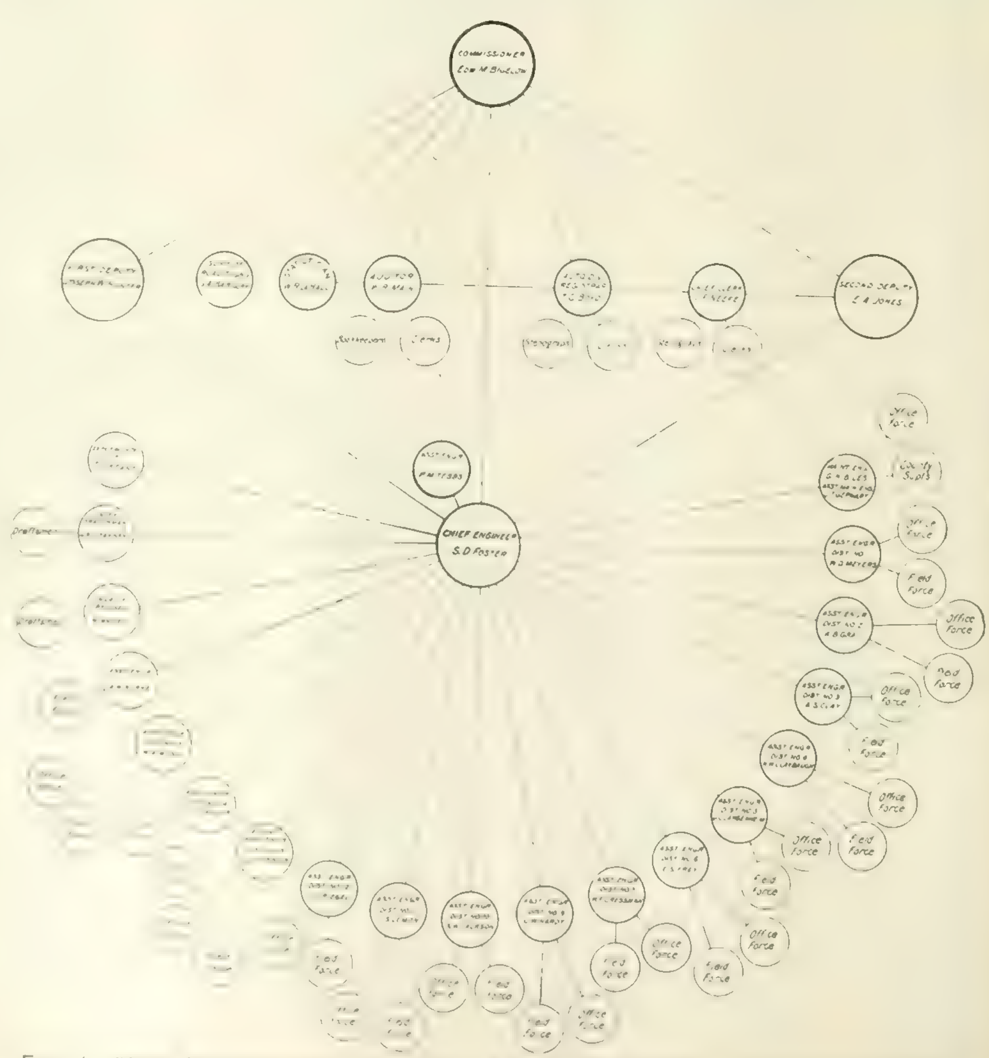


Fig. 1. Chart Showing the Organization of the Pennsylvania Highway Department.

per mile. For the year 1913 a pro rata amount of not more than $168.00 per mile was available. In addition to the above maintenance work on state highway routes considerable maintenance work was done on state-aid roads, 50 per cent of which is returnable by counties and townships.

Surveys have been completed for nearly all (over 95 per cent) of the routes and counties, and information collected for maps and plans. The route surveys were completed at an average cost of $47.87 per mile for surveying and $7.96 for checking and tracing. The average cost of the various items is given in Table I. Surveys were plotted in the offices of the district engineer and checked and traced in the main office at Harrisburg, Pa.

TABLE I—COST OF SURVEYING ABOUT 9,000 MILES OF HIGHWAY IN PENNSYLVANIA.

Item.	Total.	Cost per mile.
Surveying main line	$442,597.98	$47.87
Plotting main line	72,432.79	11.36
Checking and tracing main line	8,717.79	7.97
Surveying alternate line	15,461.22	50.45
Miles surveyed, main line	8,827.91	
Miles plotted, main line	6,373.81	
Miles checked and traced, main line	1,094.40	
Miles surveyed, alternate line	306.36	

In addition to the main lines, surveys were made and maps prepared showing the complete surveys of all roads in each county together with towns, villages and other important features. Printed reproductions of these are sold at 25 cts. per copy, which represents the cost of printing. The cost of this work was as follows:

Item.	Av. cost.
Engineering and plotting expense per county	$ 3,70[illegible]
Number of counties	66
Mileage in completed maps surveyed by state	35,512.52
Averave cost per county of printing maps	$ 1[illegible]

TOWNSHIP ORGANIZATION.

According to the provision an act of the legislature passed in 1909, the several counties having a cash tax system are entitled to receive state aid amounting to 50 per cent of the total amount of road tax collected in money, provided this does not exceed $20 per mile.

The supervision of the expenditure of these funds is in charge of a deputy commissioner appointed by the state commissioner. This division is known as the Bureau of Township Highways. The state is divided into districts each in charge of a superintendent who visits and inspects, at least once a year, all bridges and roads in his district and directs repair, maintenance and construction work, approving all plans and specifications for work within his district.

Supervision of actual work within the township is in the hands of three supervisors elected for a term of six years. The road tax levy (ordinarily about 10 mills on the dollar) is made yearly by these supervisors.

The township is subdivided by the supervisors into one or more road districts and a superintendent or road master employed for each district. These roadmasters, who are subject to the regulation of the state highway commissioner, employ and hire or purchase teams, labor and equipment for the prosecution of their work. Contracts exceeding $100, however, must be approved by the district superintendent.

PLANS AND SPECIFICATIONS.

The plans (drawn on a roll) are to a scale of 100 ft. to an inch for alignment, 100 ft. to the inch in horizontal scale and 20 ft. to the inch in vertical scale.

These plans are approved by the chief engineer and the commissioner and returned to the various division engineers after contracts are let.

The specifications and contracts used by the department are bound in a booklet 6x7½ in. in size and describe bituminous and brick surfaces, concrete structures and various details of construction. The specifications are complete and call for first-class work.

SURFACES AND DETAILS.

The types of cross-sections used are illustrated in Figs. 2 and 3. It will be noted the construction is very substantial and the details conservative. In the past a large percentage of the roads built by the state have been on a Telford base. At the present time the concrete base is coming into more general use. Brick surfacing is also being used extensively. Asphaltic concrete and macadam, however, have been used more extensively than other types of improved hard surfaces. The lengths

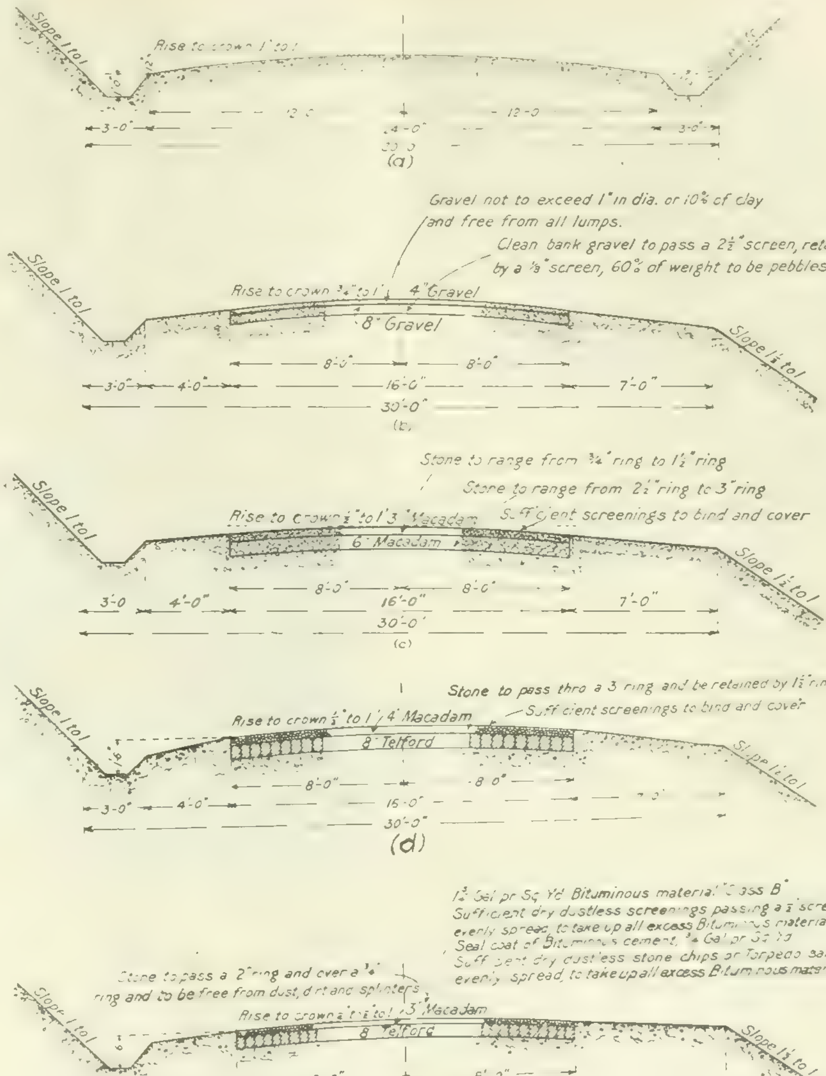


Fig. 2. Typical Standard Road Cross Sections.

(a) Earth. (b) Gravel. (c) Macadam. (d) Macadam on Telford [illegible] macadam—penetration method. (f) Asphaltic con[illegible] on Telford base [illegible] on Portland cement concrete base. (h) Portland cement concrete painted with bitumen.

of different classes of surfacing used between June 1, 1911, and June 1, 1913, are as follows:

Type of surface.	Length in ft
B[illegible]	[illegible]
Asphaltic concrete	[illegible]
Asphaltic macadam	511,977
Water bound macadam	[illegible]
Concrete	[illegible]
Total length in feet	1,943,602
Total length in miles	368.11

The general clear cut appearance of Pennsylvania state roads is shown by Fig. 4.

Figure 5 illustrates a standard type of wooden guard rail. Other types of rail are used to fit special conditions, among them a rail consisting of 6-in. posts 7 ft. long, set 3 ft. in the ground at intervals of 8 ft. A single ¾-in. wire cable or 1½-in. pipe runs through them 8 ins. from the tops which have a 2-in. taper. Another type of rustic rail has the same overall dimensions except that rough split rails are used, the top rail being trimmed to fit the notched tops of the posts as is the case of sawed material.

TABLE II. DIMENSIONS FOR VARIOUS HEIGHTS OF GRAVITY RETAINING WALLS.

Cement Masonry.						Dry Rubble Masonry.						Concrete Masonry.					
Ground level with top of wall			Surcharged.			Ground level with top of wall.			Surcharged.			Ground level with top of wall.			Surcharged.		
H.	T.	B.	H.	T.	B.	H.	T.	B.	H.	T.	B.	H.	T.	B.	H.	T.	B.
4' 0"	1' 6"	1' 8"	4' 0"	2' 0"	2' 3"	4' 0"	2' 0"	2' 0"	4' 0"	2' 0"	2' 7"	4' 0"	1' 6"	1' 6"	4' 0"	1' 6"	2' 0"
5' 0"	2' 0"	2' 0"	5' 0"	2' 0"	2' 10"	5' 0"	2' 0"	2' 6"	5' 0"	2' 0"	3' 4"	5' 0"	1' 6"	1' 9"	5' 0"	1' 6"	2' 5"
6' 0"	2' 0"	2' 5"	6' 0"	2' 0"	3' 5"	6' 0"	2' 0"	3' 0"	6' 0"	2' 0"	4' 0"	6' 0"	1' 6"	2' 2"	6' 0"	1' 6"	2' 10"
7' 0"	2' 0"	2' 10"	7' 0"	2' 0"	3' 10"	7' 0"	2' 0"	3' 6"	7' 0"	2' 0"	4' 7"	7' 0"	1' 6"	2' 5"	7' 0"	1' 6"	2' 5"
8' 0"	2' 0"	3' 3"	8' 0"	2' 0"	4' 6"	8' 0"	2' 0"	4' 0"	8' 0"	2' 0"	5' 4"	8' 0"	1' 6"	2' 10"	8' 0"	1' 6"	3' 10"
9' 0"	2' 0"	3' 7"	9' 0"	2' 0"	5' 0"	9' 0"	2' 0"	4' 6"	9' 0"	2' 0"	5' 10"	3' 0"	1' 6"	3' 2"	9' 0"	1' 6"	4' 4"
10' 0"	2' 0"	4' 0"	10' 0"	2' 0"	5' 7"	10' 0"	2' 0"	5' 0"	10' 0"	2' 0"	6' 7"	10' 0"	1' 6"	3' 6"	10' 0"	1' 6"	4' 10"

Cement—Top of wall not less than 1' 6", 2' if possible. Face battered; back perpendicular. Weep holes with blind drain in wet localities.

Dry rubble—Top of wall not less than 2'. Face battered; back perpendicular. Courses perpendicular to face batter. Face of wall pointed. Weep holes with blind drain in wet localities.

Concrete—Top of wall not less than 1' 6". Face battered 1:12; back battered or stepped. Weep holes with blind drain in wet localities. Expansion joints every 25'.

All offsets 6". Bottom of footing below frost line. (Depth of footing at least 3'.) Backfilling done with acceptable material placed in layers of not less than 6" and thoroughly rammed.

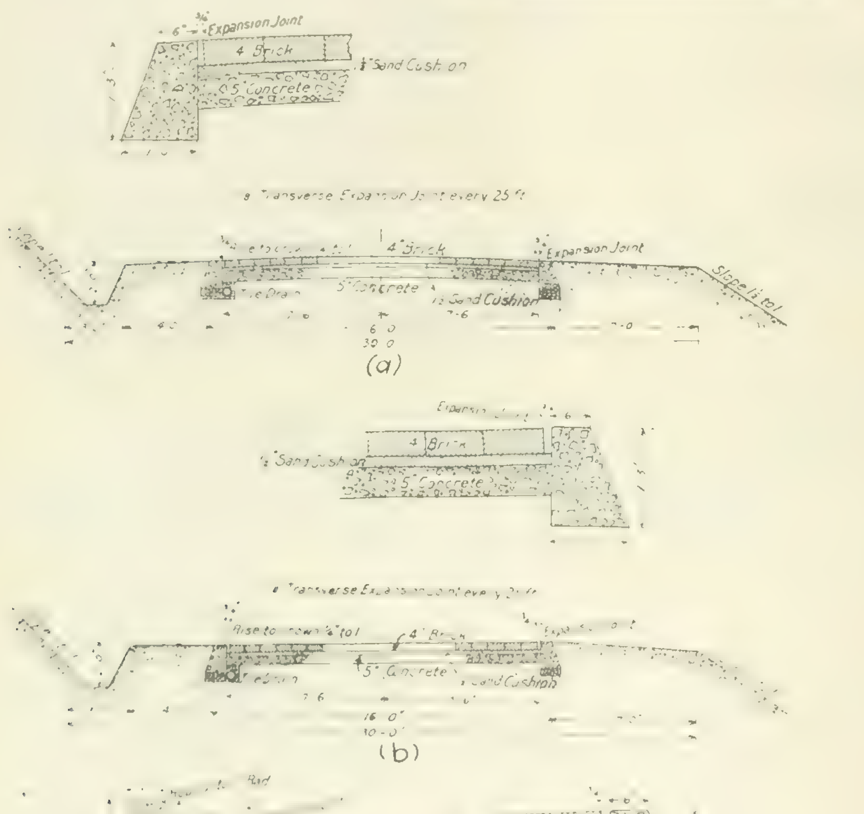


Fig. 3. Typical Standard Cross Sections for Brick Roads.

In Fig. 7 is illustrated a type of paved gutter and types of blind and tiled drains.

Standard types of retaining walls are illustrated in Fig. 8. Table II gives dimensions for various heights of walls similar to those illustrated in Fig. 8.

WATERWAYS.

At points where cast iron pipes are required the standards for headwalls to be used are shown in Fig. 9.

Roads in the hilly sections of Pennsylvania are frequently overflowed, due to their frequent location upon stream banks. Recommended methods of protecting these roads are shown in Fig. 10.

In Figs 11 and 12 are illustrated large concrete culverts.

SPECIFICATIONS.

The following is an abstract of portions of the specifications used by the Pennsylvania Highway Department:

PIPE CULVERTS.

Corrugated Iron Pipe. All corrugated iron pipe shall be made from sheets thoroughly galvanized with zinc spelter before being corrugated, said zinc spelter to be 2 ounces per sq. ft., shall be free from imperfections of any kind, and shall show no signs of cracking or blistering. The corrugations shall be not more than 2¾ ins. in width, and not less than ½ in. in depth.

All corrugated iron pipe supplied shall be of first quality, straight and true to form, and shall be of good and workmanlike construction and shall be constructed of such material as shall be approved by the State Highway Commissioner.

For corrugated iron pipe 24 ins. in diameter and less, the metal shall be not less than 16 gage U. S. Standard; for corrugated iron pipe over 24 ins. in diameter and not over 60 ins. in diameter, the metal shall be not less than 14 gage U. S. Standard; and for corrugated iron pipe 66 ins. in diameter and over, the metal shall be not less than 12 gage U. S. Standard.

All corrugated iron pipe to have tightly riveted seams, to be full circle, lap joints, and laid in a properly prepared trench, and all back filling shall be thoroughly tamped.

BRICK PAVEMENT.

Brick Block Pavement or Surfacing. All brick pavements shall be laid on a concrete base of 5 ins. or more, as may be specified, and shall have a cushion of approved sand or crushed slag sand 1½ ins. in depth and a wearing surface of 4 ins. in thickness; the joints of which shall be filled with Portland cement filler mixed of equal parts of approved sand and cement. The brick block pavement shall be paid for at the price bid per square yard in place, and shall include the sand cushion, brick block, transverse and longitudinal expansion joints and grout filler; also any labor necessary to complete same in place.

Paving bricks shall be reasonably perfect in shape, free from marks, warping or distortion, and shall be uniform in size so as to fit closely together and make a smooth pavement. All bricks shall be homogeneous in texture and free from laminations and seams. They shall be evenly burned and thoroughly vitrified. Soft, brittle, cracked or spawled bricks or kilnmarked to a height or depth of over 3/64 parts of an inch shall be rejected. If bricks have rounded corners the radius shall not be greater than

-16 of one inch. They shall not have less than two nor more than four vertical lugs, and projections shall not be more than ½ in. wide on one side of each brick. The total area of all lugs shall not be more than 3 sq. ins., so that when laid there shall be a separation between the bricks of at least ⅛ in. and not more than ¼ in. The imprint or name of the bricks or maker, if used, shall be by means of recessed and not by means of raised letters. The two ends of the bricks shall have a semi-circular groove with a radius of not less than ⅛ in. and not more than ¼ in. Grooves shall be so located that when the bricks are laid together the grooves shall match perfectly and shall be horizontal when bricks are laid in pavement.

Fig. 4. View Showing a Brick Surfaced Highway. Note Guard Rail and Hand Rail on the Plate Girder Bridge.

After the bricks in the pavement are inspected and the surface is swept clean of spalls, they must be well rolled with a 5-ton steam roller in the following manner:

Rolling and Tamping.—The brick next the curb shall be tamped with a hand wood tamper to the proper gutter grade. The rolling will then commence near the curb at a very slow pace and continue back and forth until the center of the pavement is reached; then pass to the oposite curb and repeat in the same manner to the center of the street. After this first passage of the roller the pace may be quickened and the rolling continued until each brick is firmly imbedded in the sand cushion. The

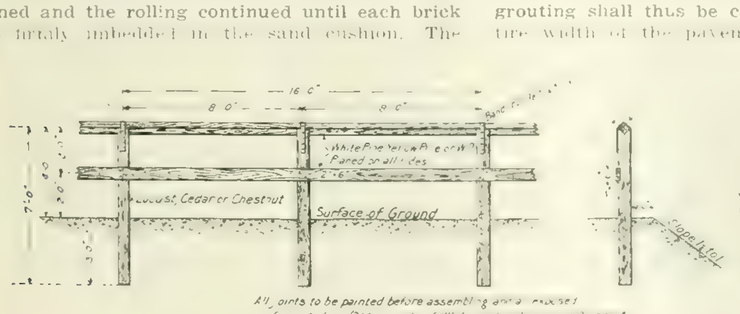


Fig. 5. Standard Wood Guard Rail

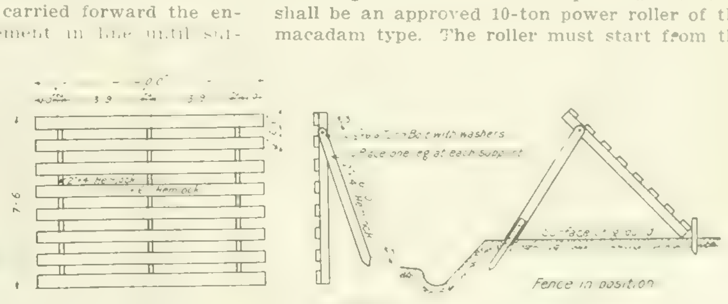


Fig. 6. Snow Fence Used to Prevent Snow Drifting Across Roads.

roller shall then be started at the end of the block and the pavement rolled transversely at an angle of 45° to the curb; repeat the rolling in a like manner in the opposite direction. After this transverse rolling takes place all broken or injured brick must be taken up and replaced with perfect ones, and thoroughly tamped into place.

On grades of five per cent or over the State Highway Commissioner may, if he deems it advisable for the traffic, order the contractor to use special form of brick suitable for steep grades and no extra allowance will be made for the use of such brick.

Bricks shall be approximately 3¼x4x9 ins. in size. All bricks shall be subject to the tests for abrasion, impact, absorption and crushing strength according to the standard methods prescribed by the National Paving Brick Manufacturers' Association.

Expansion Joints.—The bid price per square yard for brick block pavement shall include an expansion joint along each curb ¾ in. in width which shall be constructed of bituminous material approved the State Highway Commissioner. Price bid on brick block pavement shall also include a transverse expansion joint; this form of joint will be placed at intervals of 25 ft. and will extend from curb to curb. The material used for this form of joint will be a heavy grade of felt paper, approximately ⅛ in. in thickness, as approved by the State Highway Commissioner.

Grouting.—After the inspection has been completed, the joints of all bricks in paving shall be filled with grout of a consistency of one part approved sand and one part cement, in the following manner:

The mixture shall be removed from the mixing box to the road surface by means of scoop shovels and from the moment it touches the brick shall be thoroughly swept into all joints by means of push brooms. The work of grouting shall thus be carried forward the entire width of the pavement in line until sufficient time has elapsed for the grout to thoroughly penetrate all joints before the cement has attained its initial set. The entire force will then go over the portion of the work a second time using the same mixture of grout except that it shall be mixed to a somewhat thicker consistency, care being taken in each instance to thoroughly fill all joints flush with the top of the brick. To secure flush joints a third coat of cement shall then be swept in before the second has attained its initial set.

ASPHALTIC CONCRETE.

The foundation may be telford or concrete. When telford base is specified there shall be spread over the entire surface a sufficient quantity of approved crushed stone to entirely fill up all voids and depressions. This course shall be thoroughly rolled and compacted and present a surface true to the cross section of roadway.

Stone.—The stone shall be good, hard, durable granite, trap rock, limestone, gneiss, or such other stone as the State Highway Commissioner may approve. All stone shall withstand a crushing test of 20,000 lbs. to the square inch. The approved stone shall be crushed into fairly uniform cubes, and shall be clean, dry, free from dust, dirt, or vegetable matter, and comparatively free from flakes or splinters.

Mineral Aggregate.—The mineral aggregate shall consist of run of crusher stone passing a one inch ring and shall not contain more than five per cent of dust. Approved, clean, coarse sand shall be used as a filler in proportions found necessary and approved by the State Highway Commissioner.

Bituminous Cement.—The bituminous cement shall be Class "A," as specified herein, and shall have a penetration of 50 to 60 (Dow Method).

Mixture.—The asphaltic concrete mixture shall contain by weight between 53 and 62 per cent of crushed stone, 30 to 37 per cent of approved sand and filler and between 6 and 8 per cent of bituminous cement. (The proportion of the mixture given as bituminous cement shall signify only the bituminous portion of the asphaltic cement, and the proportions stated shall not include the non-bituminous material.)

The several grades and sizes of stone, sand, and filler composing mineral aggregate shall be accurately measured in proportions determined by laboratory test to give the best results; that is, the most dense mixture and one having inherent stability.

Heating.—The mineral aggregate shall be heated in an approved rotary heater to a temperature of from 250° to 350° F.

The bituminous cement shall be heated to a temperature of from 300° to 375° F. in an approved melting kettle that will provide for the uniform distribution of heat without burning or coking and permit of the proper agitation.

Mixing.—Each batch of the mineral aggregate shall be mixed in an approved asphalt batch mixer with sufficient bitumen to thoroughly coat every particle. The grading of the mixture shall be such as to produce a uniform and evenly balanced mix so that the completed mixture shall have as nearly as practicable the density of solid rock.

Laying.—The hot mixture shall be delivered to the road at a temperature of between 225° and 350° F., under cover, if necessary dumped on platforms, and forked and evenly raked over the prepared foundation so that after thorough rolling and compression it shall have a thickness of at least .... inches as designated on the plan.

Rolling.—Unless otherwise specified the roller shall be an approved 10-ton power roller of the macadam type. The roller must start from the side lines and work toward the center of the road unless otherwise directed. The rolling shall at all times be subject to the direction and approval of the State Highway Commissioner.

Seal Coat.—After rolling the wearing surface and while it is clean and warm, there shall be applied a seal coat of approved bituminous cement of proper consistency to be flexible when cold, and shall be evenly spread over the entire surface of roadway at a temperature of from 300° to 350° F. with rubber squeegees.

Immediately over this surface a top dressing of approved torpedo sand or suitable clean stone screenings, free from dust, thoroughly dry, and if necessary heated, shall be spread over the entire surface of roadway and thoroughly rolled. A small surplus of stone screenings shall be left to be worn in or away by the traffic.

Finished Surface.—The entire surface of roadway shall present an even and uniform surface, free from waves and depressions that will hold water. Porous spots which remain wet after the surrounding surface has dried shall be replaced with proper material.

BITUMINOUS MACADAM — PENETRATION METHOD.

Stone.—The stone to meet the requirements as specified under Asphaltic Concrete. The approved stone screenings shall be of the same quality of stone as specified above. The stone screenings shall be dry, free from moisture, clean, dustless screenings passing a ½-in. screen.

Bituminous Cement.—The bituminous cement used shall be Class "B" as specified herein, and shall have a penetration of 90 to 100 (Dow Method). The bituminous cement shall be heated to a temperature of from 300° to 350° F. in an approved melting kettle that will provide for uniform distribution of heat without burning or coking, and permit of the proper agitation.

Bituminous Macadam Penetration.—Upon the foundation specified and prepared there shall be spread a layer of approved stones of such size that they will pass a 2-in. circular opening and be retained on a ¾-in. circular opening.

This layer of stone shall be dry, free from moisture and of such thickness that when it has been thoroughly rolled and compressed it shall have a uniform thickness of at least 3 ins. over the entire roadway. The surface must be firm and, when completed, correspond to grade in proper crown of cross section.

Rolling.—Unless otherwise specified, the roller shall be a 10-ton power roller of the macadam type. The rolling must start from the side lines and work toward the center of the road unless otherwise directed. The rolling shall at all times be subject to the direction and approval of the State Highway Commissioner.

Pouring.—Upon this surface shall be evenly spread to each square yard, by means of approved pressure distributors or fan spout sprinkling pots, 1¾ gals. of bituminous cement. Immediately thereafter a quantity of approved ½-in. clean stone screenings shall be spread to entirely cover the surface of the roadway. The road shall then be thoroughly rolled until a firm, smooth surface results, and conforms to the longitudinal and transverse sections.

Finish.—While the surface is clean and warm a seal coat of approved bituminous cement shall be spread in a quantity of not less than ¾ gal. to the square yard, and while it is in a liquid state there must be spread a top dressing of clean, dry, approved torpedo sand, or dustless stone chips in sufficient quantity to entirely cover the surface of the road and absorb any excess bituminous material. Immediately thereafter the road shall again be thoroughly rolled and compressed. If so directed, the stone, stone screenings, stone chips, and torpedo sand shall be thoroughly heated, and shall be free from moisture.

The roadway shall be of specified thickness, true surface, free from waves and depressions that will hold water. Porous spots that remain wet after the surrounding surface is dry must be replaced with proper material to an even surface.

BITUMINOUS MORTAR MACADAM.

Construction of this class shall consist of concrete or telford foundation, as specified and approved by the State Highway Commissioner, a bituminous binder course of at least 1½-in. thickness, and a bituminous mortar wearing surface of at least 1-in. thickness, or as may be designated and shown on the plans.

The foundation shall be of equal quality, in regard to material and workmanship, as shall be specified and designated on plans for concrete or telford foundations. The binder course shall consist of clean, broken stone, heated and mixed with approved bituminous cement.

Stone.—The stone shall meet the requirements as specified under Asphaltic Concrete, and in addition shall pass a 1-in. circular opening and be retained on a 10-mesh screen. The stone shall be heated in approved dryers to a temperature ranging from 200° to 250° F. The stone at the temperature herein specified shall be thoroughly mixed in an approved batch mixer with bituminous cement hereinafter specified.

Bituminous Cement.—The bituminous cement shall be that specified under Class "A"—penetration 50 to 60 (Dow Method). The temperature of bituminous cement when mixed with hot stone shall range between 300° and 350° F. The bituminous cement shall be melted and heated in approved melting kettles that will provide for a uniform distribution of heat without burning or coking, and shall permit of proper agitation.

Proportions—How Determined.—The proper quantity of bituminous cement for given amount of stone shall be carefully determined by experiments made at mixing plant, and shall range between three and five per cent. When the proportions are determined they shall be strictly adhered to unless changed by the State Highway Commissioner. The stone and bituminous cement for every batch must be carefully measured. Should the binder appear dull from over-heating or lack of bituminous cement it will be rejected.

Handling.—The bituminous mixture shall be hauled to the road under cover, if necessary dumped on platform, forked and evenly raked upon prepared foundation to such a depth that when thoroughly rolled and compressed with a regular 10-ton power tandem roller it shall not be, in any place, less than 1½ ins. in thickness. The rolling shall continue until binder course ceases to show any signs of further compression. The rolling shall at all times be subject to the approval of the State Highway Commissioner, and shall start from the side lines and work towards the center of the road unless otherwise directed.

Surface.—Should the bituminous binder course not show a proper bond, it shall be immediately condemned, removed and replaced with new material. The upper surface of the binder course shall, after rolling and ramming is completed, be parallel with the surface of the pavement to be laid.

When it is not practical to lay the binder continuously and a joint is unavoidable, the cold material shall be cut off in a straight line across the road and as far back as necessary to insure an even surface of proper thickness. The bituminous binder shall not be laid during wet weather and not unless the foundation is dry.

The wearing surface shall be laid upon the binder course while it is clean and free from all loose particles, and if possible, within 24 hrs. after the rolling of binder course.

Paving Mixture for Wearing Surface.—The wearing surface shall be composed of approved bituminous cement, sand and filler.

The bituminous cement shall be as specified hereinunder Class "A"—penetration 50 to 60 (Dow Method). It shall be heated in approved melting kettles that will permit of uniform distribution of heat without burning or coking and permit of the proper agitation, and shall never be heated to a temperature exceeding 375° F. The sand shall be clean, moderately sharp, hard, free from sewage, clay, loam, and all vegetable matter, and shall be approved by the State Highway Commissioner.

The filler shall be fine lime dust, Portland cement, or other suitable dust, and must be of such degree of fineness that all of it will pass a 30-mesh sieve, and at least 75 per cent of it will pass a 100-mesh sieve.

The materials complying with these specifications shall be mixed in the following proportions by weight: Bituminous cement from 10 to 12.5 per cent; sand from 70 to 90 per cent; filler from 8 to 15 per cent.

The sand and bituminous cement shall be heated separately in approved dryers and melting kettles to a temperature of 350° F. Materials that have been injured or damaged by overheating shall be condemned.

Mixing.—The filler or dust, may while cold, be mixed with the hot sand in the required proportions. The bituminous cement, at the required temperature and in proper proportions, shall be added and the whole mixed in a suitable

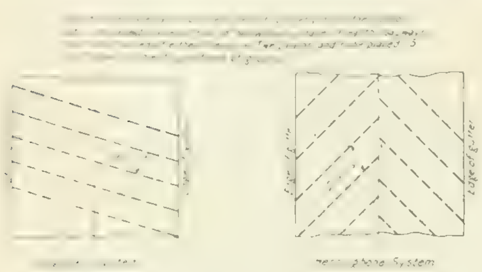
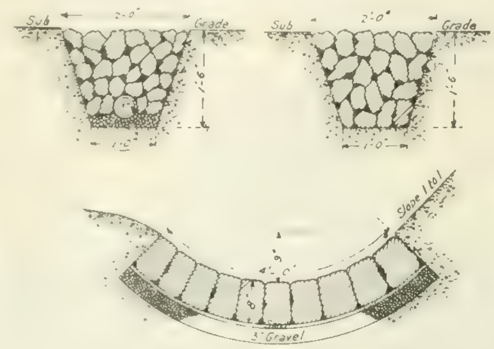




Fig. 7. Drainage and Gutter Details for Pennsylvania Roads.

Left—Arrangement of tile drains. Center—Paved gutter and stone and tiled stone drains. Right—Method of placing stone drains and treatment of side-hill roads.

Fig. 8. Cross Sections of Gravity Retaining Walls.

batch mixer until a thoroughly homogeneous mass is produced.

The paving mixture prepared in the manner described herein shall be hauled to the road under cover and when dumped, if necessary upon a platform, shall have a temperature of not less than 250° F.

compression shall be secured with an approved 10-ton tandem roller. The rolling shall be done under the direction of the State Highway Commissioner and shall continue until all roller marks shall have disappeared, and the surface ceases to show any signs of further compression, and shall be at lest one inch thick.

wearing surface of at least one inch thickness, or as may be designated and shown on the plans.

Materials.—The binder course shall consist of approved sand, clean, broken stone, heated and mixed with approved bituminous cement. The sand shall be approved, clean, moderately sharp, hard, free from sewage, clay, loam, and all vegetable matter, and at least 18 per cent of the sand shall pass an 80-mesh screen, and not more than 40 per cent nor less than 15 per cent shall be retained upon a 40-mesh screen. The

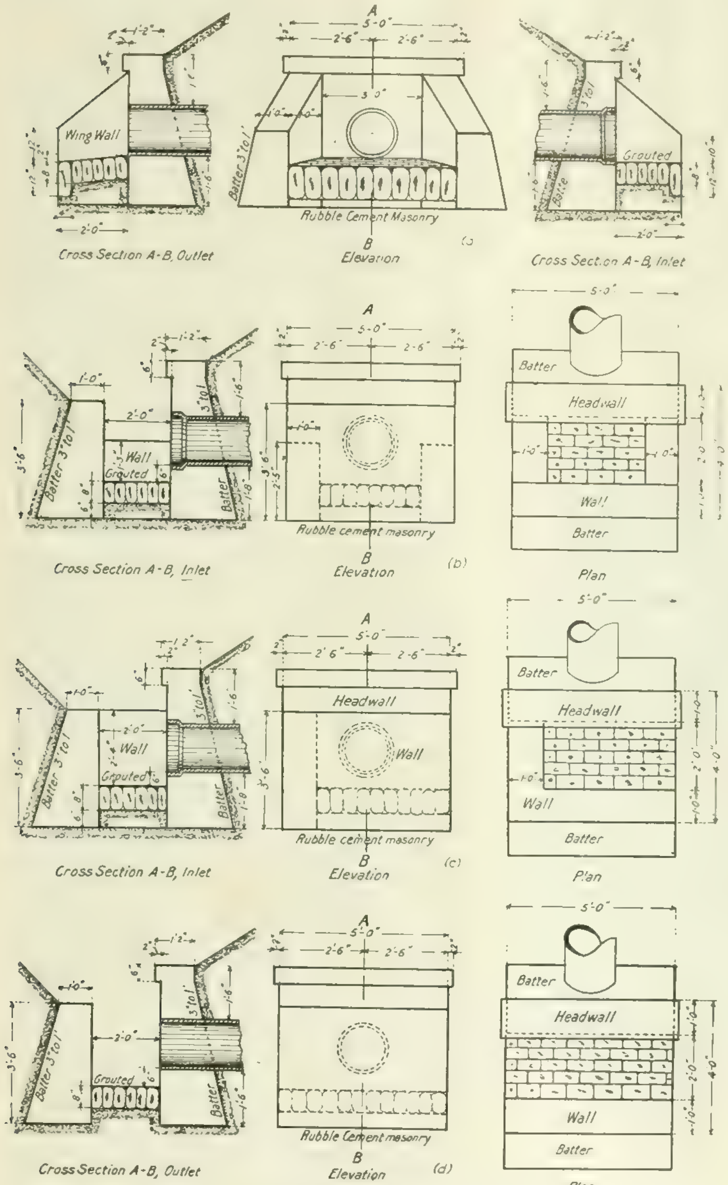


Fig. 9. (a) Cast Iron Pipe Culverts.

(a) Standard type. (b) Inlet end used where water comes from two directions. (c) Inlet end on a steep grade. (d) Outlet end used in a cut.

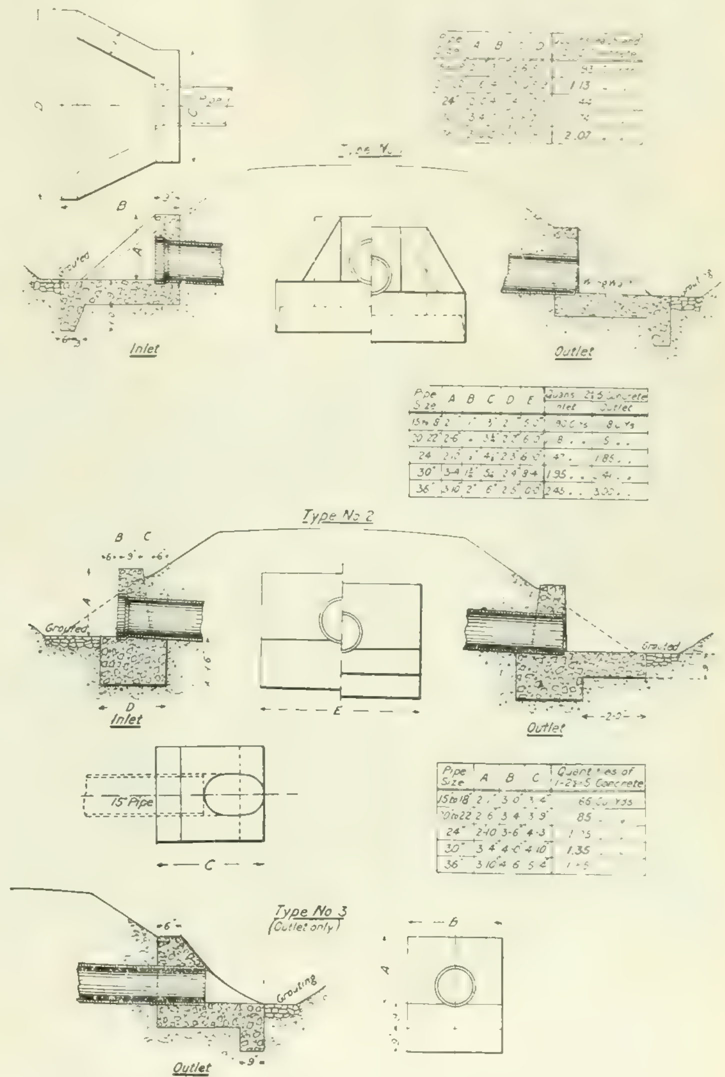


Fig. 9. (b) Details and Quantities for Pipe Culvert End Walls.

Laying.—All surfaces against which the wearing surface will be laid must be painted with bituminous cement. When it is not practical to lay the surface continuously and a joint is unavoidable, the utmost care shall be exercised in lapping cold joints with hot mixture in order to secure a thorough bond and true surface. It shall be carefully and evenly spread by means of hot shovels and iron rakes and in such a manner as to give a uniform and regular grade.

The roadway must present an even, uniform surface, free from waves and depressions that will hold water, and shall be of uniform composition.

Places which remain wet after the surrounding surface has dried must be replaced with proper material.

The proportions of the mixture given as bituminous cement shall signify only the bituminous portion of the asphaltic cement, and the proportions stated shall not include the non-bituminous material.

stone shall meet the requirements as specified under Asphaltic Concrete, and in addition shall pass a one inch circular opening.

The sand and stone shall be heated in approved dryers to a temperature ranging from 250° to 350° F.

Mixing Asphaltic Macadam Binder Course.—The sand and stone at the temperature herein specified shall be thoroughly mixed in an approved batch mixer with bituminous cement hereinafter specified.

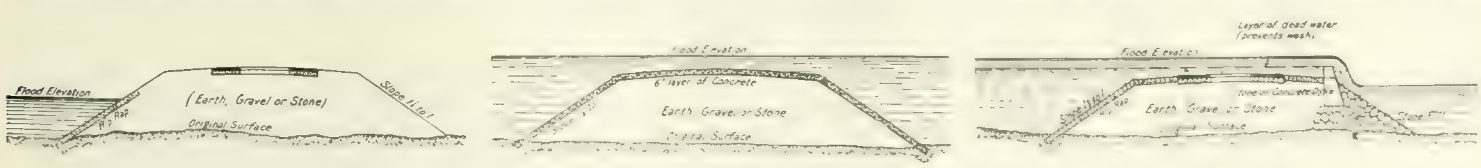


Fig. 10. Methods of Treating Roads Frequently Overflowed.

The mixture having been spread in accordance with specifications shall receive surface compression with an approved light tandem roller of either three or five-ton capacity, after which a small amount of Portland cement shall be swept over the entire surface. Final

ASPHALTIC MACADAM—NO. 1.

Construction of this class shall consist of concrete or telford foundation, a bituminous binder course of at least 1½-in. thickness, and a mechanically mixed asphaltic macadam

The bituminous cement shall be that specified under Class "A"—penetration 50 to 60 (Dow Method). The temperature of bituminous cement, when mixed with hot stone, shall range between 250° and 350° F. The bituminous cement shall be heated in approved melting kettles that will provide for a uniform distribution of heat without burning or coking and shall permit of proper agitation.

Proportions.—The materials complying with these specifications shall be mixed in the following proportions, and measured by weight. Asphaltic cement, 7 to 10 per cent; stone, 50 to 65 per cent; sand, 30 to 45 per cent.

The proper quantity of bituminous cement for the given amount of sand and stone shall be carefully determined by experiments made at the mixing plant. When proportions are determined, they shall be strictly adhered to, unless changed by the State Highway Commissioner. The sand, stone, and bituminous cement for every batch must be carefully measured. Should the binder appear dull from overheating or lack a proper bond, it shall immediately be condemned, removed, and replaced with new material. The upper surface of the binder shall, after rolling and ramming is completed, be parallel with the surface of the pavement to be laid.

When it is not practical to lay the binder continuously and a joint is unavoidable, the cold material shall be cut off in a straight line across the road, and as far back as necessary to insure an even surface of proper thickness. The bituminous binder shall not be laid during wet weather and not unless the foundation is dry.

The wearing surface shall be laid upon the binder course while it is clean, and free from moisture and all loose particles, and if possible,

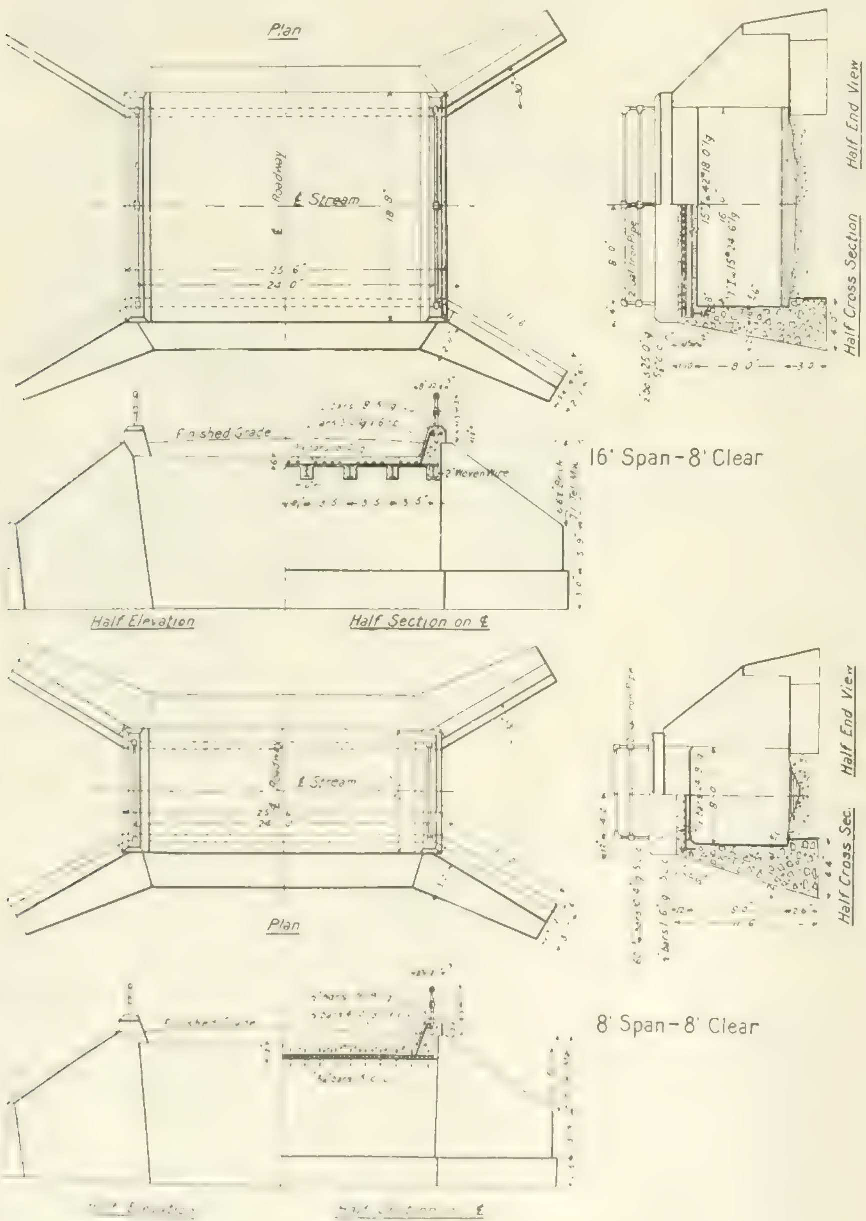


Fig. 11. Reinforced Concrete Culverts of Slab, and Slab and Beam Types.

of bituminous cement, or overheated, it shall be rejected.

Construction.—The bituminous mixture shall be hauled to the road under cover, if necessary [illegible] upon prepared foundation to such a depth that when thoroughly rolled and compressed with a regular 10-ton power tandem roller it shall not be, in any place, less than 1⅛ ins. in thickness. The rolling shall be continued until the binder course ceases to show signs of any further compression. The rolling shall at all times be subject to the approval of the State Highway Commissioner, and the roller shall start from the side lines and work towards the center of road unless ordered.

Should the bituminous binder course not show within 24 hrs. after the rolling of binder course.

The wearing surface shall be composed of bituminous cement specified hereinunder Class A [illegible] to 60 (Dow Method). It shall be heated in approved melting kettles that will permit of uniform distribution of heat without burning or coking and permit of the proper agitation, and shall never be heated to a temperature exceeding 375° F.

The approved sand shall be clean, moderately sharp, hard, free from sewage, clay, loam, and all vegetable matter, and at least 18 per cent of the sand shall pass an 80-mesh screen, and not more than 40 per cent, nor less than 15 be retained upon a 40-mesh screen. The stone screenings shall be hard, durable trap rock, limestone, gneiss, granite, or such other screenings as the State Highway Commissioner may approve. They shall be clean, dry, free from dust, dirt, vegetable matter, and shall pass a ½-in. circular opening.

The filler shall be fine lime dust, Portland cement, or other suitable dust, and must be of such a degree of fineness that all of it will pass a 30-mesh sieve, and at least 75 per cent of it will pass a 100-mesh sieve.

The materials complying with these specifications shall be mixed in the following proportions and measured by weight: Bituminous cement, 7 to 10 per cent; sand, 55 to 65 per cent; stone, 30 to 40 per cent; filler, 8 to 10 per cent.

The mineral aggregate and bituminous cement shall be heated separately in approved dryers and melting kettles to a temperature of 350° F. Materials that have been injured or damaged by overheating shall be condemned. The filler or dust, may, while cold, be mixed with the hot mineral aggregate in the required proportions. The bituminous cement, while at the required temperature and in proper proportions shall be added and the whole mixed in a suitable batch mixer until a thoroughly homogeneous mass is produced.

The paving mixture prepared in the manner described herein shall be hauled to the road under cover and when dumped, if necessary on a platform, shall have a temperature of not less than 250° F. All surfaces against which the wearing surface will be laid, must be painted with bituminous cement. When it is not practical to lay the surface continuously, and a joint is unavoidable, the utmost care shall be exercised in making joints between the hot and cold mixture in order to secure a thorough bond and true surface.

The mixture shall be carefully and evenly spread with hot shovels and iron rakes in such a manner as to produce a uniform and regular grade, and shall be thoroughly compressed to a depth of at least one inch in thickness with an approved tandem power roller weighing not less than 10 tons. The rolling shall continue until all roller marks have disappeared, and the surface ceases to show signs of further compression, and shall be subject to the approval of the State Highway Commissioner.

Immediately after rolling and while the surface of the roadway is still warm, a small amount of Portland cement shall be swept over the surface of the roadway. The roadway must present an even, uniform surface, free from waves and depressions that will hold water, and shall be of uniform composition. Places which remain wet after surrounding surface has dried must be replaced with proper material.

The proportions of the mixture given as bituminous cement shall signify only the bituminous portion of the asphaltic cement and the proportions stated shall not include the non-bituminous material.

ASPHALTIC MACADAM— NO. 2.

Construction of this class shall be laid upon concrete foundation and shall consist of a mechanically mixed asphaltic macadam wearing surface of at least 2 ins. in thickness or as may be designated and shown on the plans.

In this mixture the stone is limited to ¾ in. size, in other respects it is similar to No. 1.

SPECIFICATIONS FOR BITUMINOUS MATERIAL.

ASPHALTIC CONCRETE.

Class "A." Hot Mix.—Bituminous cements must be uniform, homogeneous, and free from water. Asphalt cements must be free from tar or tar products.

General Conditions.—The asphaltic cement shall be prepared by refining crude native bitumen which is liquid or which becomes liquid upon the application of heat. The softening of native bitumens which are too hard for use before refining shall be accomplished by the addition of a flux prepared from native liquid bitumen. In no case, however, shall flux of paraffine base be used to the extent of more than 25 per cent of the resulting asphaltic product. Crude liquid bitumen shall not be reduced in refining to less than 30 penetration, and in softening of such bitumen only semi-asphaltic or asphaltic flux shall be used.

The crude materials used, the preparation, and the refining of asphaltic products shall be subject to such inspection at the refineries by the authorized representatives of the Pennsylvania State Highway Department, as will ensure compliance with these specifications.

The asphalt cement under this class shall comply with the following requirements:

(1). It shall have a consistency of 50 to 90 penetration at 77° F. (No. 2 N.—100 gr.—5 sec.)

(2). The ratio of consistency at 77° F. (No. 2 N.—100 gr.—5 sec.) to consistency at 32° F. (No. 2 N.—1 min.—200 gr.) shall not be greater than as 5 is to 1.

(3). When 20 grams of the asphaltic cement are maintained at a uniform temperature of 325° F. in an oven for 5 hrs. in a tin box 2¼ ins. in diameter and ¾ to 1 in. deep, there must not be volatilized more than 5 per cent by weight of the bitumen present, and the residue shall have a penetration at 77° F. of not less than one-half of the original penetration.

(4). The asphaltic cement considered apart from its native non-bituminous matter shall be soluble in cold carbon tetrachloride to the extent of 99 per cent or more.

(5). The asphaltic cement shall not flash at less than 350° F., closed cup.

(6). A briquette of the asphaltic cement or the pure bitumen thereof, shall, at 77° F. and at 50 penetration, elongate at least 40 cm. before breaking. If the consistency of the cement as used, be above 50 penetration, there shall be required additional ductility of 1 cm. for each unit of penetration above 50.

(7). The yield of fixed carbon from the bitumen of the asphaltic cement shall accord with either of the following: (a). It shall not exceed 15 per cent. (b). It shall not exceed by over 3½ per cent, the quotient of the fixed carbon from the crude bitumen, divided by the yield of asphalt therefrom.

(8). If the asphalt cement contains more than 5 per cent of non-bituminous material insoluble in cold carbon disulphide, it shall when used, be subjected to sufficient mechanical agitation to ensure thorough uniformity.

ASPHALT MACADAM.

Class "B." Penetration Method.—The asphaltic cement under this class shall accord with the general conditions under Class "A," and shall, in addition meet the following requirements:

(1.) It shall have a consistency of 60 to 110 penetration at 77° F. (No. 2 N.—5 sec.—100 gr.).

(2.) The ratio of consistency at 77° F. (No. 2 N.—5 sec.—100 gr.) to consistency at 32° F. (No. 2 N.—1 min.—200 gr.) shall not be greater than as 5 is to 1.

(3.) When 20 grams of the asphaltic cement are maintained at a uniform temperature of 325° F. in an oven for 5 hrs. in a tin box 2¼ ins. in diameter and ¾ to 1 in. deep, there must not be volatilized more than 8 per cent by weight of the bitumen present, and the residue shall not be reduced to less than 40 penetration, at 77° F.

(4.) The asphaltic cement considered apart from its native non-bituminous matter shall be soluble in cold barbon tetrachloride to the extent of 99 per cent or more.

(5.) The asphalt cement shall flash at not less than 350° F., closed cup.

(6.) A briquette of the asphaltic cement or the pure bitumen thereof, at 77° F. and at 60 penetration, elongate at least 30 cm. before breaking. If the consistency of the cement as used be above 60 penetration, there shall be required additional ductility of 2 cm. for each 5 units of penetration above 60.

(7.) The yield of fixed carbon from the bitumen of the asphaltic cement shall accord with either of the following: (a) It shall not exceed 15 per cent. (b) It shall not exceed by over 3½ per cent the quotient of the fixed carbon from the crude bitumen, divided by the yield of asphalt therefrom.

(8) If the asphalt cement contains more than 5 per cent of non-bituminous matter insoluble in cold carbon disulphide, it shall, when used, be subjected to sufficient mechanical agitation to ensure thorough uniformity.

METHODS CLASS "A" and "B."

1 and 2. Consistency shall be measured by penetrometer, equipped with a "cambric needle," and registering in hundredths cm., at the temperatures and with the constants stated.

3. Loss at 325° F. is taken in a New York testing laboratory air oven or its equivalent in which a uniform temperature is maintained throughout. The temperature is recorded by a thermometer, the bulb of which is immersed in bitumen in the oven. The penetration of the residue from the volatilization test will be made after same has been liquified at a gentle heat and thoroughly stirred.

4. The solubility in cold carbon tetrachloride will be made upon a sample of from one to two grams covered with 100 ccs. of chemically pure carbon tetrachloride and allowed to stand for 15 hours protected from the light before filtering. Correction is made for any mineral matter passing through the filter.

5. The flash test is made in a New York state closed oil tester or its equivalent.

6. Briquettes for ductility test shall be in accordance with the District of Columbia standard. Test is made at 77° F., and the briquette is pulled apart at the rate of 5 cm. per minute, by any apparatus capable of effecting a uniform rate of elongation. When it is necessary to prepare pure bitumen for ductility test, a suitable quantity is dissolved in freshly distilled carbon disulphide, allowed to stand over night, filtered, and the filtrate distilled in a flask with a thermometer. The last remaining traces of carbon disulphide are removed by carrying the temperature of the bitumen in the flask to 450° F. and maintaining there for half an hour or until nothing more comes over. The recovered bitumen is then tested in the usual manner.

7. Fixed carbon is determined according to the standard method given in the Journal of American Society, 1899. The test shall be made with a Bunsen burner, preferably the Chaddock type, burning illuminating gas. The burner is regulated so that the entire length of the flame is 20 cm., and its length from the tip of the burner to the top of the inner cone is 8 cm. The crucible shall be placed so that its bottom is ½ cm. above the top of the inner cone.

LIQUID ASPHALT—CLASS "C."

Material under this class shall meet the following requirements:

(1) The liquid asphalt cement shall be of such consistency as to flow readily at 77° F.

(2) When a portion of liquid asphalt is thinly spread over a sheet of sized paper and allowed to remain in a vertical position for twenty-four hours at room temperature (65 to 75° F.), the film of asphalt remaining shall be dry and adhesive, and not liquid or oily.

(3) The liquid asphalt cement shall be soluble in cold carbon disulphide to the extent of 99.5 per cent.

(4) Upon distilling approximately 100 ccs. of the liquid asphalt in an Engler flask up to 700° F., the percentage of residue in the flask shall be between the limits 70 to 85 per cent by weight.

(5) The asphaltic residue from the above distillation shall have a consistency of not over 250 or less than 30 penetration at 77° F.

(6). The asphaltic residue from the above distillation shall, when tested for ductility in accordance with section 6, "Class A." elongate not less than 40 cm. before breaking.

(7). The gravity of the distillate from test No. 4 shall not be less than 50° Bc., at 60° F.

Note.—The distillation test No. 4 shall be conducted in an Engler flask described in "Petroleum and Its Products," by Sir Boverton Redwood, in Volume 2, page 535. The thermometer shall be immersed in the liquid, and the distillation conducted at the rate of 2½ cm. of the distillate per minute, or at an increasing temperature rate of ten degrees per minute.

LIQUID ASPHALT—CLASS "D."

(1). The specific gravity of the liquid bitumen shall at 60° F. be not less than .92.

(2). Upon distilling approximately 100 ccs. of the liquid asphalt in an Engler flask up to 700° F., the residue shall not exceed more than 90 per cent by weight, and shall have a specific gravity of at least .980 at 60° F.

(3). The viscosity of the bituminous material at 115° F. shall not exceed 480 seconds, Engler instrument, the first 50 ccs. only being recorded.

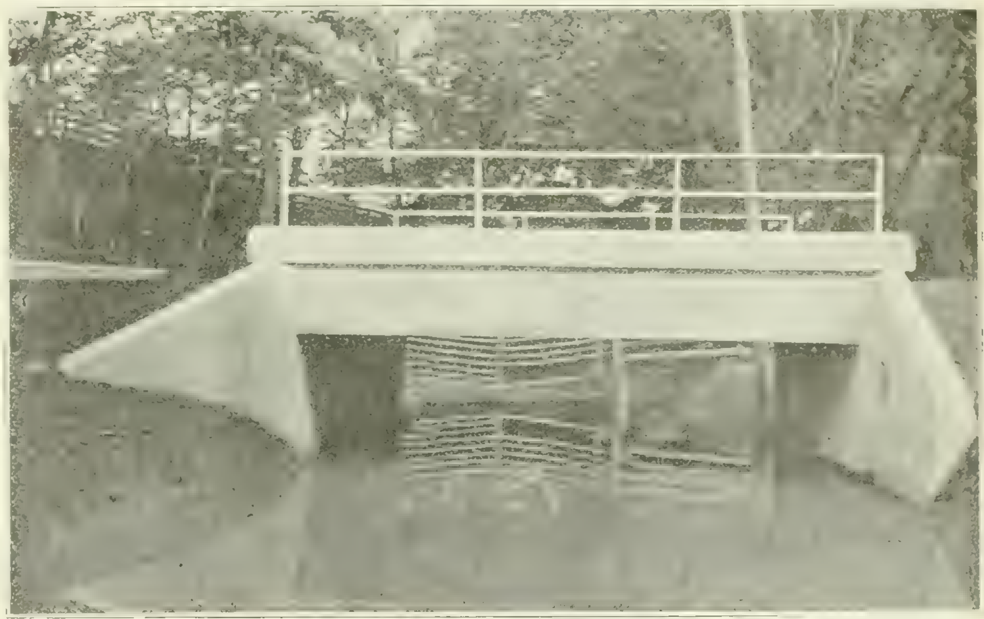


Fig. 12. View Showing Typical Slab and Beam Concrete Culvert.

CLASS "T."

The bituminous material "T" shall be a tar having the following characteristics:

(1). The tar shall contain no water and not more than 2 per cent of mineral matter or dirt.

(2). It shall be uniform in character, appearance and viscosity.

(3). It shall not contain over 0.5 per cent of water soluble material.

(4). It shall have a specific gravity of between 1.17 and 1.25 at 77° F.

(5). It shall contain no body that distills at a lower temperature than 338° F., at least 10 per cent by weight of creosote oil—having a gravity of not less than 1.03 at 60° F. shall distill between 338° F. and 568° F. and at least 75 per cent by weight of pitch having a melting point not higher than 165° F., shall remain after all bodies up to 568° F. have been distilled.

(6). On extraction with C. S. 2 it shall not contain more than 25 per cent of free carbon, nor less than 15 per cent.

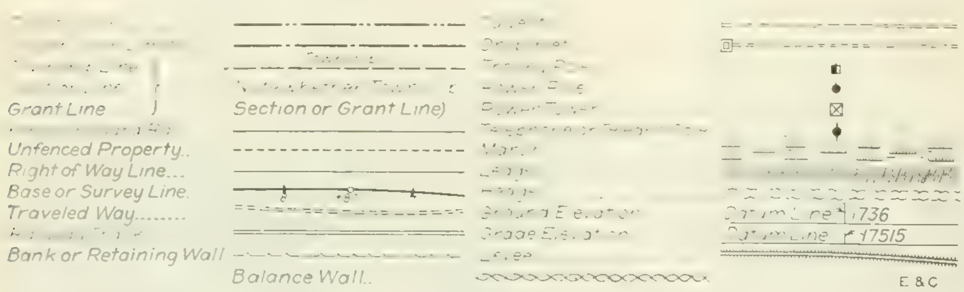
This article has been prepared from reports and information furnished us through the courtesy of the officials of the Pennsylvania Highway Department.

Method of Preparing Plans Used by the California Highway Commission.

To secure uniformity, standard methods of preparing road plans have been adopted by the California Highway Commission, A. B. Fletcher, highway engineer. A short discussion of them in a recent publication of the commission is interesting and is given here.

The data turned in by the surveyors include location notes of all structures, tracks, trees, poles, and other similar objects, within 150 ft. of the base line; cross-section notes of the road taken at intervals of 50 ft. or less; computations of a traverse, one each side of which is the base line of the survey; and notes as to soil, drainage, location of desirable materials, railroad sidings, local wages, etc. So far as may be these notes are computed and checked before they leave the field.

CONVENTIONAL SIGNS



Conventional Signs Used on California Highway Commission Plans.

On receipt of the notes in the division office the plan and profile are plotted on 30-in. brown detail paper of such lengths as are convenient for flat filing when folded, and the cross-sections are plotted on specially prepared cross-section paper. This specially prepared cross-section paper is of map bond, 20-in. by 30-in. sheets, printed in orange ink from a "zinco" engraving with co-ordinate lines one-tenth of an inch apart. At one end the margin reserved is wider for the purpose of binding.

Contract Plans.—The contract plans are blue prints from the layout plans and from the cross-section sheets, but for the convenience of the engineering force pocket size photographic reductions of the layout plans are made also. These photographic reductions are 4¾ ins. by 7½ ins. in size, and are printed on glossy velox paper. They are remarkably legible, even without the use of a reading glass.

The layout plans, contract plans, and cross-section sheets are all bound in temporary binders for their protection and preservation. These binders are composed of two heavy covers hinged by cloth to metal strips, and five bolts for clamping together the metal strips and the intervening plans.

Binding.—The binders are canvass and leather finished, and the back or lower cover is sufficiently long to permit of the steel binding strip being folded in between the plans and the cover, thereby concealing the bolt heads and insuring a non-scratching lower surface. The nut used is a brass, round-headed one, with a threaded sleeve projecting one-half inch through the top binding strip, and the screws are stove bolts of the many varying lengths. Great elasticity is obtained by this combination. Binders of similar design, but of lighter weight and clamped with paper fasteners, are used for the "form" papers, etc. They are of two sizes, letter size and note size.

Filing System.—The basis of most of the filing is the road location, whether the filing be of plans, field books, papers, letters, or something else. The state has been divided into seven divisions, each of which is designated by a roman numeral. Each of the trunk lines or routes as determined is given an arabic number, and within each county these routes are divided into convenient contract lengths and lettered. The division number, county abbreviation, route number, and section letter are entered on all plans and papers, and they are filed according to this entry.

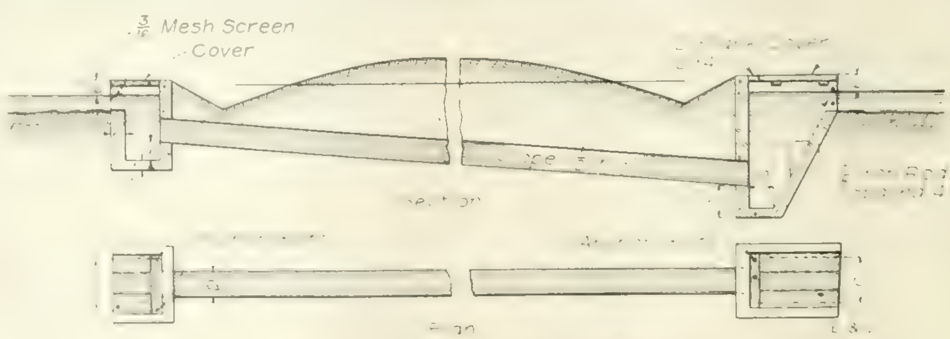
Siphon Culverts for Irrigation and Drainage Ditch Crossings.

(Staff Article.)

In sections of the country where extensive irrigation is necessary and where the country roads are crossed at frequent intervals by irrigation laterals a type of siphon culvert crossing has been used in some cases, which possesses advantages. For irrigation laterals this type of crossing is often the only feasible one.

The conditions to be met, in the case of irrigation lateral crossings are, briefly, as follows: At points where the flow line of the lateral is at a higher elevation than the bottom of the road drainage ditch provision must be made to prevent flooding of the road ditches. It is usually desirable to exclude road drainage water from irrigation laterals. A type of crossing must be provided which is certain in action and not subject to stoppage by drift which may accumulate in road ditches. The cost must be low when frequent crossings are necessary.

To meet these conditions the crossing illustrated was designed and used by the Utah Highway Commission. It will be noted that the depth of the pipe which forms the bottom of the siphon is controlled by the depth of the road drainage ditch on the side of the road opposite to the flow of the water in the irrigation lateral. A good slope in the pipe is provided. The inlets at either end have sufficient depth below the bottom of the pipe to permit some accumulation of silt. These inlets are constructed of concrete, the same material being frequently used for the pipe; although vitrified pipe is permitted. The bottom of the siphon is at sufficient depth so that vitrified pipe is well protected from breakage by road traffic, but some danger from freezing exists.



A Siphon Culvert Ditch Crossing.

The dimensions of the inlets for various sizes of pipe in the design illustrated are as follows:

[illegible]	B. Ins	C. Ins.
[illegible]	18	24
[illegible]	18	24
[illegible]	24	30
[illegible]	24	30
[illegible]	30	36
[illegible]	30	36

It is needless to state that the basins at the inlet and outlet ends require frequent cleaning for the successful operation of the siphon.

The same type of construction may be used where it is necessary to convey road drainage water under a lateral. In this case the length of pipe is much shorter and the size of pipe used should be quite large.

Administration of Highway Maintenance.

The following notes on the essential features of administration of affairs related to the maintenance of country roads were abstracted from a paper by W. H. Maxwell, before the Institution of Municipal and County Engineers.

Good administration, particularly with reference to the disposition of labor, materials, and plant, is one of the leading factors in highway maintenance essential to efficient and economical work. In urban areas suitably placed central depots, with railway sidings into which materials can be delivered direct, are a great convenience, and involve the minimum haulage of materials. In these days of frequent labor disturbances, as in the case of the railway, dock, and colliery strikes, the necessity of getting all materials on the spot well in advance of requirements has been greatly emphasized of late, in order that annoying delays may be avoided. Delay in any form means increased cost of the work in hand, frequently entails loss of good weather suitable for the work, and always gives regrettable inconvenience to the general public.

Wherever much work is in progress constant inspection and supervision is a good investment, and to this end the inspecting staff should be provided with appropriate means of quick and convenient locomotion. Local materials should be employed wherever of suitable quality, but some counties are particularly deficient in good quality stone suitable for first-class roads carrying considerable traffic.

Over-centralization of the practical work of road maintenance is not desirable. It is apt to lead to so-called "red-tape" methods, costly delays, and lack of individual attention. Road maintenance should be carried out through the responsible highway authorities by the direct employment of labor, thus ensuring a local interest in the work, close supervision and prompt attention, which is almost impracticable under any too distant and highly centralized system of administration.

Stereotyped standardized methods in many matters connected with road-making and maintenance are to be deprecated. Such methods destroy the useful application of personal experience and judgment to individual cases, and so lead to mere routine and lack of interest. It is unlikely that there can ever be any universal solution of the road problem, inasmuch as local conditions and requirements, character and extent of the traffic, local facilities of obtaining suitable materials, considerations of cost and the like must ever be deciding factors in arriving at the most suitable and satisfactory mode of treatment in each particular case.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

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The Salvage of Road Hauling Machinery.

A few years ago the cost of hauling over country roads was thought to be well established for ordinary conditions. Accurate cost data collected under many varying conditions established a reasonably accurate ton-mile estimating cost of hauling with teams; accepted for earth roads as varying between 20 and 30 cents. New developments in machinery have, however, placed at the disposal of the road contractor and superintendent other methods of hauling which, under proper conditions, are more economical than team hauling. The steam and gasoline tractor with wagon train, the motor truck alone or with trailers, and the industrial railway, each have a definite field in which their use results in a saving over any other method that might be employed.

It is not our purpose to discuss these methods of hauling or their limitations. Indeed, any but a very general discussion is valueless unless applied to a particular problem. Moreover, each construction problem is a study in itself and similar conditions do not recur with any great degree of frequency. A single factor introduced in the problem may change plant requirements completely. For example, an unusually large amount of shallow earth excavation might call for so many teams that team hauling of materials would be most profitable. Or, vice versa, a large brick or concrete construction job with light grading would be handled most economically by the use of mechanical hauling equipment.

Experience has demonstrated that under most conditions the use of machinery results in lower hauling costs and a saving in time. There exists, however, one serious drawback to its more universal use. The first cost and depreciation of mechanical hauling equipment is greater than that of teams and *the salvage value is proportionately less,* not only in the actual cash but in readiness of market. What causes contribute to this condition and how may the condition be remedied? First cost is of small moment if the output is increased and a ready salvage market established. Depreciation is a definite factor to consider in first cost. Manifestly the problem turns on the question of plant salvage.

Probably the most serious danger that confronts the modern road contractor is that of over equipment. Plant suitable for most economical results in road building is available but it is expensive and a large amount is required. The amount of capital invested in plant that must remain idle, from the character of the work, for a portion of the year is large. Unless this plant can be disposed of at a reasonable price upon the completion of the work, profits are tied up in equipment that may be of small value on the next work undertaken, or may become obsolete.

To alleviate this condition as well as to promote sales many reliable machinery manufacturers have established service departments and will redeem used machinery. The principle of this plan is excellent. Second hand machinery is, undoubtedly, worth more to the manufacturer than to anyone else. When rebuilt it is practically as good as new and, in the case of well standardized machines, sometimes better. One manufacturer of automobiles, noted for the profitableness of his business, has a definite policy of replacement, allowing a substantial value for old machines. This practice is of great assistance to contractors and should be developed. The proper salvage of plant used in road haulage is by no means a new idea, but the importance of its consideration with reference to hauling equipment will become more apparent with the increased use of machinery for this work which will undoubtedly come in the next few years.

The Ownership of Vault Space under Streets.

The ownership of vault space under streets has long been a matter of dispute between cities and abutting property owners. In cities where the street ownership is in question the regulation of vault space by these municipalities has often proved difficult. Where vault rights have been granted it has been necessary for the cities to supervise the construction of retaining walls, which in a measure has made them responsible for the safe construction of these walls. Where failures have occurred, resulting in the sinking of streets and often in damage to adjacent structures, the question of damages has often been a troublesome one.

The New York Supreme Court recently handed down a decision defining the ownership of the vault space under streets, the fee of which has always been retained by the abutting property owners. The right of the city to demand payment for vault space under Cortlandt St. was denied by the tenants and owners of the property at the northwest corner of that street and Broadway. A judgment was entered against the city at an earlier trial and the case was appealed. In a unanimous decision the Appellate Division upheld the right of the city to regulate and demand payment for vault space, irrespective of the street ownership, the decision being based on the broad grounds of public interests.

In handing down this decision, Justice Laughlin made the following comments:

> The city owes a duty to the traveling public. That duty requires the city to supervise the construction of vaults, and, regardless of whether the city does or does not own the fee of the street, in permitting an abutting owner to construct a vault, the city has the right to exact a reasonable fee to cover the expense to which it will be subjected in supervising the construction of vaults and in inspecting them. If, as the trial court decided, the plaintiffs have not the right to maintain the vaults without a permit from the city they are not entitled to enjoin the city from interfering with or filling up the vaults.
>
> The sound doctrine must be, and is, that even though the abutting owner owns the fee he may not open or undermine the street without a permit from the local authorities, who, if they determine that it is compatible with the public interests to grant it, may even, where the applicant owns the fee, impose such reasonable conditions as will protect the municipality against liability to third parties and indemnify it against the expense to which it will be subjected.

Competitive Bidding Between Hired Forces of the City of Philadelphia.

The report of the Bureau of Highways of Philadelphia states that an innovation was tried out during the latter part of 1913 by having the foremen of the bridge repair gangs bid against each other on several jobs. The plan seems to be a successful one and capable of development in the way of having the city's employes bid against the contractors on repairs to bridges, where it is practical.

The scheme had the effect of placing the foremen more on their mettle and increasing their pride in the work, and much to their credit even the ordinary laborers in the different gangs took more than a passing interest in the job after it was awarded to their foremen. The competition was keen and in one instance $10 represented the difference in the range of the lump sum prices submitted by four men on a job, which the low bidder agreed to do for $410.21. The actual cost of the work was $393.96, approximately four per cent below the figure submitted. This method is bound to create a healthy condition in the personnel of the repair corps if it is properly encouraged, and will ultimately redound to the benefit of the city if the scheme can be perfected so as to have the men bid against the contractors on the more extensive repair jobs.

The possibilities here may be appreciated when it is known that on account of the hazard attending this kind of work, contractors invariably add to their percentage of profit a sum sufficient to cover all contingencies.

Unpublished Engineering Studies.

On several occasions we have urged, in these columns, that the results of departmental experiments and studies be given publicity through the medium of the annual report of the municipal department under which such studies were made. We are well aware that such investigations often are of such nature that their results are too voluminous to be given space in the annual report which must be limited in size. There are other well recognized mediums for the dissemination of such information, however, and it is to these that we desire to call attention. In short, it is not from a lack of a suitable vehicle that the results of studies are not given publicity, but because those who could make them public do not take the trouble to do so.

To illustrate, in all of the engineering departments of our larger cities the principal engineering assistants conduct, at times, correspondence with those in similar positions in other cities with a view of learning the experiences of the latter in regard to certain engineering questions.

Thus an engineer in the sewerage department of a certain city may send out, to other cities, a list of questions drawn to bring out the results of their experience in the use of patented jointing materials for use in laying pipe sewers. The fraternal spirit existing between the engineering departments of the various cities ensures satisfactory co-operation in such enquiries. The replies to the questions are usually made the basis of a report by the department conducting the study. These reports, as a rule, are not given the publicity they deserve. They are used by the department preparing them and are filed after they have served their immediate purpose. The profession as a whole is seldom made any better informed as a result of the investigation. This is an unfortunate and unnecessary condition and should be corrected.

The engineering journals are continually looking for just such matter for publication. While the journals make many efforts to collect at first hand information of this character they are less successful than the practicing engineer, since they do not enjoy the same co-operation acccorded by one engineering department to another. For this reason investigators will find the several journals willing to give publicity to the results of these studies.

Again, these matters may properly be made the subject of papers before engineering societies. The desirability of papers on specific rather than general subjects is well recognized by the secretaries of the societies and carefully prepared papers giving the results of special studies and investigations are usually eagerly accepted by societies for discussion and publication.

Of course some investigators make use of the agencies here suggested, but many others do not. All are urged to do so.

An Interesting Development in Municipal Research: A University's Report on a City's Pavements.

Among the first works undertaken by the newly created Municipal University of Akron, Ohio, was an investigation of and report upon the pavements of Akron, made at the request of the city council. The council requested a report on the present condition, cost and durability of Akron pavements; the cost of various types of pavements and the adaptability of each type to different classes of traffic; and the difference, if any, between the contract and day labor system of performing municipal work—a large order and inclusive.

In approaching this problem considerable ingenuity was used. To secure necessary data, in addition to the collection of reports and information from other cities, the following men and organizations were employed and paid from funds appropriated by the University for this work.

Two co-operative students in civil engineering from the University of Cincinnati, inspected every pavement in the city and classified it as to its present condition.

Four students from the University of Akron tabulated the data thus obtained.

The New York Bureau of Municipal Research made a complete report upon the paving specifications.

The American Society of Civil Engineers made a search in its library and furnished a bibliography on paving materials covering publications made during the last two years

In reporting upon the proper paving materials to be used and whether pavements should be laid by contract or by labor directly employed by the city, some general information is given but the report states that the decision of this problem rests with the city officials. In the language of the report, "The university is primarily an educational institution and cannot assume to dictate the policies of the city administration." In the final summing up, a number of excellent recommendations are submitted for the consideration of the city council.

The interesting point in connection with this report, and others of a like nature prepared by research bureaus, is the consulting nature of the work. The question arises as to the present status of consulting municipal engineers; and the tendency toward the development of "research bureaus" for consulting work in the field of municipal organization, engineering and efficiency, being shown in increasing measure in the last few years. What is the future of this field of work? It is a commonly accepted notion among engineers that there is small demand for expert and accurate advice upon municipal problems, in short, that the field is limited in extent. Perhaps this is true; and yet, the appointment of numerous investigating commissions, the employment of research bureaus to prepare reports on work accomplished and various other "reform measures" adopted by cities throughout the country; the almost universal, as it were, locking of the stable door after the horse is gone, point out most clearly the great need of expert planning of a city's work.

The information desired by the city council of Akron is similar to that which every city council should have with reference to its own particular paving problems. Such information is necessary to the proper apportionment and administration of a city's finances, and to the economic solution of the larger problems of construction. How these data have been secured in the past is a mystery. Perhaps, in addition to other duties as numerous as those of the illustrious Pooh-Bah, the city engineer has been called upon to furnish information, often to his undeserved undoing.

The development of the idea of the application of business methods to municipal problems has been rapid in the past few years. We have city planning commissions with their experts, city managers, research bureaus and other plans for securing efficiency in planning work. The field evidently exists, the lines along which it will develop we do not venture to predict.

BRIDGES

Design Features of the Approaches of the North Side Point Bridge, Pittsburgh, Pa.

(Staff Article.)

In our issues of March 25 and June 17 we described the design features of the two 531-ft. steel spans and of the three river piers of the North Side Point Bridge, Pittsburgh, Pa. In this issue we shall consider the design features of the north and south approaches of this bridge.

NORTH APPROACH.

The north approach consists of a series of six reinforced concrete arches of varying spans and a long fill between concrete retaining walls. The total length of the approach is about 865 ft. The clear spans of the arches are: 85 ft. 0 in.; 73 ft. 6 ins.; 63 ft. 5 ins.; 54 ft. 8 ins.; 47 ft. 0 in., and 40 ft. 4 ins.; the longest span being adjacent to the 531-ft. river span. These arches rest on concrete pile foundations. The retaining wall construction has a length of about 442 ft. and an extreme width of 55 ft. 6 ins. Figure 1 shows an elevation of the north approach. This drawing indicates the type of construction and gives the general dimensions of the various units of the approach. It will be noted that the approach is on a 4.9383 per cent grade. To facilitate drainage the inside faces of the retaining walls are lined with a 9-in. course of broken stone. A system of tile drains near the bases of the walls is also provided.

The retaining walls rest on an old fill, which was formed by general dumping. They are [illegible] with a vertical expansion joint at each pilaster. A horizontal tongue-and-groove joint is also provided between the coping and the body of the wall to prevent the cracking of concrete along this line.

To decrease the lateral pressure of the fill against the reinforced concrete retaining walls it is proposed to use granulated blast-furnace slag for the filling behind them, although the central portion of the fill is earth. The weight of the granulated slag is about 55 lbs. per cubic foot, and it can be obtained at a comparatively cheap price in the Pittsburgh district.

SOUTH APPROACH.

The south approach has a total length of about 913 ft. It consists of two arches, having clear spans of 73 ft. 6 ins. and 85 ft. 0 in., and about 728 ft. of retaining wall construction. This approach is on about a 5 per cent grade. The portion adjacent to the river span has an extreme width of 55 ft. 6 ins. This width continues for about 175 ft., where the approach forms a "Y," one branch of which joins the approach to the adjacent Point Bridge over the Monongahela River.

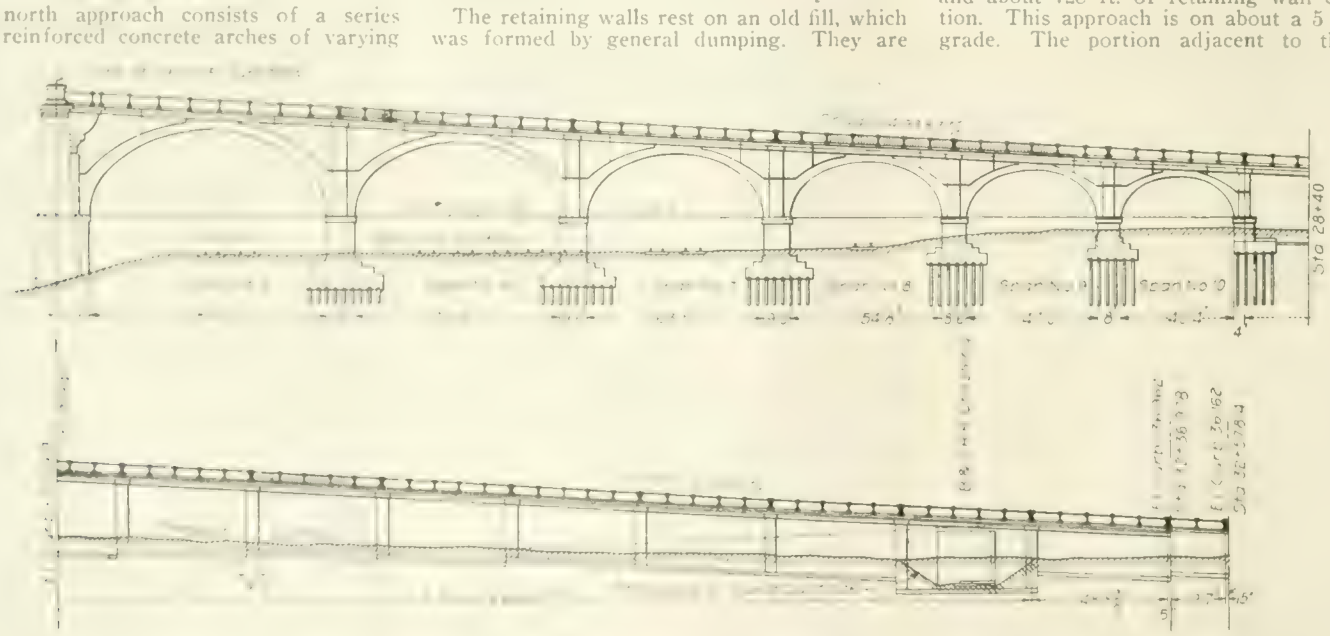


Fig. 1. Elevation of North Approach of North Side Point Bridge, Pittsburgh, Pa., Showing Type of Construction.

while the other continues in a direction approximately parallel to that river. The latter branch has an extreme width of 72 ft. and continues for a distance of about 623 ft. beyond the "Y." The roadway of this part of the approach has a clear width of 48 ft., while that between the "Y" and the river span has a clear width of 36 ft. The pilasters of the retaining walls are spaced 41 ft. apart on centers, a vertical expansion joint being provided in the retaining wall at each pilaster. On account of the shape of fill, upon which this approach is built, the heights of the retaining walls on the two sides of the roadway are generally unequal. Figure 2 shows an elevation and a plan of the portion of the south approach between the "Y" and the river pier.

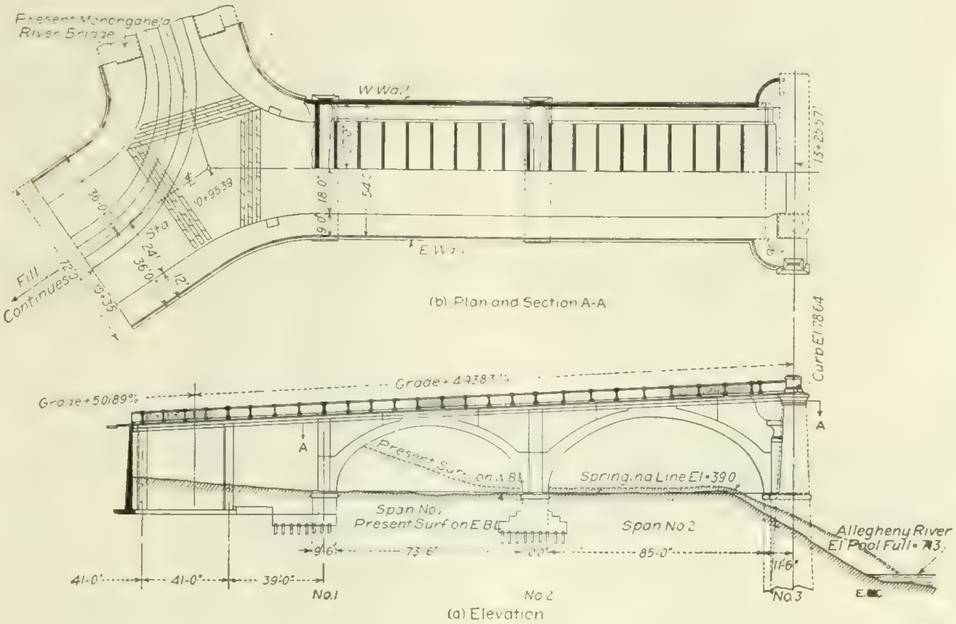


Fig. 2. Arch Spans and Y Connection of Solid Fill of South Approach of North Side Point Bridge.

TYPES OF RETAINING WALLS.

As the height of the retaining walls varies greatly at different points three types of retaining walls are used.

Up to a height of about 15 ft. the gravity type of retaining wall is used. Figure 3 (a) shows a cross section of this type of wall, designed for a height of 15 ft. The concrete in the coping is a 1:2:4 mix, while that in the remainder of the wall and in the footing is a 1:2½:5 mix. The drawing shows the details of the construction joint between the coping and the body of the wall. To facilitate drainage the inner face of the wall is lined with a 9-in. layer of broken stone, while a 6-in. pipe drain runs parallel to the wall near its base.

For walls having heights between about 15 ft. and 25 ft. the cantilever type is used. Figure 3 (b) shows a cross section of the cantilever type at about its highest point in the wall. The section shown has a height of 25 ft. 3¼ ins. and a width of base of 4 ft. The footing has a width of 11 ft., projecting 3 ft. beyond the face and 4 ft. beyond the back of the wall. Below a plane 6 ft. above the bottom of the footing the concrete is a 1:2½:5 mix; above this plane a 1:2:4 mix is used. The arrangement and sizes of the reinforcement are shown in the drawing.

For walls having a height greater than 25 ft. the counterfort type is used. Figure 3 (c) shows a cross section of the counterfort type at about its highest point in the wall. The section shown has a height of 37 ft. 4½ ins. and a width of footing of 14 ft. 11 ins. The counterfort extends from the rear face of the footing to the under side of the coping. The concrete in the footing is a 1:2½:5 mix, while that in the body of the wall, the coping and the counterfort is a 1:2:4 mix. The counterforts for the retaining walls are spaced about 13 ft. 8 ins. on centers; their thickness varying with the height of the wall.

The 9-in. course of broken stone, which is placed against the back of all retaining walls, contains stones ranging in size from 3 to 9 ins. The 6-in. terra cotta tile drains are laid with open joints, and are bedded in a 2-in. layer of clay near the rear of the footings.

ARCH SPANS.

The north approach contains six arches having clear span lengths as follows: 85 ft. 0 in.; 73 ft. 6 ins.; 63 ft. 5 ins.; 54 ft. 8 ins.; 47 ft. 0 ins., and 40 ft. 4 ins. The south approach contains two arches having clear spans of 85 ft. 0 in., and 73 ft. 6 ins. All arches have solid arch rings, the intrados radii being

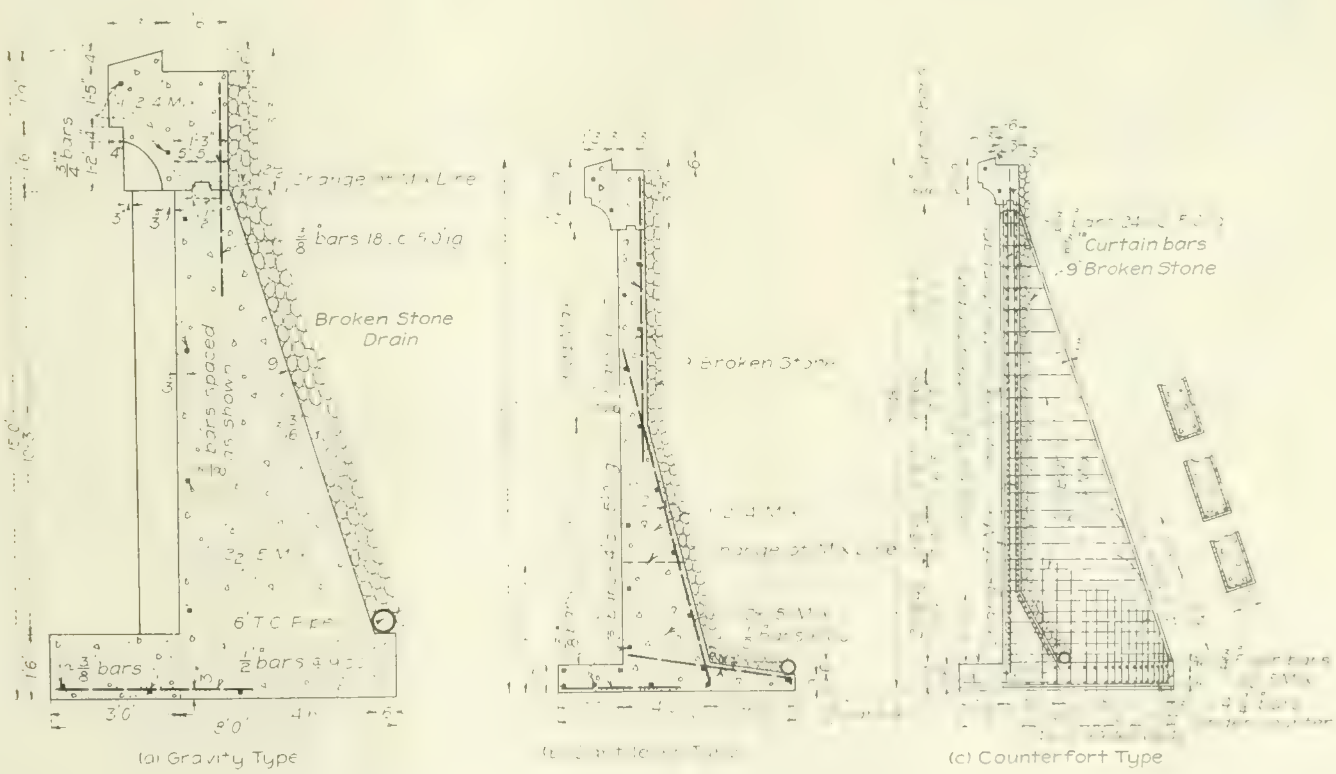


Fig. 3. Types of Retaining Walls Used in the Approacnes of North Side Point Bridge.

three-centered. The arch rings are constructed in monolithic strips between construction joints at the abutments and piers. The transverse spandrel walls have construction joints just above the arch ring, except for the walls near the center of the arch, where their heights are small. These spandrel walls are continuous under the roadway and are connected by means of a reinforced concrete strut to the continuous longitudinal spandrel walls under the outer face of the sidewalk. The arch ring is reinforced with a transverse lattice truss under each spandrel wall. The following table gives the general dimensions of each arch span:

Clear span, ft. ins.	Rise of arch, ft. ins.	Thickness of arch at crown, ft.
85—0	[illegible]	3.49
73—6	26—2½	3.18
63—5	22—7½	2.93
54 [illegible]	[illegible]	2.70
47 [illegible]	[illegible]	2.52
[illegible]	14—4½	2.35

Fig. 4, a). At the expansion joints the spandrel walls are capped with channels, with their flanges turned downward.

The facing ring of the arch, which supports the outside of the sidewalk slab, is shown in Fig. 4 (c). This drawing also shows a portion of a transverse lattice girder and indicates the I-beam tie which connects the coping of the facing ring to the floor slab.

The piers which support the arches rest on Raymond concrete pile foundations. Figure 5 (p. 199) shows a half plan, an end elevation and a half longitudinal section of the pier supporting one end of an 85-ft. arch span. These drawings give the principal dimensions of the pier and indicate the reinforcement at the bottom of the footing. The average settlement of eight piles, due to a 60-ton test load, left in place for 36 hours, was 3-16 in.

Both approaches are provided with a structural iron railing, with cast-iron posts spaced from about 9 to 11 ft. apart.

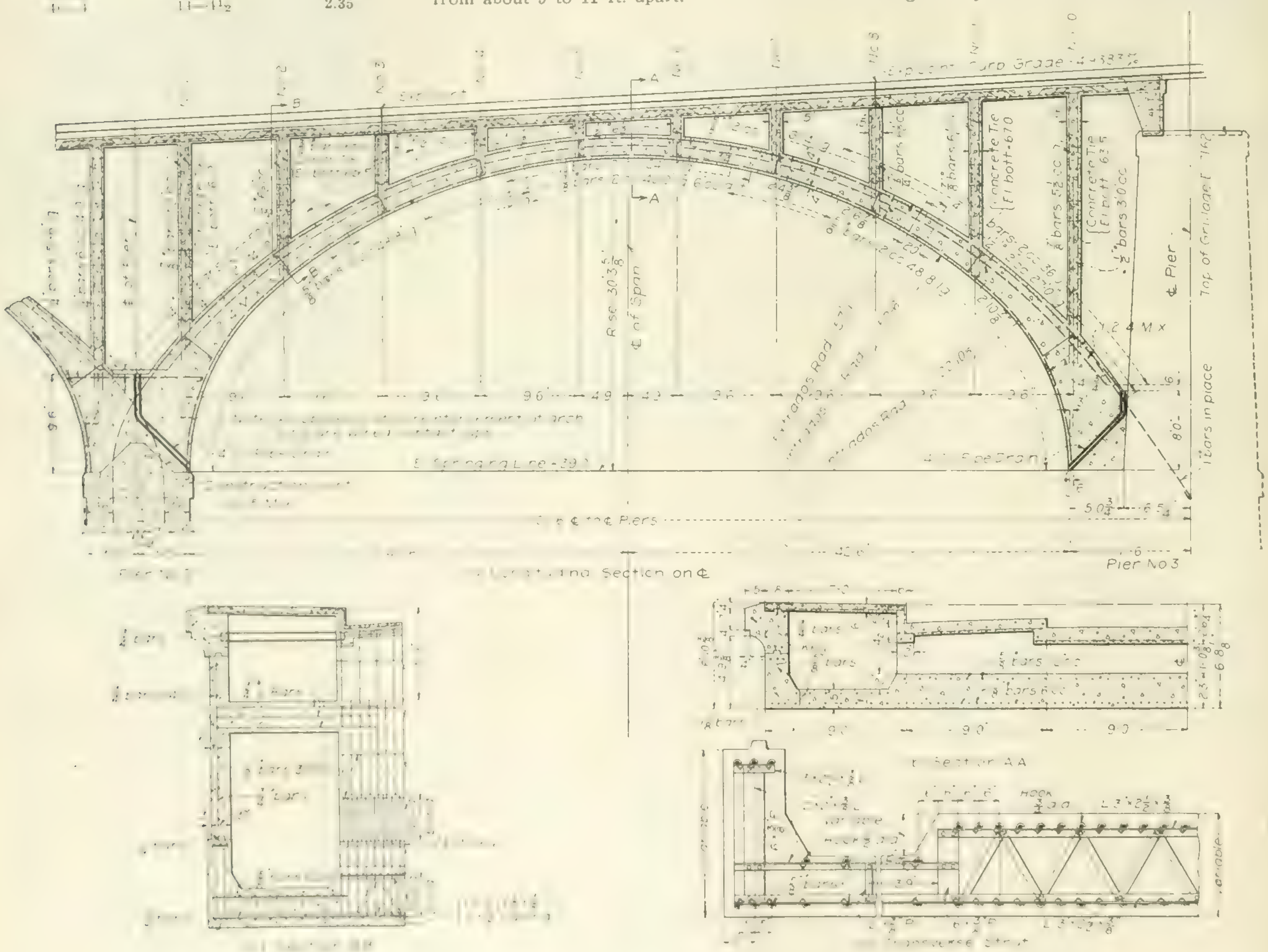


Fig. 4. Longitudinal and Cross Sections of 85-Ft. Reinforced Concrete Arch Spans in Approaches of North Side Point Bridge, Pittsburgh, Pa.

Figure 4 (a) shows a longitudinal section on the center line of an arch having a clear span of 85 ft.; Figure 4 (b) shows a half cross section of the arch at the center of the span; Figure 4 (c) shows a cross section [illegible] and Fig. 4 (d) shows details of the steel reinforcement under the spandrel walls. These detail drawings show the size and arrangement of the reinforcement, the dimensions of the various parts of the arch, and the position of construction joints.

The reinforced concrete floor slabs have transverse expansion joints at about the quarter points of the spans, these joints being centered over the transverse spandrel walls (see

[illegible] OF MATERIALS AND COST.

The approximate quantities of materials contained in the two approaches are given in the following table:

Concrete, except paving base, cu. yds.	16,970
[illegible] and pylons, [illegible]	51
[illegible]	528.7
Structural steel, tons	75.5
[illegible]	13,100
Slag fill, cu. yds	12,800
Hand railing, lin. ft	3,500
Concrete piling, lin. ft	28,800
Concrete sidewalks, sq. ft	14,192
Stone block paving, including base, sq. yds.	2,952
Granite crossings, sq. ft	1,220
Concrete curb, lin. ft	1,496
Stone block street railway paving, sq. yds.	1,314
15-in. terra cotta sewer pipe, lin ft	750
Catch basins	13
Manholes	2

The contract price for this work, based on the above quantities, was $264,063.

PERSONNEL.

The structure was designed by the Bureau of Construction, Department of Public Works of Pittsburgh, of which N. S. Sprague is superintendent; T. J. Wilkerson, division engineer, and Emil Swensson, consulting engineer. Booth & Flinn, Ltd., are the contractors for the construction of the approaches.

Cast-Iron Pipe in England.—Cast-iron soil pipes and pipe fittings for house drainage have largely replaced wrought iron throughout Great Britain. While the subsidiary mains are constructed of vitrified earthenware pipes, the conduits leading thereto from buildings are now generally of cast iron. Under the building laws it is at present necessary that the drain pipes from each building or house shall run separately into the subsidiary street main, and not connect up with such drains from other buildings. For premises erected prior to this requirement, inspection has been made by the local sanitary authorities and the owners of the buildings served with a notice requiring the substitution of the new method of drainage. According to a building trade journal, the prices in England of cast-iron soil pipes are as follows per ton: 3-in. diameter, $29.81 to $30.90; 4 to 6-in, $29.20 to $30.42; 7 to 24-in., $26.15 to $29.20; coated with composition, $1.22 per ton extra; turned and bored joints, $1.22 per ton extra.

Process of Oxy-Acetylene Welding and Equipment of Typical Torches.

Autogenous welding is a general term given to welding when the metal fuses. In the ordinary blacksmith's weld the metal does not fuse; it becomes plastic and is hammered together, but unless hammered no weld would be produced. Autogenous welding is carried on in several different ways, but as here referred to it is by means of acetylene burned with oxygen. The use of oxygen with other combustible gases, consisting of an oil gas, hydrogen and a few other forms, is somewhat common, but these gases are not efficient owing to limitations of temperature. The process using acetylene necessarily makes very much the hottest flame, owing to the large amount of carbon in the compositon of the gas.

The following data on the process and control equipment of typical oxy-acetylene torches were taken from a paper by Henry Cave, in the Journal of the American Society of Mechanical Engineers.

The process of acetylene welding is not generally understood. There is a knack in making a strong weld that can be acquired only by the exercise of considerable judgment and practice. Figure 1 illustrates the procedure followed in making a typical weld. The torch tip here carries a flame of 6,300° F. playing on the metal, and a pool of molten metal is first formed at *A*. The metal is heated approximately as indicated by the shaded portion shown in Fig. 1. Upon that spot is deposited a drop of molten metal from the rod of adding metal *B*, the end being fused into the melted pool *C*. The drop is then fused still further so as to adhere thoroughly to the piece to which it is being welded by playing the flame around that drop until it assumes the shape *D*. With a sharp corner, such as at *C*, no satisfactory weld would be produced, as there would be a spot on each side which could not be welded.

The theory of the process of oxy-acetylene welding is explained in Fig. 2. A "Davis-Bournonville" high-pressure, positive-mixer torch is shown in Fig. 2 (a). The acetylene, C_2H_2, passes in at the side orifices and is ignited. When it is ignited in air a reaction takes place as shown (2½ parts of oxygen uniting with 1 part of acetylene yield carbon dioxide and water), and the reaction produces a temperature about the same as that of ordinary illuminating gas (about 2,500° F.). The oxygen in this case is taken from the air.

When oxygen is supplied, from a cylinder or other source of supply of pure gas, passing in under pressure at the opening indicated at the end of the tip (Fig. 2, b) and striking the acetylene, which is also under pressure, at right angles, a thoroughly homogeneous mixture is produced, and when a mixture of 1 part of oxygen and 1 part of acetylene is provided by the size of orifices, the reaction is as shown in Fig. 2 (b), (1 part of oxygen uniting with 1 part of acetylene yielding hydrogen and carbon monoxide).

The temperature of 6,300° F. is obtained in the bright cone at the end of the tip. The hydrogen cannot burn in that high temperature zone (as it cannot unite with the oxygen), because it is above the dissociation temperature of water. It therefore passes out to the long flame, which is called "the envelope," and burns to H_2O; neither can the carbon monoxide (the product of the first reaction), unite with the additional oxygen to produce carbon dioxide in the high temperature zone, and therefore this reaction must also take place in the envelope. The additional amount of oxygen is in both cases obtained directly from the air, if a satisfactory torch is used.

The carbureting arrangement in the torch, which is generally at the base of the tip, corresponds to the carburetor of an automobile, and if it is not adequate or not of a good type the mixture and reaction will not take place as given above. If too large a proportion of oxygen is supplied this additional oxygen cannot possibly unite with a combustible element in the high temperature zone of this reaction because all the carbon is used up by one part of oxygen. Therefore the surplus must pass through to the envelope, and, as shown at Fig. 2 (c), we have the reaction indicated (2 parts

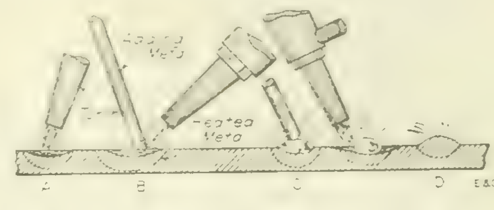


Fig. 1. Diagram Illustrating Process of Making Weld With Oxy-Acetylene Torch.

of oxygen uniting with 1 part of acetylene yielding hydrogen, carbon monoxide and a surplus of free oxygen).

Part of the surplus oxygen going to the envelope unites with either the hydrogen or the carbon monoxide and forms carbon dioxide or water vapor in the secondary reactions, so that a portion only of the oxygen required for these reactions is taken from the air instead of all of it; i. e., some of it is taken from the source of supply, at a cost of about 3½ cts. per cubic foot, instead of being obtained from the air

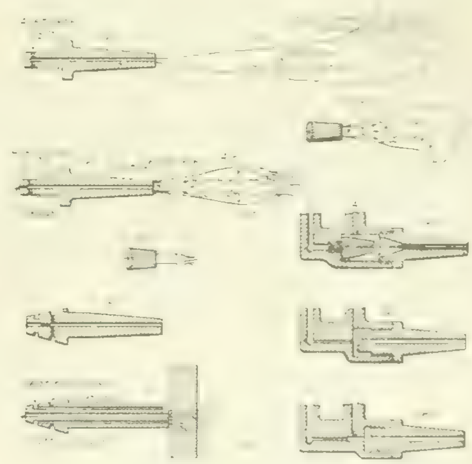


Fig. 2. Diagrams Illustrating Theory of Oxy-Acetylene Process and Showing Several Types of Torches.

without cost, which is obviously an uneconomical condition.

If the means for bringing the oxygen and acetylene together are not such as will produce a vortex, and therefore cause all the molecules of oxygen to come into contact with those of the carbon in the acetylene, a certain proportion of the oxygen will pass out of the tip unconsumed, through the high temperature zone. To allow for this a larger amount of oxygen must be supplied to obtain the normal adjustment of the flame, which is distinguished by the clear outline of the inner cone. It can thus be seen that this clear outline does not necessarily mean that the flame is neutral, as the flame must necessarily be oxydizing if the surplus oxygen is present as above noted. The normal adjustment may therefore indicate a proportion of 1 part oxygen to 1 part of acetylene (1.12 to 1 is the nearest to this which has been obtained) or it may indicate as high as 2 parts of oxygen

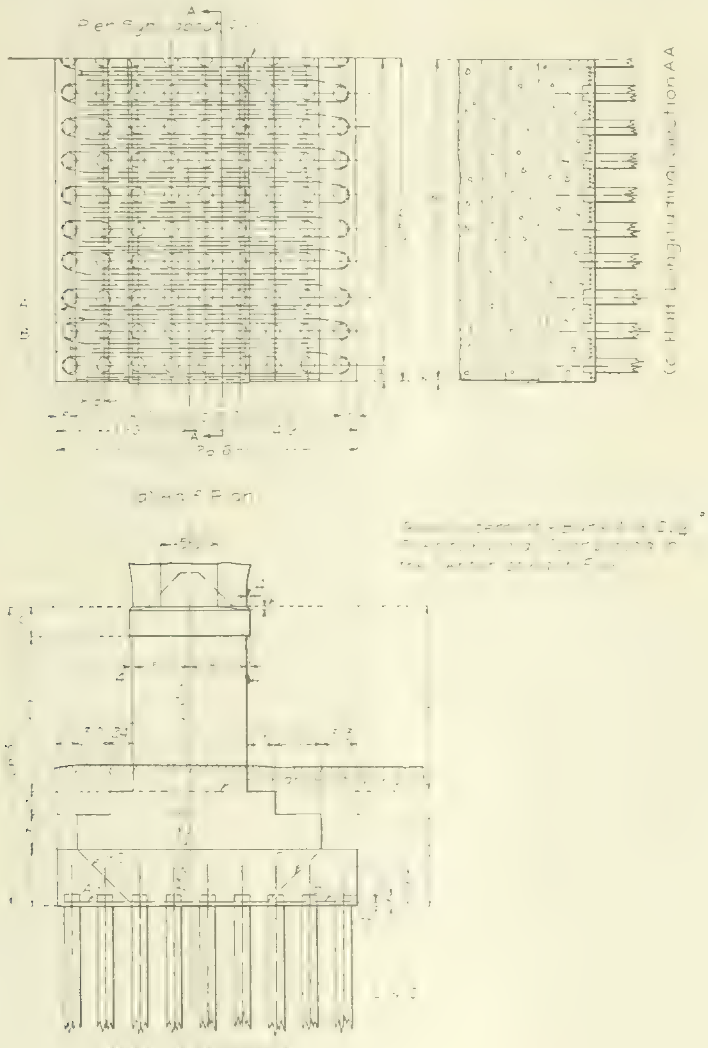


Fig. 5. Half Plan, End Elevation and Half Longitudinal Section of Pier for 85-Ft. Arch Spans of North Side Point Bridge, Pittsburgh, Pa.

to 1 part of acetylene (1.8 to 1 being very common).

If too large a proportion of oxygen is supplied through the tip for the normal setting as given above, the flame is shortened and the color is changed. The flame condition shown in Fig. 2 (d) indicates too large a proportion of acetylene or a shortage of oxygen from the normal setting, the acetylene showing an extra or extended point on the bright welding flame. These two extreme conditions are used by the operator to adjust the flame, but he can only adjust it within the limits of the mixing device. The tip shown in Figs. 2 (a) and (b) is illustrated because it is a simple representation of the "high-pressure positive-mixture" carbureting device, but it has now been changed to the form shown in Fig. 2 (e), the principle of which, however, is the same. The tip is seated in a cone, and the oxygen passes through the central orifice and the acetylene through the radial orifices, striking together so that there is no question of a thoroughly molecular contact of the gas being produced before the gases pass out and are ignited.

Another type of torch, which is shown diagrammatically in Fig. 2 (f), is that in which the two gases are brought together by what is termed the injector principle, oxygen passing in and shooting straight through a chamber which contains acetylene either under several pounds pressure or under a pressure of only a few ounces. The conditions are practically the same in both cases. The oxygen shoots through and the capillary attraction or "skin friction" of the oxygen draws in the acetylene and produces a condition in which there is pure oxygen in the center and pure acetylene around the outside, with the mixture in a zone between the two. The injector is made just as smooth as possible inside, so as not to produce any vortex. This torch is made in various styles. Sometimes the injector cone is a narrow passage and sometimes it has an area of considerable size. Frequently there are jogs or enlargements in the passage to facilitate a mixture of the gases. All the gas, however, passes through the center.

Another type of tip is shown in Fig. 2 (g). This type differs only in detail from the one previously described, the principle being that of an injector (although without a cone), there being no control of the proportion of the two gases which pass in by orifices—such as are shown in Figs. 2 (a), (b) and (e)—the size of the orifices in these tips being accurately determined for the various sizes of tips by the amount of gas required, thus insuring the correct proportions at all times.

Another type is shown in Fig. 2 (h), in which there are passages which regulate the amount of gas passing through, but these passages are in the head of the torch instead of in the tip. When a larger or smaller tip is inserted, the mixing passage remains the same; therefore only one of the set of tips for that torch has orifices which are correctly proportioned.

Figure 2 (i) represents the cutting operation. This torch tip has a control passage through which the jet of oxygen passes. It also has a side passage which carries an ordinary heating flame which heats the metal in advance. There is also a following flame, the same as that used for welding, which merely keeps the metal in a fluid condition so that it will run out of the cut.

Labor on New York Subways.—Some 12,-[illegible] the construction of subway systems in New York City, according to the last report of progress of Alfred Craven, chief engineer of the Public Service Commission. It appears that more than half the work is now under contract, but that when all contracts have been let, a matter perhaps of only a few months, the number of men will be almost doubled. The city has $126,-414,000 in contracts, in progress or completed.

Bridge Cost Record Used by the Illinois Highway Commission.

(Staff Article.)

The Illinois Highway Commission has prepared a "Bridge Cost Record" for use in keeping data in the field and in determining the final cost of its bridge work which greatly facilitates and systematizes this work. The record is in book form and contains 87 pages. It is 4½x7⅜ ins. in size and has a flexible leather binding. Each inspector in charge of the construction of a bridge is given a copy of the record. In addition to keeping account of the quantities of materials and the costs, as outlined by the forms contained in the cost record, the inspector is required to mail to the office daily a report card showing the progress of the work. This report card is post card size.

The upper half of the first page of the record book contains the form for recording general data on the particular bridge under construction, the following data being recorded:

File No.................

Name of bridge ....

County ....

Township ....

Span .... Roadway ....

Type ....

Inspector ....

Contractor ....

Time limit...................................

Date			
Removing old bridge			
Dry excavation			
Wet excavation			
Driving sheet piles			
Pulling sheet piles			
Pumping while excavating			
Pumping while concreting			
Driving bearing piles			
Building forms			
Bending and placing steel			
Mixing and placing concrete			
Building falsework			
Removing forms			
Removing falsework			
Total			
Schedule of wages			

Fig. 1. Left-Hand Page of Form for Labor Record.

Pages 2, 3 and 4 are blank, and pages 5, 6, 7 and 8 contain the following set of "Instructions":

As soon as work starts on the bridge assigned to you, fill in the blank spaces on the cover of this book with ink.

ITEM	DATE					191	Total for Week	Total Deliv'd
Concrete placed cu. yds.								
Cement used bbls.								
Cement del bbls.								
Stone del. cu. yds.								
Sand del. cu. yds.								
Gravel del. cu. yds.								
Steel del. lbs.								
Form lumber del. M ft. B. M								
Piling del. No.								
Sheet piling del. M ft. B. M.								
Falsework timber del. M ft. B. M.								

Schedule for hauling

Fig. 2. Form for Material Record.

Diagram.—On the first page following these instructions, draw a diagram of the bridge showing the abutments, wings, footings, piers, girders and rails. Show the north point and direction of stream flow. Use the following abbreviations to denote the different parts of the structure:

N, S, E and W = north, south, east and west.
A = abutment.
W = wing.
f = footing.
P = pier; P1 = pier No. 1, etc.
S = span; S1 = span No. 1, etc.
F = floor; F1 = floor span No. 1, etc.
G = girder.
R = rail.

By combining these abbreviations, each of the various parts of the bridge may be indicated as NAf = north abutment footing; NG1 = north girder span No. 1, etc.

The above system of abbreviations may be extended or diminished to suit the bridge in question and is to be used in the labor and material records for the purpose of designating the part of the structure on which the work is done.

Labor Record.—Under "Schedule of wages" at the bottom of the first page of the labor record, make a schedule showing the rate of pay per hour received by the various men employed on the work such as a = foreman at 60 cents per hour; b = laborers at 25 cents per hour; c = carpenters at 40 cents per hour; etc. If the contractor pays the board of the men in addition to the daily wage, the cost of board to the men should be included in the rate of pay per hour. The daily labor record should show the part or parts of the work under way, the number of hours at each rate of pay and the total labor cost. The foreman's time should be distributed each day among the various items in proportion to the labor. For instance, if the schedule of pay is as given above and five laborers work ten hours each in excavating pier No. 1; three laborers work ten hours each driving sheet piles for pier No. 2, and two carpenters work ten hours each building forms for the south abutment, the day's record would be as follows: Opposite the item of "Dry" or "Wet excavation," as the case may be, would appear, P1, 50 b, 5 a = $15.50. Opposite "Driving sheet piling" would appear P2, 30 b, 3 a = $9.30; and opposite "Building forms" would appear SA, 20 c, 2 a = $9.20.

Material	Cost: At R. R. Station	Cost: At Bridge	Name of Company Furnishing Same
Cement	....	....	....
Stone......	....	....	....
Sand........	....	....	....
Gravel........	....	....	....
Steel........	....	....	....
Piling.......	....	....	....
Form lumber..	....	....	....
Sheet piling...	....	....	....
....	....	....	....
....	....	....	....
....	....	....	....

Remarks: ....
....
....
....
....

Fig. 3. Form for Material Data.

Materials.—Under "Schedule for hauling" at the bottom of the first page of the "Material Record," make a schedule showing the rate per day or per unit of material for team hauling, using letters x, y, z, etc., to avoid confusion with the labor schedule. For instance, if the hauling is done by the day at $4.50 for a ten-hour day, at the bottom of the page would appear x = 45 cents per hour per team. Then if on a given day three teams haul gravel for ten hours and deliver three yards each, one team hauls cement ten hours and delivers thirty barrels; then in the day's record for hauling would appear opposite "Cement delivered" 30, 10 x = $4.50, and opposite "Gravel delivered" 9, 30 x = $13.50.

If the hauling is done at a certain rate per unit of material, as, for instance, 50 cents per cubic yard for gravel, at the bottom of the page should appear "Gravel hauled at 50 cents per cubic yard," and it is not necessary to record for the day anything more than the number of units hauled and the total cost for the day.

If gravel, stone or sand is purchased or hauled by the ton, the equivalent number of cubic yards may be computed on the basis of one cubic yard of gravel weighing 3,000 lbs.; one cubic yard of sand weighing 2,750 lbs.; one cubic yard of broken stone weighing 2,500 lbs.

On a succeeding sheet under "Schedule" give the cost of materials at point where team haul begins.

Determine each day the amount of concrete placed by measurement in the forms and in the proper space record this amount and by abbreviations where used.

Record the amount of cement used each day and check against concrete placed, and amount required by the specifications for the material being used.

Falsework lumber includes piles or posts, bracing, wedges, beams and stringers. The lumber for the floor belongs under "Form lumber." If piling is used for the falsework, add a sufficient number of board feet to the rest of the falsework lumber to make up for the cost of the piling.

Permanent Piles.—The "Piling Record" refers to piles used as a part of the permanent work and should show the original length, cut off and net length of each pile, and should also show the total for each abutment and for each pier. The total lineal feet of permanent piling left in the ground for the entire bridge should be given at the end of the record.

Tests.—Record in a systematic manner all gravel and sand tests, tests on foundation and any other tests which you may make.

Instructions.—Record all suggestions given to the foreman or received from your division engineer.

Cement ....bbls. @ $........per bbl. = $........
Hauling cement ....miles @ $........per bbl. = $........
Crushed stone ....cu. yds. @ $........per cu. yd. = $........
Hauling crushed stone....................miles @ $........per cu. yd. = $........
Sand ....cu. yds. @ $........per cu. yd. = $........
Washing sand ....cu. yds. @ $........per cu. yd. = $........
Hauling sand ....miles @ $........per cu. yd. = $........
Gravel ....cu. yds. @ $........per cu. yd. = $........
Washing gravel....................cu. yds. @ $........per cu. yd. = $........
Screening gravel ....cu. yds. @ $........per cu. yd. = $........
Hauling gravel ....miles @ $........per cu. yd. = $........
Form lumber ....M. ft. B. M. @ $........per M. = $........
Falsework lumber ....M. ft. B. M. @ $........per M. = $........
Wire, nails, etc....................lbs. @ $........per cwt. = $........
Hauling form lumber....................miles @ $........per M. = $........
Hauling falsework lumber....................miles @ $........per M. = $........
Reinforcing steel ....lbs. @ $........per cwt. = $........
Hauling steel ....miles @ $........per cwt. = $........
Rockers ....lbs. @ $........per cwt. = $........
Asphalt ....gals. @ $........per gal. = $........
Hauling rockers and asphalt....................miles @ $........per day = $........
........round piles at bridge site........ft. long @ $........per pile = $........
Cost of driving piling....................lin. ft. @ $........per linear foot = $........
Sheet piling....................M. ft. B. M. @ $........per M. = $........
Hauling sheet piling....................miles @ $........per M. = $........
Cost of driving.................... ......M. ft. B. M. of sheet piling = $........
Cost of removing.................... ......M. ft. B. M. of sheet piling = $........
Labor on......cu. yds. dry earth exc......hrs. @ $........per cu. yd. = $........
Labor on......cu. yds. wet earth exc......hrs. @ $........per cu. yd. = $........
Cost of pumping during exc....................hrs. @ $........per hr. = $........
Cost of pumping during conc....................hrs. @ $........per hr. = $........
Labor building forms....................hrs. @ $........per hr. = $........
Labor building falsework....................hrs. @ $........per hr. = $........
Labor bending and placing steel....................lbs. @ $........per cwt. = $........
Labor mixing and placing concrete....................hrs. @ $........per hr. = $........
Labor removing forms....................hrs. @ $........per hr. = $........
Labor removing falsework....................hrs. @ $........per hr. = $........
Traveling expenses .... = $........
Extras .... = $........
Extras .... = $........
Incidentals .... = $........
....
....
Total.
Salvage on lumber and falsework.................... = $........
Total net cost to contractor.................... = $........
Total amount due contractor.................... = $........
Profit or loss.................... = $........

Fig. 4. Form for Detail Cost of Bridge Work—Actual Cost.

Material Data.—The material data sheet is inserted to show in a concise form all data on materials. The cost should be for units of material. The column, "Name of Company Furnishing Same," should show the brand of cement and the name and address of the company furnishing stone, sand or gravel, and steel. For the other items, it will be sufficient to give the hauling station. Under "Remarks" give the hauling station or stations of the cement, stone, etc.

If the gravel is used from a local pit, make it a point to visit the pit at some convenient time and give a description of the same on the preceding blank page giving location, name of owner, a rough estimate of the available gravel in pit, etc.

Detail Cost Summary.—As soon as the work is finished, with the exception of removing the falsework, fill out the detail cost sheets at the

Summation of cost per cu. yd. for placing......cu. yds. of concrete and.........pounds of steel.

This does not include cost of round piling, expansion rockers, or removing old bridge.

Cost per cu. yd. of concrete.

Cement ....bbls. @ $.................per bbl $...........

Stone ....cu. yds. @ $.................per cu. yd. = $...........

Sand ....cu. yds. @ $.................per cu. yd. = $...........

Gravel ....cu. yds. @ $.................per cu. yd. = $...........

Labor on forms per cu. yd. of concrete .... = $...........

Form materials per cu. yd. of concrete.................................. = $...........

Labor on falsework per cu. yd. of concrete.................................. = $...........

Falsework materials per cu. yd. of concrete.................................. = $...........

Cost of steel in place per cu. yd. of concrete.................................. = $...........

Cost of mixing and placing concrete.................................. = $...........

Cost of excavation per cu. yd. of concrete.................................. = $...........

Total = $...........

Cost of exc. per cu. yd. of substr. concrete.................................. = $...........

Cost of falsework per cu. yd. of superstr. conc.................................. = $...........

Cost of removing old bridge.................................. = $...........

(Give brief description of old bridge.)

Fig. 5. Form for Cost per Cubic Yard.

end of the book with pencil. Each item of cost should include all items that may properly be included thereunder, as, for instance, cost of pumping during excavation should include cost of rigging of pump, fuel, etc., and cost of driving piling should include cost of rigging up driver, removing same, etc.

The item, "Labor Removing Falsework," will have to be estimated in most cases, as you will probably not be on the job when this work is done.

The item, "Incidentals," should include only such items as cannot properly be applied to the detail work items. Incidentals, extras, traveling expenses, etc., are to be distributed among the cost per cubic yard items as may seem most appropriate, giving in the detail items an indication of the distribution.

The salvage on lumber and falsework is an item subject to considerable variation. It depends mostly upon the dimensions and quality of the lumber and the complication of the form work. If the contractor has work on which the lumber will be used several times, this should be taken into consideration in figuring salvage. For instance, if the form lumber can be used on three different jobs, the salvage would be 67 per cent less the value of the lumber wasted. If, however, the contractor buys new lumber and sells it when the work is finished, the salvage would be the selling price of the lumber which might not be more than 15 per cent.

In summarizing the cost per cubic yard of concrete, each item in the summary should include all of the detail items that properly belong in the summary item. For instance, forms per cubic yard of concrete should include lumber, nails, wire, etc., and excavation should include cost of sheet piling, excavation, pumping, etc.

Concrete shown on plans........................Superstr. ....cu. yds. Substr.........cu. yds.

Reinforcing steel shown on plans................Superstr. .... lbs. Substr............. lbs.

Concrete actually used..........................Superstr. ....cu. yds. Substr.........cu. yds.

Reinforcing steel actually used..................Superstr. .... lbs. Substr............. lbs.

Extra concrete authorized in superstr............................19.....................cu. yds.

Extra concrete authorized in substr..............................19.....................cu. yds

Reduction in concrete authorized in superstr.....................19.....................cu. yds.

Reduction in concrete authorized in substr.......................19.....................cu. yds.

Piling authorized ....19.....................lin. ft.

Fig. 6. Form for Summary of Quantities.

Contract price substructure...Dollars ($.)

....cu. yds. extra concrete @ ($....) per cu. yd............................Dollars ($.)

Fig. 7. Form for Payments Recommended.

Disposal of Records.—When your cost record is complete, mail the book together with the plans, to your division engineer. He will check the same and return the book to you for corrections if any are necessary, and you should then letter with ink the four detailed cost sheets. The other data may be in pencil, but [illegible] After inking on the cost sheets, send the book to the office. Have this book with you on the job at all times so that your division engineer may examine it when he comes to visit the work.

If in doubt about anything, ask your division engineer.

Pages 9 and 10 are blank, and on page 11 there is the following heading: "Plan of Bridge, Showing Abbreviations to Be Used on Labor Sheets," with space below for recording these data.

Pages 12 and 13, to 34 and 35, inclusive, which face each other, contain duplicate forms for making out the "Labor Record." The left-hand page contains the data shown in Fig. 1. The right-hand page is ruled in a similar manner, the last column of which has the heading "Total."

Pages 36 to 47, inclusive, contain duplicate forms for recording the "Material Record." Each form fills one page and is made up as shown in Fig. 2.

..................Br. 1914

Tp..............County......................

INSPECTOR'S DAILY REPORT.

Kind of Work	No. of Hrs. Work	Percent Completed	Remarks
Excavating			
Abutment			
Abutment			
Pier.			
Pier.			
Erecting Forms			
Abutment.			
Abutment.			
Pier...			
Pier.			
Floor			
Girder			
Placing Rods			
Concreting			
Abutment.			
Abutment.			
Pier.			
Pier			
Floor			
Girder.			
Building False Wk.			
Removing Same			
Pumping			
Hauling Material			
Team and Man			
Removing Forms			

Material on Ground	Sand	Gvl.	Stone	St'l Rods	Bridge Material

P. O. . Inspector

Fig. 8. Form for Inspector's Daily Report.

The forms for keeping the "Piling Record" data are given on pages 50, 51, 52 and 53. Each page is divided into four columns by vertical lines, the heading for these columns being "Pile No."; "Length"; "Cut Off," and "Net Length." Each page contains space for 25 pile records.

From page 54 to page 74, inclusive, there are given blank spaces for recording such items as: Bearing Tests on Foundation (3 pages); Gravel and Sand Tests (3 pages); Suggestions Given to Foreman (4 pages, ruled horizontally); Instruction from Division Engineer (4 pages, ruled horizontally); and 7 blank pages.

On page 75 there is given a "Material Data" form which is made up as shown in Fig. 3.

On pages 76, 77, 78 and 79 there is given a form for determining the "Detail Cost of Bridge Work—Actual Cost." In the cost record book the item and the quantity are on the left-hand page and the unit and total cost on the right-hand page. This form is made up as shown in Fig. 4.

Pages 80 and 81 contain the "Cost Per Cubic Yard" form. This form occupies both the left-hand and the right-hand pages, a small space being left at the bottom of the pages for a brief description of the old bridge. This form is shown in Fig. 5.

On the upper half of pages 82 and 83, extending across both pages, there is given the form for the "Summary of Quantities." This form is shown in Fig. 6.

The lower half of pages 82 and 83 contain the form for "Payments Recommended." This form extends across both pages and is made up as shown in Fig. 7.

Pages 84, 85, 86 and 87 are blank pages.

DAILY REPORT FORM.

The form used by the inspectors in making out their daily report is shown in Fig. 8. The form is post card size (3¼x5½ ins.), and contains on the opposite side the commission address and a space for a 1-ct. stamp.

SEWERAGE

Method and Cost of Making House Connections to the New Orleans Sewerage System.

The work of the sewer house connection sub-department of the Sewerage and Water Board of New Orleans, consists of the building by force account of connections between the public sewers and the property lines; the supervision of the building, by plumbers, of a few such connections less than 7 ft. in length and furnishing such information as may be required by the other departments of the Sewerage and Water Board, by plumbers and by the public. The procedure and cost of conducting this work is here described from information taken from the recent report of Messrs. Rudolph Hering, George W. Fuller and Harrison P. Eddy on the work and organization of forces of the Sewerage and Water Board.

Organization.—The house connection sub-department of sewers, water mains and connections, is under the charge of an assistant engineer, who is responsible to the engineer in charge of the above mentioned department. His organization, as of June 18, 1914, consisted of 1 chief clerk, 3 assistant clerks, 1 stenographer (on part time), 8 foremen, 7 pipe layers, 42 laborers and 7 teams. Each of the seven foremen is provided with three gangs each consisting of one pipe layer and two laborers. Each foreman also has one team, serving the three gangs, keeping them supplied with tools and material. Each foreman is entitled to an annual vacation of two weeks. On June 18 one of the eight foremen employed was acting as substitute for the foreman who was then on vacation. The total force employed in this department on June 18, therefore, consisted of 83 men, including drivers, on full time, and one-quarter of the time of one stenographer.

Amount of Work Done.—This branch of the work has increased from year to year as the system has been extended, 7,367 house connections having been built in 1913. The monthly number of connections made from June, 1913, to May, 1914, both inclusive, was as follows:

Month.	Number.
June, 1913	[illegible]
July	[illegible]
August	[illegible]
September	681
October	783
November	415*
December	615
January, 1914	668
February	653
March	885
April	1,079
May	1,016
Total	8,835

*A portion of the force was laid off in November because of doubt as to legality of doing the work by force account.

From the above data it appears that the number of house connections made during the period designated averaged 30 per day, and that during April, 1914, the average was slightly more than 43 per day. It is important carefully to consider these figures, in order to reach a clear understanding of the duties of and the necessity for the employes of this sub-department.

Applications for Connections and Office Work Connected Therewith.—The first work involved in making a house connection is the result of a request for information as to the exact location of the Y-branch in the sewer, with which the connection is to be made, by such plumbers as may be submitting bids to the property owner for doing his plumbing. Such requests are usually received from several plumbers, often as many as six or eight, for a single piece of work.

The next step is the application for the connection. This application is made out by the plumber on a printed form furnished by the Board for this purpose. This application is taken to the record office and duly entered. If application has previously been made by some other plumber, or if it appears that the property, as described in the application, has already been connected with the sewer, the same is returned for correction. After receiving the approval of this department, the application is presented to the clerk of the sewer house connection sub-department, who looks up the location of the Y-branch with which connection is to be made, and checks off this connection on his records as having been used. This clerk also furnishes information as to the locations of Y-branches to plumbers and others requiring such data, and makes such additions and corrections to his lists of Y's and T's as may be required by changes or additions made.

The application, having received a serial number, is next turned over to the clerk, who enters the same in the Work Record Book, and makes out instructions upon proper forms for the use and guidance of the foreman who is to make the connection. The Y slip, from which the foreman can ascertain the exact location of the Y with which he is to connect, is then attached to the work slip or foreman's report. This report blank is so printed as to provide blank spaces for all information required as to the location and work done, and the supplies used. This form is attached to the Y slip, is turned over to the foreman, as his order for the work to be done.

Upon the completion of the work, the foreman returns the work slip with the data entered thereon, including an accurate account of the quantities of materials used. The foreman also makes out a daily report, stating the number of uncompleted connections left over from the day before, the number laid and completed on the day of the report, and the trenches left open that night. This report also includes a memorandum of the time of the pipe layer, teamsters, laborers and team working that day.

Upon the completion of a connection the foreman making it turns in a report, stating the condition in which he found the street and sidewalk when he began work, the condition in which he left the sidewalk upon the completion of his work, and how he left any paving or other materials piled on the street or sidewalk. The data contained upon this report card are turned over to the paving sub-department, and forms the basis for its work in maintaining safe conditions, restoring the pavement and cleaning up the street.

Upon the completion of a house connection a postal notice to that effect is sent to the plumber making application for the connection, with instructions to connect the plumbing therewith within fifteen days.

Cost Accounts.—The payrolls are made up from the foremen's daily reports, and bear the signatures of the assistant engineer in charge of the sub-department, the foreman in charge of the men whose names are upon the payroll sheet, and the initials of the clerk carrying out the totals.

The foreman's reports, specifying the quantities of materials used in making each connection, form the basis for the charging of materials to the work. All such materials are entered daily in a book account covering the individual work of each foreman. On the other side of the ledger are entered the materials furnished the foreman from the yard. At the end of the month the quantity of materials furnished to each foreman is balanced with the quantity reported by him as used or on hand at the end of the month, showing any deficit or excess. To this account is added the daily labor and team expense, the total showing the actual expenditures made for the work done by each foreman. The account also shows the number of connections which have been made and the number of feet of pipe which have been laid during the month, from which the unit costs per connection and per foot of pipe laid are computed, thus furnishing an accurate record in units, which are easily compared and carried in mind. By aid of these accounts, comparison is made from month to month of the cost of the work done by the several foremen, who are advised of the cost of their work. In this way a friendly rivalry may be established among the foremen in charge of this work.

The summary of the accounts of one foreman for the month of April, 1914, was as follows:

Foreman's salary	$ 100.00
Materials	314.35
Labor	596.50
Supervision	76.50
Total	$1,087.15
Connections made	145
Average cost per connection	$7.50
Supervision, per cent	7
Pipe laid, ft	3,704
Cost per foot, cts	29

To show the comparative cost of work done by the several foremen, the data given in Table I. have been compiled, showing the costs of work done in December, 1913, and April, 1914, the months within a year when the minimum and maximum numbers of house connections, respectively, were made. These costs do not include the cost of repaving, allowances for yard expenses, or overhead expenses other than those of this sub-department, and consequently are not comparable with the costs given in the annual reports, which include all expenses.

TABLE I. UNIT COST OF SEWER HOUSE CONNECTIONS AT NEW ORLEANS IN TYPICAL MONTHS.

No. of connections.	Cost per connection.	Pipe laid. feet.	Cost per foot.	Supervision. pct.	Foreman.
DECEMBER, 1913.					
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	A
141	8.19	3,618	.32	7.1	B
124	[illegible]	[illegible]	[illegible]	[illegible]	C
141	[illegible]	[illegible]	[illegible]	[illegible]	D
Worked only part time					E
APRIL, 1914.					
144	$7.48	3,382	$0.32	7.1	A
146	7.55	3,698	.30	6.9	B
119	8.74	3,190	.33	7.3	C
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	D
141	7.70	3,590	.30	7.1	E
146	7.54	3,722	.30	6.9	F
141	7.69	3,450	.31	7.1	G

From the data given in Table I. it will be seen that there is some variation in the cost per connection and cost per foot of pipe laid as between foremen and as between different months of the year. The cost of the work done in December was found to be somewhat higher than the cost of work done in April, partly because of the difference of the amount of work done, 615 and 1,079 connections, respectively, and partly because of the broken time due to the holiday season. It is to be expected that there will be a fluctuation of costs from time to time because of differences in the difficulty of the work done. Some foremen are working in districts where there are large quantities of pavements to be removed, which necessarily increases the cost of their work, although the cost of replacement of pavement is not included in the figures given.

Traffic conditions and the necessity of crossing under car tracks and the presence of pipes and other structures in the streets all affect the cost of work done. These items and many others, however, are made a matter of record in the log of the work done by the sewer house connection sub-department.

SEWER HOUSE CONNECTIONS MADE AS REQUIRED.

It has been suggested that connections could be made at less cost if all that are ever likely to be required in a given street are made at one time, instead of putting in connections as requested by property owners.

A number of house connections were put in by this method ahead of street paving by contractors upon some of the sewers. Many of these will never be used and substitute connections have had to be built on account of requirements of property owners and other conditions which could not be anticipated at the time the original connections were built. The data on this subject are as follows:

Connections brought to property lines ahead of paving by contractors	1,510
Number of above used	673
Number of above not yet used	837
Substitute connections built to January 14, 1914, to permanently replace original connections which may never be used	[illegible]

These unused connections constitute an unnecessary cost of original construction and a menace to the sewer system.

It is possible that the cost per connection of work done in such a way would be slightly less than the cost per connection built upon request. On the other hand, a large number of connections made in this way have proved unnecessary, and some have had to be changed, or new connections substituted therefor. There can be no doubt that the cost of unused connections and connections which have to be changed would much more than offset any slight apparent saving in cost due to putting in connections for every lot in a block at one time.

The Probable Future of Various Sewage Treatment Methods.

In his report to the Metropolitan Sewerage Commission on the collection and disposal of the sewage of the city of New York Mr. George W. Fuller, at the request of the Commission, gave his opinion on the probable future changes in sewage disposal procedure. The opinion is here quoted from the final report of the Commission.

OXIDATION METHODS.

These procedures are aimed essentially at the prevention of putrefaction of sewage matters, and hence deal particularly with soluble and colloidal constituents of sewage.

Dilution.—This is the prevailing method all over the world for securing the oxidation of soluble or non-settling sewage matters wherever there is ample water to bring about satisfactory results. Without attempting to state precisely the digestive capacity of New York harbor, it is sufficient to say that for all time to come it will oxidize under suitable conditions the clarified sewage of many millions of people. Departures from this method will be required solely in areas where the sewage cannot readily be delivered to bodies of water of sufficient volume.

The dilution method has usually been applied in this country under improper conditions. Clean watercourses can be maintained with the dilution methods and past shortcomings should not militate against the advantages and economies of utilizing the digestive capacity of the harbor waters.

Filtration.—Where sewage cannot be applied to diluting bodies of water on account of the distance from same and the expense of delivering the sewage, filters will undoubtedly continue to serve a useful purpose. I believe the future will show no radical departures in the efficiency or cost of either sprinkling or contact filters. The tendency will be towards improvements in the design of certain details. I am clearly of the opinion that filtration has no future so far as relates to the main volume of New York sewage.

For certain areas, particularly those now comparatively sparsely populated, I think that a number of filter plants installed and operated for a period of 10 to 20 years may in the end prove cheaper, even if then abandoned, than would be the case if at the outset the general plan should embody the execution of collection works for a central plant with several subdivisions.

I look for improvements in the preliminary treatment of sewage to lessen materially the odors sometimes attending sprinkling filters. For certain thickly populated districts I believe contact beds are entitled to careful consideration.

The lessening of odors requires primarily the prevention of putrefactive conditions becoming established in the unfiltered sewage by aeration, oxidizing chemicals and probably to some extent the application of sewage so as to minimize the opportunity for wind action to transport sewage spray.

Aeration.—I look upon aeration as a promising means of guarding against putrefaction. If conditions arise where it is necessary to deal with sewage which has to some extent undergone putrefaction I consider it possible that organic matter to a limited extent may be reduced through oxidation by air. Except for limited quantities of unstable organic substances aeration cannot be counted upon as an oxidizing agent in practice. It provides molecular oxygen, whereas the bulk of the organic matter of sewage is oxidized only by the atomic oxygen of certain powerful oxidizing agents, or the slow oxidation effected through bacterial action.

The function of aeration is essentially one of artificially prolonging the period of decomposition of sewage on an inoffensive aerobic basis. It may be that conditions will arise where the occasional application of air by artificial means will permit the dilution method to be employed, whereas without aeration or some similar treatment the dilution method would present objectionable and offensive shortcomings.

On the score of cost the aeration method for general use does not look promising; and this is particularly true of the New York conditions where the sewage oscillates back and forth for a period of time that makes a demand on the atmospheric oxygen greater than is the case with many flowing inland streams.

Oxidizing Chemicals.—The cost of applying such chemicals as liquid chlorine and hypochlorite of lime or soda is prohibitive on account of the relatively small amount of organic matter that is oxidized. The true function of such agents is the killing of bacteria, and within certain limits they are useful as a preservative for sewage to prevent decomposition rather than to effect complete ultimate oxidation of the nonsettling organic matters.

Electrolytic Treatment.—A good deal is claimed from time to time for the economical advantages derived from the application of electricity in various types of electrolytic cells in oxidizing the organic matters of sewage. In particular is this claimed where the current may be applied to sea water. While improvements in this direction may be possible, I do not believe that they are promising enough to warrant consideration of electrolytic treatment as a substitute for the dilution method.

The electrolytic production of chemicals to precipitate certain colloidal and other nonsettling organic matters may prove desirable in the future and worthy of adoption under some conditions. This method would be used, however, in conjunction with settling tanks, and could be employed supplementary to such tanks without requiring material changes in the design of tanks built for plain sedimentation alone.

CLARIFICATION.

The purpose of clarification devices is to prevent unsightly sewage solids appearing in the diluting water, or the formation of sludge banks in watercourses.

Fine Screens.—Fine screens afford the cheapest way of removing visible objects of sewage origin from the waters receiving sewage where such screening treatment alone is sufficient for obtaining satisfactory results. Under conditions where the limit is at times reached in the amount of clarified sewage which a watercourse will oxidize satisfactorily, settling tanks as a general rule are cheaper to install than screens, because for a given cost they will remove a greater quantity of organic matter. Where screens will suffice for a term of years—say ten or more—it is quite possible that it will prove economical to install screens first, to be followed later by settling basins as occasion demands.

American experience with fine screens is quite limited and not altogether satisfactory, owing to the devices requiring much attention and repair. This has meant not only expense, but also serious interruptions in continuity of service.

Screens are employed to much better advantage in Europe, particularly in Germany. At Dresden, Frankfort and Hamburg experiences demonstrate on a large scale that it is feasible to operate moving plate screens with an opening as small as 0.04 in. At present the so-called Reinsch type of screen, as installed at Dresden, seems to be most popular. It is of the disc type with slots about 0.085 in. wide and 1.25 ins. long.

As fine screens come into service in America I look for marked improvements in continuity of service and freedom from expensive repair costs. I do not believe their efficiency will be any greater than that indicated by European evidence unless it should result from applying to screens sewages which are fresher and less comminuted than has been the case at Dresden, Frankfort and Hamburg.

Settling Tanks.—Sedimentation for a two-hour period at the average rate of sewage flow will give as good results as it is prudent to obtain from sedimentation. For local conditions I favor single-story tanks with hopper bottoms, as it is undesirable to septicize the sludge at its point of origin.

The construction of settling tanks along the New York water front presents some difficulties on account of the high cost of ground, complications from salt water backing into the sewers at high tide, and the necessity of caring for storm flows. I believe these difficulties can be overcome by careful engineering study.

Fine Screens vs. Settling Tanks.—My views have been already indicated in the foregoing paragraphs. But as the question has much significance in New York I will state that, in my opinion, screens are preferable to settling tanks only where it is desirable or necessary to remove only relatively large sewage matters in supension. Where settling solids would form deposits in the watercourses if screening alone were adopted, to install settling tanks will prove wiser than to install fine screens. Available data in this country are now too meager to allow fine lines to be drawn in this comparison. For problems of this magnitude special study should be given before construction, not only of the relative merits of the two systems as a whole, but also of the local conditions at each main sewer outlet or groups of outlets which can be conveniently united. Both screening plants and settling plants can be operated for treating fresh sewage without creating any nuisance.

Chemical Precipitation.—The use of coagulating chemicals has been known for 50 years as a helpful adjunct in removing particles of suspended matter which are so fine that they will not subside in ordinary settling tanks. The expense of the added chemicals and the large increase in the volume of the resulting sludge is such that as a general proposition chemical precipitation is not worth while.

There may be exceptional conditions where plain sedimentation might be at times inadequate preparatory to the discharge of sewage into some arms of the harbor, and further purification then resulting from the application of chemicals might be wise.

SLUDGE DISPOSAL.

In the clarification of sewage there results in all cases a substantial quantity of solid matters more or less mixed with water and ordinarily spoken of as "sludge." Experience at other large seaport towns employing sewage treatment works, such as London, Glasgow, Manchester and Salford, demonstrates conclusively that the barging of sludge to the open sea is the cheapest way of disposing of the sludge without nuisance.

Cases may arise in isolated areas removed from the water front where barging to sea will not be desirable. In such cases I advise the consideration of two-story tanks of the Imhoff type, with the drying of the sludge on drying beds so that the residue can be carted off readily.

Use for Fertilizer.—For more than 50 years efforts have been made to employ sewage sludge economically as a fertilizer. Rarely if ever has the result been a commercial success when operations were conducted on a large scale. I do not look for any substantial change in the future, although sewage sludge may perhaps be employed as a "filler" for fertilizers. In such event it is safe to assume that the users of the material coming from settling tanks would be willing to prepare it for their processes, and if this is so it would simply mean that New York would be relieved of operating barges to sea up to the limits to which sludge is diverted to fertilizer purposes. While some slight saving in the operation of barges may be accomplished, I do not look for any substantial compensation to the city from fertilizer manufacturers who might use the sludge as filler.

Incineration.—I am aware that at Frankfort, Germany, sewage sludge is freed of water by centrifuging and the application of heat in revolving drums, so that the dried sludge may be burned with other city refuse in an adjoining municipal incineration plant. While this procedure may have a future for inland cities, the expense will never be reduced to a point where this method will displace barging to sea for the sludge of a large seaport town.

Destructive Distillation.—In a small way there are some data available as to the production of combustible gases and coke by the dry distillation of sludge. The cost of removing the water is so great that the future will never see this treatment become commercially feasible.

Sterilization.—With the existing system of combined sewers, without any possibility or necessity of making the harbor waters of a "drinking water" standard of purity, sterilization for the great bulk of the New York sewage need never be seriously considered. For some of the outlying areas where there are shell fish layings or bathing beaches I anticipate a resort to sterilization.

MISCELLANEOUS PROCEDURES.

From time to time attention is attracted by claims for unusual efficiency or economy, or both, resulting from the employment of special types of strainers or other clarifying agents, or the use of oxidizing procedures resulting from the use of new chemicals or of new applications of electricity.

Presumably such claims will be heard from at intervals in the future, and it may be that some of them will produce more effect for a given cost than could now be attained.

I do not believe, however, that the business aspects of the New York problem will ever be materially modified by improvements in methods or devices which may become available.

As time advances a clearer understanding will be obtained as to just what is needed in certain of the subdivisions and the progressive installation of any project as large as the New York sewage disposal problem will give opportunity to adopt in the designs the current improvements, which, as above stated, will in all probability relate to minor details rather than to underlying principles.

The Collection and Treatment of Sewage in Philadelphia.

The sewers of Philadelphia as originally built discharged crude sewage directly into the rivers and other natural water courses. This eventually led to serious stream pollution in the Delaware and Schuylkill Rivers from which the city's water supply is drawn, and in order to protect public health it was necessary to take measures to alleviate this unsanitary condition.

The first work of this character was commenced about 1883 when an intercepting sewer was constructed along the East bank of the Schuylkill River, with a main branch, so located as to keep out of the water supply from the Schulkill River all the sewage collected in an extensive system of separate sewers. Recently an interceptor has been constructed along the East bank of Cobbs Creek to collect the dry-weather flow and first flush of the rain discharged from the combined sewers into that stream; and within the past two years an intercepting sewer has been built along Pennypack Creek, in the northeastern part of the city, to collect the sewage from the village of Holmesburg and from three large municipal institutions and convey it to treatment works, where it is purified sufficiently to protect the water supply taken from the Delaware River.

The city has, therefore, what may be considered three systems of sewers: The combined system which covers by far the larger part of the city, the separate system which has been adopted in the areas adjacent to the portion of the Schuylkill River from which the water supply is taken, and the intercepting sewers along Cobbs Creek and Pennypack Creek.

The city is now engaged in the preparation of a comprehensive plan for the collection and treatment of its sewage, for submission to the state department of health, and the work which is now being constructed is in harmony with the plans which will be recommended. The general aspects of the new collection and disposal systems are here discussed from matter taken from a paper by Mr. George S. Webster, Chief Engineer of the Bureau of Surveys, before the Sanitary Section of the Boston Society of Civil Engineers, as published in the Journal of the Society for May, 1914.

The investigations which have been carried on include a large number of studies of possible methods of collecting the sewage, the operation of a sewage experiment station, sanitary surveys of the water courses and rivers, and the construction and operation of a plant to treat the sewage in one section of the city adjacent to the water supply. The magnitude of the problem will be appreciated when it is considered that the area of the city is 130 square miles, that it has a population of 1,650,000, and that the water consumption is approximately 200 gals. per capita daily; this, with the infiltrated ground water from the older sewers, produces at the present time a large volume of sewage estimated at 400,000,000 gals. a day.

The problem of sewage disposal in Philadelphia is two-fold: first, to collect the sewage from the present system and abolish the nuisance which now exists in the large creeks and lower Schuylkill owing to the insufficiency of diluting water, and to carry it to distant points for treatment; and second, the protection of the public health by the treatment of the sewage, which is now and in the future will be discharged into the Delaware River, this treatment to be carried to such a degree that the drinking water can be safely and economically purified before delivery to the consumer.

It is of great importance, in preparing designs both for the collecting system and the treatment works, that relief may be given from the present objectionable conditions in the shortest possible time, that the work may be carried on economically and advantageously from time to time as funds become available, that each step taken may give some relief, and that it will not be necessary for the completion of the entire project before benefits may be obtained.

It was found early in the investigations that the cost of installing the separate system in those parts of the city already sewered on the combined plan would be prohibitive, for in addition to the cost to the city of laying sewage pipes in every street, the plumbing fixtures in all buildings connected to the sewer system would have to be rearranged so that the sewage and the rainwater could be carried in separate conduits. This latter expense, which would amount to many millions of dollars, would have to be borne by the individual owners.

In the design of sewers for the purpose of carrying sewage only, the factors used are the contributing population, water consumption and the amount of infiltrated ground water. It is quite common practice in many cities in designing sanitary sewers to calculate that they shall run half full, when carrying a water consumption of 150 gals. per capita from the population tributary. To reach a conclusion as to the quantity of sewage to be treated by the city of Philadelphia in the future, and to obtain data for the design of the intercepting sewers, gagings were made of the dry weather flow of a number of main sewers, some of which were located in solidly built up areas and others in partly built up districts, and from the factors thus obtained estimates were prepared, based upon the probable increase and density of population, of the quantity of sewage that must be cared for in the future, the estimates and population curves being projected to the year 1950.

The amount of sewage flow determined by the gagings in all cases included the infiltrated ground water, no practical way appearing by which it could be differentiated from the sewage proper. As a majority of the sewers in which gagings were taken are of considerable size and length, the variation between the maximum, minimum and average rates of flow is not as great as in smaller sewers. The mean of all gagings showed that the maximum flow was 128 per cent of the average, and the minimum 78 per cent of the average.

In addition to the flow of sewage, it has been decided to admit into the intercepting sewers, through automatic regulators, the first flush of the rain, which is usually as polluting as sewage, and the amount to be admitted has been fixed at 10 per cent of the maximum dry weather flow of the sewers, but a much larger percentage can be intercepted when the sewage is not flowing at a maximum rate. This additional 10 per cent makes a storm maximum flow of 141 per cent of the average flow.

When it is considered that the per capita consumption of water in Philadelphia is 200 gals. a day and that in the towns of England only about 40 gals. is used, it will be seen that by the arrangement proposed the degree of dilution of the sewage, in time of storm, compares well with the English practice of treating six times the normal dry weather flow.

In studying the methods of disposal it has been found that the sewage may be treated with much less offense if it reaches the works in a comparatively fresh state before putrefaction has set in, therefore great care is being taken in the design of the sewer system that the velocity of flow shall not be less than that required to carry the materials in suspension. This is accomplished by providing proper gradients and by the exercise of care to secure smooth surfaces, avoiding all roughness and projections on the interior of the sewer where organic matter might find lodgment and be retained until putrefaction sets in and stench begins. Upon examination of many of the sewers in Europe there was no odor noticeable because the interior surfaces were smooth, either vitrified tile or smooth glazed brick being used, and all connections so made as to provide a natural flow without the creation of eddies where deposits might occur. It is, therefore, recognized that a solution of a part of the problem of sewage treatment is to construct sewers with smooth interiors and to keep them clean and inoffensive.

In designing the collecting system, it is proposed to construct intercepting sewers at two levels, and in this way to utilize the potential energy in every foot of head and carry to the treatment works by the high level interceptors the greatest possible volume of sewage, and thus reduce to a minimum the quantity to be pumped.

The collecting systems in many European cities are constructed so as to convey the sewage to one or more suitable locations for treatment, and care is exercised in their designs to secure the greatest economies.

SANITARY SURVEYS.

There are in Philadelphia five large creeks and the Schuylkill River, all of which are tributary to the Delaware River, which forms the eastern boundary of the city. Poquessing Creek flows through a territory but little developed and is therefore not at the present time polluted. The sewage formerly discharged into Pennypack Creek and Wissahickon Creek has been intercepted and the water in these streams restored to a normal condition. A large part of the sewage on the Philadelphia side of Cobbs Creek has been intercepted and the condition of the water in this creek greatly improved. Frankford Creek, which empties into the Delaware River about five miles south of the Torresdale Water Filters, flows in part through a densely built up and industrial part of the city and receives crude sewage from about 140,000 people. It has several dams along its length and therefore low velocities. The water is not only grossly polluted by the discharge of sewage into it, but the deposits of sewage origin upon the bed of the creek add to the nuisance.

The Schuylkill River flows through the city in a generally southerly direction, and about midway there is a dam which forms the end of tidal influence. The section of the river north of the dam has been protected within the city limits by intercepting sewers. Into the tidal portion of the river below the dam, however, there is now being discharged the sewage from about 455,000 people. At times of drought almost the entire upstream flow is used for water supply, leaving a very inadequate volume of diluting water for the sewage from this large population. The examinations made during the summer months showed the water in this part of the river to be depleted of dissolved oxygen. Furthermore, the tidal velocities in the lower part of the Schuylkill River are insufficient to maintain the sewage matter in suspension, so that in addition to the polluted condition of the water, the putrefying deposits upon the bed of the river increase the nuisance, particularly in warm weather; but as in the case of Frankford Creek, above described, the natural sedimentation processes and the gasification of the resulting sludge, together with the refreshing action of the tide, lighten the load of organic matter placed upon the waters of the Delaware River.

The Delaware is one of the large rivers of the United States, and forms the natural drainage for portions of the states of Pennsylvania, New York and New Jersey. The normal flow of upland water is at the rate of 4,050 sec.-ft., in addition to which there is a tidal range of 5½ ft., and it is estimated that during the ebbing of the tide 2,421,000,000 cu. ft. of water flow past the city. As the sewage of the city at present and the effluent from the treatment works in the future must be disposed of in the waters of this river, its present condition has been examined with considerable care and it was found that with the exception of the docks, where sewers discharged, the Delaware River is successfully disposing of the crude sewage of the present population of Philadelphia in addition to that of the neighboring towns. Even in summer weather and in times of extreme drought, there has been no nuisance or offense created, although the amount of dissolved oxygen in the river has been small. The surveys indicated that the river water after passing beyond the points of discharge of the sewage of the city gained [illegible]

velocities, due to tidal flow in the river, maintain the sewage matters in suspension, and the examination shows that the entire bed of the river (excepting the docks) is clean and free from deposits of sewage origin.

It must, however, be realized that, with the increase in the population and the consequent added load placed upon the river, its oxidizing power will soon be overtaxed and that the time to begin the building of the collecting and treatment works is at hand.

TREATMENT WORKS.

The sanitary surveys of the water courses in Philadelphia show that sewage must be excluded from the creeks and the Schuylkill River, and that the treatment works must be located so as to discharge their effluents into the Delaware River in order to utilize to the fullest extent the great diluting and oxidizing capacity of that river.

It is proposed to locate the first treatment works in the northeast section of the city. The collecting system tributary thereto will eliminate the pollution of Frankford Creek and also prevent the discharge of crude sewage into the Delaware River within the tidal influence of the Torresdale Water Filters which provide three-fifths of the city's water supply. The degree of treatment required at this works must therefore be based upon a hygienic standard in order that the public health will not be jeopardized by overtaxing the economical and safe operation of the water filters.

The second treatment works will be located in the southwest part of the city, near the mouth of the Schuykill River, the most distant point within the city limits from the source of water supply. The collecting system tributary to this works will eliminate the pollution of the lower Schuylkill River and will result in concentrating the sewage from over half the population of the city at one point for treatment. As the effluent of this works will be entirely below the influence of the city's water supply, the degree of treatment required need only be sufficient to prevent nuisance in the Delaware River.

It appears to be economical to construct temporarily a clarification works in the southeast district, to care for the sewage now discharged into the Delaware River below the center of the city.

TREATMENT.

In selecting methods for the disposal of the great volume of sewage produced in large cities, the adaptability of the various processes to a comprehensive plan must be considered so that the treatment works may be constructed by successive steps as needs arise for more refined treatment and as funds become available. It is desirable to obtain intensive methods, so as to secure a maximum of efficiency upon a minimum area of land, but in all cases exercising care to prevent nuisance from odors.

Various methods for the treatment of the sewage of the city of Philadelphia were studied, and those best adapted to the local conditions selected.

It has been frequently urged that the sewage of the city could be purified to advantage by applying it to farm land. Mr. John D. Watson, after years of experience, aptly states that this method of disposing of sewage "may be ideal in theory, but it is difficult, if not impossible, to obtain the ideal on a farm of large size." Berlin and Paris dispose of their sewage in this way, but, owing to the small volume of sewage which can be treated per acre, large areas of suitable land are required.

The Metropolitan Sewerage Commission of New York City has estimated that 175 square miles of land would be required to treat the sewage of that city if it were applied at the rate of 12,000 gals. per acre, and that the cost of this method of treatment would be $153,000,000, and therefore dismissed it as being impracticable. The city of Birmingham, England, has abandoned its sewage farms and substituted the more intensive biological method, and the same course will probably be followed in Paris.

To treat at the present time the sewage of Philadelphia on farm land would require an area of approximately 60 square miles. To secure this amount of land in Pennsylvania adjacent to the city would be prohibitive on account of the cost, and would be opposed by citizens and property owners, hence this method of treatment need not be further considered.

London, the largest city in the world, with a population of 6,000,000 people, situated on the banks of a river with but little larger flow of upland water than the Schuylkill, disposes of its sewage by removing about 75 per cent of the suspended matter by chemical precipitation and depends upon the oxidizing power of the river to accomplish its ultimate purification. That this is being successfully accomplished may be seen from the diagram which was prepared by Sir Maurice Fitzmaurice, late chief engineer of the London County Council, showing the percentage of saturation of the Thames River with dissolved oxygen, in connection with which he states, "With respect to the minimum amount of dissolved oxygen that should be present to prevent offense, it is rather difficult to answer this exactly, but I may say that the only complaint in recent years was for a short time in 1901."

While this method has been successfully used in London, it would not be applicable to Philadelphia, on account of the long haul to dispose of the sludge in the ocean, over one hundred miles distant. Another objection is the large quantity of sludge produced. From the best information available, it appears that the sludge, containing about 95 per cent water, resulting from this method of treatment, amounts to 800 tons per day for a population of 500,000 people.

The city of Manchester, England, has probably the largest installation of contact beds. These are found to be expensive to operate and fail to produce an effluent up to the requirements of the Rivers Board. The consensus of opinion among experts in England seems to be that contact beds for a large installation are not as efficient as percolating filters.

From the results obtained at the Philadelphia Experiment Station and from the plant in operation at the Pennypack Creek, confirmed by the testimony of the city engineers who inspected a number of plants in Germany, it has been found that the two-story sedimentation tank, known as the Emscher tank, offers the best solution for the preliminary treatment of the sewage. The advantages are that the separation of the settling sewage from the digesting sludge maintains the sewage in as fresh a condition as it enters the tank, that it is equal in efficiency to any other type of tank in removing the suspended matter with shorter retention periods and that the sludge withdrawn is without offensive odor, is smaller in volume than sludges resulting from other processes and more easily dried, and is so thoroughly decomposed that it resembles garden soil and may be used for filling in low lands without nuisance.

It is the purpose to recommend for the northeast and southwest treatment works the following processes in sequence: Coarse screens to restrain the large floating objects, grit chambers designed to intercept the inorganic matter only, two-story sedimentation tanks of the Emscher type, and percolating filters or such other improved methods of oxidation as may be developed by the time this refinement is needed. All of these processes are so related that they can be incorporated in the work successively; each one is the most intensive of its kind, therefore a minimum amount of land will be required for the works.

At the Southeast Works it is proposed temporarily to clarify the sewage either by fine screening or by the tankage method and then to discharge it without further treatment into the river. If more refined treatment is required in the future the effluent from this plant may be carried to the Southwest Works, where ample area is being provided for additional processes.

The full utilization of the diluting and oxidizing power of the river water largely depends upon securing a thorough mixture of the effluent from the works and the water of the river. At each of the three proposed treatment works for Philadelphia it is planned to accomplish this by discharging the effluent through submerged outlets into the main channel of the river.

The distribution of sewage over the surface of percolating filters is one of great importance, as efficient distribution will allow the use of high rates. At Bolton and at Hampton, in England, a traveling distributor is used. This requires but little head, but it is doubtful if it could be successfully used in countries subject to severe winter weather. At Wilmersdorf, Germany, and in a number of English plants, there are percolator filter installations in which distribution is effected by means of rotary arms. This method accomplishes good distribution but has not been looked upon with favor in America. At Birmingham, distribution is through fixed nozzles operating under a constant head, and this method was followed in the early American installations, but it results in uneven application of the sewage. Latterly this has been improved by the use of the tapered dosing tank with syphonic discharge.

At the Pennypack Creek Works in Philadelphia there is in service a method of distribution through fixed nozzles, operating under a fluctuating head, which yields results equal to that from a mechanical distributor.

The work so far accomplished has demonstrated the feasibility of the methods suggested for the comprehensive treatment of the sewage of the city. The state board of health having been in touch with the work so far completed, it is anticipated that the plans to be recommended will meet with its approval; and it is hoped that funds will soon be available to commence the work on the larger installations.

WATER WORKS

Designing Small Water Works Systems.

As we have many times stated, the proper design of a small water works system calls for the services of a good engineer. The designer of small works is nearly always narrowly limited as to funds and must find the best solution of the problem consistent with the small, fixed sum of money available for his use. Moreover, most water works systems are small ones, and this, of course, is especially true at this time when nearly every city in the country of any considerable size is provided with some sort of a public water supply. In many small towns, already supplied, changes must be made owing to inadequate original installations. To engineers for such localities and for small towns not yet supplied, a consideration of the sound engineering principles underlying the design of small water works systems will prove most helpful. We are here giving, in substance, the paper on this subject presented before the New England Water Works Association by Mr. William S. Johnson, Hydraulic and Sanitary Engineer, Boston, Mass.

In Massachusetts there are 215 water supply systems, and of these, 152 were installed before the population of the towns they supply was 5,000, and 111 or more than half are now still supplying towns of less than 5,000. In fact, the opportunity seldom comes to design a complete system of works for a large community, the larger systems being extensions of the system designed for the small town. As there are only four towns in Massachusetts having a population of more than 3,000, which remain unsupplied with water, it is likely that, in that State at least, the problems arising in small systems will be more numerous than the problems connected with the larger systems although perhaps not so interesting to any but those immediately affected.

It has been my lot to put in a number of small waterworks plants, and it has surprised me to see how easily 10 per cent of the cost of construction may be saved by a thorough study of the problem. I have also found that many problems over which I have worked have been solved by others who have kept the results to themselves, not because of unwillingness to part with the information, but because the matter seemed too small to be of general interest. It is with a view of encouraging the discussion of the problems connected with small water works installations, as well as to give a few of the results of my own experience, that this paper is presented

Provision for Future Requirements.—The works should be designed for a long time in the future, but the design should provide for as little immediate construction as is possible except where the cost of extensions or increased capacity will be much more than the cost of doing the work in the beginning.

To build works for a long time in the future assumes a power of prophecy which most of us do not possess. It is impossible to foretell the future growth of the whole or any part of the community. The advancements in the art of water supply engineering are so rapid that portions of the plant are likely to become obsolete before they are worn out. The requirements of the public which uses the water are changing very rapidly. For these reasons it is desirable to build only for the immediate future, and to plan for additional works to be constructed as they become necessary.

Fire Protection Requirements.—The capacity of the distributing system is determined, in the case of the small town, by the fire-protection requirements solely. One good effective fire stream will use water at as great a rate as the ordinary town of 5,000 inhabitants will require for domestic purposes during the hours of maximum draft.

The quantity of water which will be drawn from the sources of supply depends upon the population to be supplied, the character of the residences supplied, the care taken to prevent leaks and other wastes and the use of water for manufacturing or mechanical purposes. It is exceedingly difficult to evaluate these factors since it is impossible accurately to forecast the industrial development or retrogression of the average small community. The population to be served depends primarily upon the growth of the community's industries. A future per capita domestic consumption of from 75 to 100 gals. per day for ordinary small towns does not seem unlikely and in the case of towns having large estates and a large area of well kept lawns the consumption may be much larger.

Metcalf, Kuichling and Hawley, in a paper before the American Water Works Association, published in abstract in ENGINEERING & CONTRACTING of June 7, 1911, state that "the cost of the portion of the waterworks plant involved by fire protection probably constitutes from 60 to 80 per cent of the entire cost of the physical property in the case of communities having less than 5,000 population." This is undoubtedly true except in those places where it is necessary, in order to secure water of sufficient purity for domestic purposes, to go to a large expense in obtaining it or in its purification.

In a small town it is usually necessary to depend entirely on hydrant streams without the use of steamers and it is essential, therefore, that the works, either by themselves or assisted by some outside source, should furnish both the requisite quantity of water and the proper pressure with which to fight the greatest conflagration which is likely to occur, and to do this under the most unfavorable conditions in regard to domestic consumption and quantity of water in reservoir or standpipe.

Fire Streams.—The standard fire stream is now considered to be that thrown by a 1⅛-in. smooth nozzle, discharging 250 gals. per minute, and it is generally considered by the insurance engineer that a hose stream which does not throw 200 gals. per minute is not a good stream. There are many cases, however, where smaller streams throwing from 150 to 175 gals. per minute would furnish reasonable protection and this is all that a town would be justified in providing in some districts. In the outlying sections of small towns, any fire which gets sufficient headway in the ordinary building so that it cannot be controlled with two streams of 150 to 175 gals. each is not likely to leave much of value if it is extinguished by using six streams. In such cases, the value of the water is chiefly in saving adjacent buildings and for this purpose even small streams are of great value. For streets in a district where the houses are small and occupy comparatively large lots with no prospect of any considerable increase in density of population, and where extensions of the mains are not likely, a hydrant which will furnish 300 gals. per min. under a suitable head to any building in the territory supposed to be covered by this hydrant will be good fire protection. More is desirable but the advantages are not sufficiently great to warrant the expense of larger mains to secure it. In a district where the houses are nearer together but where there are no businss blocks, apartment houses or other large buildings, it should be possible to get 500 or 600 gals. per minute at any point. Where there are business blocks and other large buildings and where the buildings are very close together, as they frequently are in the center of a small village, it should be possible to get 1,000 gals. per minute. Where there are factories or other special fire risks, a much larger quantity may be necessary and a special study should be made of each case.

Pressure and Hydrant Spacing.—The pressure required at the hydrants while the hydrants are being used should be great enough to force the water through the greatest length of hose which will be used and throw it to a sufficient height to cover any building. The hydrant spacing, and the pressure should, therefore, bear some relation to each other. Table I, from E. V. French, gives the limit of a good efficient fire stream with different lengths of good rubber-lined cotton hose, with a constant pressure of 60 lbs. at the hydrant, with moderate wind. Table I also gives the corresponding amount of water which would be discharged.

TABLE I.—HEIGHT AND VOLUME OF ONE 1⅛-IN. STREAM FLOWING FROM A SMOOTH-BORE NOZZLE.

Length of hose, ft.	Limit of height, ft.	Discharge, gals. per min.
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]

Table I shows the importance of having hydrants near the buildings to be protected. Unless the hydrants are near enough to furnish water at the fire under a good pressure, the expense of large pipes and a high reservoir is largely wasted. The cost of a two-way hydrant in place is about $40 and in the ordinary distribution system about eight hydrants are required per mile of street main to keep the hydrants within 250 ft. of every building in the territory covered; so that the expense of hydrants is very small compared with the expense incurred in other parts of the system to obtain efficient fire protection. In general, hydrants should not be more than 500 ft. apart in the outlying sections. In the more closely built-up sections they should be so spaced as to make it possible to get the number of streams which are considered necessary at any particular point with the use of not more than about 300 ft. of hose for each stream. This figure may be modified, however, if the pressure is unusually high and should be if the pressure is unusually low.

The minimum pressure desirable for good fire protection with hydrants spaced as suggested is about 50 lbs. per square inch at the hydrants when they are in use. This pressure will give, with 300 ft. of best quality hose and a 1¼-in. nozzle, a stream of 185 gals. per minute, which can be thrown to a height of 44 ft. With 200 ft. of hose, the quantity thrown would be about 200 gals. per minute and the height would be increased to 50 ft. There are cases where in the higher sections of the town these pressures are almost impossible and in such places the hydrants should be so located as to require as little hose as possible. In the case of thickly built up villages, the pressure should be 60 lbs. per square inch at the hydrant when the water is being drawn at the maximum rate. This pressure will give a stream of more than 200 gals. per minute with 300 ft. of hose and with 200 ft. of hose will throw 222 gals. per minute to a height of about 60 ft.

[illegible] Water Mains.—The size of the mains depends entirely on the requirements determined on to give fire fighting facilities and on the head available. When these are known, the system can readily be designed. Generally, however, the head which can be secured is not fixed but can be made whatever is desired by going to additional expense and the determination of the most economical arrangement of height of reservoir, size of pipes and spacing of hydrants is a matter for careful study. The rule adopted in many places to put in no street main less than 6 ins. in diameter in most cases works out properly but there is no excuse for it as an arbitrary rule. A short street will many times be better served with a 4-in. pipe than other streets in the same system with 6-in. mains and the money saved by using the smaller pipe could well be expended in strengthening those parts of the system which are weaker. The standard should be the quantity of water the pipe will deliver and the head under which it delivers this amount. If a 4-in. pipe will do this satisfactorily, there is no good reason why it should not be used.

The loss of head due to friction in a 6-in. pipe which has been in the ground for some time, when water is being drawn at the rate of 300 gals. per minute, is about 10 ft. per 1,000 ft. of length, and this figure, together with the required pressure at the hydrant of 50 lbs. per square inch should, in general, determine the allowable length of 6-in. pipe as [illegible]

The maximum desirable pressure is a matter on which there is much disagreement, but the limit is constantly being extended. If it should prove to be more economical to have a system where the pressures run up to 150 lbs., there would seem to be no good reason why this should not be done in a new system of waterworks and it would be likely to prove much more satisfactory than to maintain two levels. In 86 small towns in Massachusetts the average static pressure in the central portion of the town is 79 lbs. per square inch. Nine of these towns have pressures of less than 50 lbs.; 33 have pressures of from 50 to 75 lbs.; 31, from 75 to 100 lbs., and in 13 the pressure is more than 100 lbs.

Pumping Engines.—The development of the oil and gasoline engine has done much to make waterworks systems for small places financially possible. When the only available pumping machinery was the steam pump, the cost of installation of pumps and boilers, the cost of the pumping station to house them and the cost of maintaining the plant made waterworks practically out of the question unless a gravity supply could be secured. With the new form of engine, however, the conditions are quite different. The cost of the machinery has been greatly reduced, the cost of the station is much less and the cost of operating is reduced to a minimum on account of the fact that with oil or gasoline there is no consumption of fuel except while work is being done, while with a steam plant a large proportion of the coal is used in banking the fires and getting up steam. Electricity is also becoming an important factor in connection with small pumping plants, although the cost of current is so great (in New England) that there are few places where oil or gasoline are not more economical, except for auxiliary plants used only occasionally.

Pumps should usually be designed for the greatest economy in doing the work which they are called upon to do regularly in supplying the domestic needs of the town, without regard to their use for fire protection purposes. It is seldom feasible in the small system to have pumps of sufficient capacity to be of very great value in case of fire, and dependence for fire protection should be placed on water stored in a reservoir or large standpipe or tank, or on some connection with factory pumps through which a large supply of water can be quickly secured. With increased size of pumps it is necessary to have a larger force main, a larger suction pipe, more wells, if the supply is taken from driven wells; in fact, a considerable portion of the plant must be increased in size and made more expensive in order to operate large pumps. Large pumps are somewhat more efficient than small ones and if an attendant remains at the station while the pumps are in operation there is a saving in the shorter hours required with the large pump. With the oil engine, however, or with electricity, constant attendance is unnecessary, especially in the case of the smaller plants.

Pumping machinery should always be provided in duplicate, and works, although designed to run most economically when one unit is in operation, can be operated, if necessary, at double capacity with a somewhat reduced efficiency. A plant designed for a community which will use from 100,000 to 150,000 gals. per day, should generally have a capacity of about 250 gals. per minute for each unit. This would mean the operation of one of the pumps for from six to ten hours each day. Such a plant would give two large fire streams in case of fire by starting both of the pumps. Many pumping plants are undoubtedly of too large capacity for economy and the tendency in recent years has been to make them smaller, especially when the power used is some form of internal combustion engine.

Distributing Reservoir.—The design of the distributing reservoir is affected chiefly by the topography, the requirements for fire protection and by facilities for pumping. In a much less degree it is affected by the consumption of water. The effect of the topography upon the design is generally to change the reservoir from what is desirable to what is practicable. In a comparatively flat community it is practically impossible to store as much water at so great an elevation as is desirable and in such cases the distributing reservoir must be cut down and other portions of the system must be designed to do the work which should properly be done by the distributing reservoir.

When the topography is such that it is feasible to build a reservoir of any desired size and any height, the design is dependent almost entirely upon the requirements for fire protection. Generally it is found that the desirable static pressure from the reservoir at the point where there is likely to be the greatest demand for water is from 80 to 100 lbs., depending to a large extent upon the distance from the reservoir to the center of distribution. The reservoir should be, if practicable, large enough to hold at the required elevation, in addition to the domestic supply for 24 hours, a sufficient quantity of water with which to fight any fire which is likely to occur. A fire in the built up portion of a village may take about 1,000 gals. per minute or 60,000 gals. per hour. In general, the time during which this quantity will be required will not be more than from two to four hours. Applying this rule to the ordinary town with no large fire risks, the capacity of the reservoir or standpipe should be from 300,000 to 400,000 gals.

Pipe Thickness.—The part of the design in which theory plays the smallest part and in which even experience is likely to count for little is in the determination of the thickness and weight of cast iron mains. The static pressure which pipes have to withstand and the breaking strength of the cast iron are the only elements in determining the proper thickness of the pipes which are even approximately known, and determining the thickness from these elements alone would give pipes of about the thickness of cardboard.

The formula used by the New England Waterworks Association is:

$$t = \frac{pr}{\frac{1}{5}(16{,}500)} + \frac{p'r}{\frac{1}{5}(16{,}500)} + 0.25$$

where t is the thickness of the shell in inches; r, is the radius of the pipe; p, the static pressure in pounds per square inch; p', an assumed water hammer in pounds per square inch; 16,500 is the breaking strength of cast iron ;and 5 is a factor of safety.

The chief uncertainties in the determination of the proper thickness of pipes are the water hammer, the effect of corrosion, the possibility of breakages in handling, the strains due to imperfect foundations or unequal settlement and the eccentricity of castings and other imperfections in the pipe. There is no reason why the water hammer in a small system should not be kept below the figures ordinarily used in the formula. There are few, if any, authentic cases where corrosion of a cast iron pipe has caused its failure. In fact, if a pipe has been in the ground long enough to corrode, it seldom fails from any cause. The strains due to imperfect foundation and settlement, in the case of small pipes, can be neglected if proper precautions are taken during construction. The difficulties due to imperfections in the casting and to the handling of thin pipes are the most serious and to overcome these is the duty of the founders. That they will be overcome, if the engineers insist on light pipe, there is no doubt, for already much has been done along these lines; the cost per ton may be somewhat increased if lighter pipes are used, but this increased cost will be nothing like the saving accomplished by the use of the lighter pipe.

In my own practice, I have put in many miles of Class C pipe where the pressures run up to 115 lbs. per square inch and have never known of a failure which would have been prevented by using thicker pipe. The breakage in the handling may or may not have been greater. In any case, it was not excessive. For the ordinary conditions in a small town, I would never use a heavier pipe than the Class C and lighter pipes may safely be used in many cases.

[illegible] Pipe Lines.—The depth to which street mains should be laid has been investigated by a special committee of the New England Waterworks Association, see report presented at November, 1909, meeting, and the experience of the cold winter of 1911-12 has given valuable experience to those who have had charge of waterworks. The depth determined on affects the cost of the works materially, especially if rock is encountered, and if it is safe to reduce the depth it certainly should be done.

Theoretically, street mains might be laid at [illegible] different [illegible] a foot nearer the surface in clay than in gravel, but in the average New England town there are so many soils that it is not wise to make any distinction. The only discrimination which it would appear safe to make is in the case of places where the ground water always stands near the surface, where the pipes may be laid in shallow trenches. The freezing of the pipes is such a serious matter that it would seem to be unwise to take any chances in an attempt to save money on trench excavation. The best practice seems to be, for a climate like that of Massachusetts, to have the center of the pipe from 4.75 to 5 ft. beneath the surface.

Notes on Meterage From Various Cities.

In our report of the latest annual convention of the American Water Works Association we commented upon the discussion which made "Superintendents' Day" so much of a success. The subject of water meterage is always of immediate interest to the operators of water works and this subject brought out a very interesting discussion at the session of the convention to which we have referred. The formal questions, introduced under the Question Box, which prompted the discussion, here presented in substance, were as follows: Do water meters increase or decrease the cost of water supply to consumers? Should meters register in cubic feet or gallons? How is the cost of installation and maintenance of meters borne? What is the most economical method of reading meters? The discussion brought out many interesting data pertaining to various water systems. Some speakers touched upon several phases of the meterage question and all, presumably, presented the most interesting aspects of the subject as viewed in his locality. For that reason the comments of several speakers are here given in substance.

Mr. A. A. Reimer, superintendent of water at East Orange, N. J., speaking on the effect of meterage on the consumer's bill said that he had taken the actual records of several hundred cases from the old flat rate system, and compared them with the same places under the meter system. He finds that 71 or 72 per cent of the consumers who go from flat rate to meter rate save money. The department makes that up from the other 28 or 29 per cent. Thus the department breaks even, but over 70 per cent of the people actually save over the old flat rate.

Speaking along the same lines Mr. J. N. Chester, of Chester and Fleming, Pittsburgh, Pa., stated that plants where he had made investigations show these results: To big hotels, saloons and large consumers generally, the change from flat rates to meters ordinarily increased the cost of water; families who have servants in the house generally pay more, and those who handle their own spigots pay less. He concluded that it depends largely upon the community. In a community of workingmen's houses—unless the meter rates are minimum or extremely high—he believes that water rents will be cut down by putting in meters. On the other hand, in residences where there are servants who are careless, and who let the spigots run most of the time, failing to report leaky plumbing, etc., the consumer is going to pay more for the water metered.

Mr. Wirt J. Wills, superintendent of water works at Memphis, Tenn., considered that the installation of meters decreases the price to consumers since the people voluntarily asked for meters four years ago. His city has 16,000 meters. Up to recently the department had no right to put a meter on unless the consumer asked for it. Most people have had experiences with gas, electric and other kinds of meters, and are naturally prejudiced against the meter; but after 8,000 had them in and got to talking around to their friends, in the last four years 2,000 a year were put in, the consumers voluntarily asking for them. The cost of installation is charged up to construction of the plant. The benefit to the water plant is from the fact that most of the water is being paid for. The works are furnishing per capita about 90 to 95 gals. per day, but are not pumping as much water as ten years ago when the city was some 40,000 less than now in population. Mr. Wills believes that metering has reduced the consumption approximately 4,000,000 gals. per day.

Speaking as to the ownership of meters Mr. C. W. Wiles, Secretary of the Delaware, Ohio, Water Co., stated that in his opinion the department should furnish the meters. It is a question whether the department should charge rental to the consumer. That is optional; but in some places the department furnishes the meter and charges a rate sufficient to cover the cost and maintenance of the meter. He does not believe the department should maintain meters against frost or freezing; he thinks the consumer should pay the cost of that, because it is often due to his neglect that a meter freezes. He believes the department should furnish the meter, and should charge a nominal amount to maintain it in good repair. In his opinion the ordinary domestic consumer will save over a $16 or $18 flat rate, $4 or $5 every year by the use of meters, and, on the other hand, that the department will save in consumption and wasted water.

Mr. H. C. Hodgkins of Syracuse, N. Y., gave an effective illustration, drawn from the experience of his city, of the economy resulting from meterage. He stated that in the city of Syracuse the meter rate has been reduced to 12 cts. per 100 cu. ft. The consumer pays for the meter. The department charges the repairs to the consumer. It ought to be self-evident that the consumer will save by purchasing water at that rate. But there is one other important saving: On the conduit line at the outset the estimated cost was $1,000,000, or something less than that. Fifteen years following the installation of the first conduit a second one had to be built at a cost of, say, another million dollars. It is estimated that in 1925 an additional conduit, or additional supply, will be necessary. Four per cent on $1,000,000 means $40,000 a year saved to the taxpayers; so that while the consumer saves from his water payment the city budget is also saving a large item in interest charges per year every year that another conduit is postponed, due to the saving of water through meters.

Procedure as to meter charges in Madison, Wis., was explained by Mr. E. E. Parker, superintendent of water works and city engineer. The city supplies a meter for every service. Two charges are made on the supplies; the first charge being a capacity charge, or the charge made whether any water is furnished or not; the second charge is the output charge, or the cost of supplying the water used. The first charge on a ⅝-in. meter is $1.50 for six months. It represents the cost of maintaining the service, interest on investment, cost of reading meters, etc. The second charge is 6 cts. per 100 cu. ft., and represents the actual cost of supplying the water.

The saving in plant effected by meterage was commented upon also by W. C. Hawley, general superintendent of the Pennsylvania Water Co., of Wilkinsburg, Pa. He said that it is not only a question of what the consumer saves on his water bill, but also as to what is saved in the way of additional investment in the plant. At Atlantic City some years ago the consumption per capita was from 250 to 260 gals. After meters had been installed two years this consumption was cut down to 50 or 60 gals. per capita. This postponed an investment of more than a quarter of a million dollars for a number of years, and the saving in coal alone at the pumping station more than paid the cost of installing meters.

At the plant with which Mr. Hawley is now connected they are furnishing a large population just inside and outside of the city of Pittsburgh. Water is supplied on a meter basis exclusively, or practically so. The consumers are paying less at the rate of 20 cts. per 100 cu. ft. than the people of Pittsburgh who have corresponding properties are paying on a flat rate. Pittsburgh has a nominal rate of 18 cts. per 1,000 gals., or 13½ cts. per 100 cu. ft., but is selling water to less than 10 per cent of its domestic customers by meter. In this case the meter rate is doubtless too low for the cost of the service, but when the city does go on to a meter basis, as it must in the not distant future, and has increased its meter rate to a reasonable figure, the great majority of those now paying on a flat rate basis will save money, and the reduction in waste will materially reduce the expense of operating the city's plant besides deferring large investment for increase in plant. This is a concrete illustration of the saving to be effected by the introduction of meters, and, as shown, the saving is not alone in the reduction of water bills, but a material part of the saving of investment which would be necessary if it were not for the meters.

Mr. Chester A. McFarland, general manager of the Tampa (Fla.) Water Works Co., presented the following interesting data on the decrease in operating expenses and revenue following the adoption of meterage in his city. His plant is operating about 1,800 meters. The consumer pays for the first installation, and after that the company pays all repairs. If that meter for any reason wears out or becomes useless the company installs another meter at its own cost. The consumer pays for one installation only. The result is very satisfactory to the company.

The revenue is falling off in most cases where meters have been installed, but the falling off in revenue has been amply compensated by the saving in the cost of additional installation and in the consumption of water. Water is pumped from well supplies and the securing of this water is expensive, so that it is thought wise to keep down the consumption and to be paid for what the works are actually serving. The rate is 17 cts. per 100 cu. ft. They do not use the term gallons because of the trouble in computing quantities. Therefore they are using cu. ft. as a basis, and 17 cts. per 100 cu. ft. is the maximum charge, with a discount of 10 per cent. The first 5,000 ft. consumed in a month is charged at 17 cts., the second 5,000 ft. at 15 cts., and so on down to 9 cts. for the large consumers. The consumption is only about 60 gals. per day per capita.

At Nashville, Tenn., Commissioner of Water Works Robert Elliott states the pumpage has been reduced 15 per cent and the revenues have been increased 25 to 30 per cent during the last five years. There are about 18,000 services in the city, and 15,000 of these are metered. Meters are furnished and installed by the department, but it does not install the services. The department keeps the meter in repair without cost to the consumer.

Mr. G. Shoemaker, secretary of the Waterloo Ia., water works, stated that in his city there are 4,600 consumers and 4,500 meters. The meter rate is 25 cts. per 100 cu. ft. On the old flat rate the average was $15 a year. Out of the last 1,917 consumers who were changed from flat rates to meters, for 97 of that number the rates remain the same, for 1,106 the rate was reduced an average of $3.43 a year, for 714 the rate was increased an average of $4.80, making a total decrease for the 1,106 consumers of $3,800.68; the total increase for the 714 consumers $3,429.39, or a total loss on 1,917 meters of 19.4 cts. per annum each. In 1910 there were 3,100 consumers, and the pumpage was 526,000,000 gals. per annum. In 1913 there were 4,600 consumers, and the pumpage was 490,000,000 gals.

Mr. E. E. Davis, superintendent at Richmond, Va., said that everybody in Richmond was at first opposed to meters, because they said people did not use water enough to keep their premises clean, and that the question of saving waste was simply imagination on the part of the head of the water department. On one occasion, to test it, Mr. Davis had four houses supplied from an 8-in. tap under 80 lbs. pressure. The rate was at that time 15 cts. per 1,000, with no scale. He put a first-class meter on that tap to demonstrate to the city council that it was mistaken and not the water works people. The meter was started at 3 o'clock on the 31st day of July. Mr. Davis sent for the man who set the meter, and said to him: "Go down there now on the 31st day of August, which will make it just 31 days, and bring me a report of that meter." The report read 456,000 gals. of water for four houses in 31 days. The rates they were paying at that time were $3.38, $2.38, $2.88 and $3.25 respectively per quarter. We have never had an excess bill over half a dozen times in three years for any one of those houses. They are all getting water now for $2 for three months and using 13,500 gals. There is a difference between the flat rate and the meter rate, concludes Mr. Davis.

Commenting on noisy water meters, Mr. A.

A. Reimer said that there had been a considerable number of complaints in East Orange in this particular. In fully nine out of every ten cases he finds that the fault is not in the meter direct, but is due to the faulty construction of the piping system in the house, which simply acts as a telephone or vibrator, carrying the noise all over. When the department gets complaints of that kind it sends a man around to investigate. He puts a few pieces of felt between the pipes and their supports, and then there is no more trouble from noisy meters. In the few cases where the noise is directly attributable to the meter on investigation they find that something has gone wrong, usually in the gearing, which allows more play than there should be to the working parts.

A method of economizing time in reading meters was outlined by W. S. Cramer, superintendent of the Lexington, Ky., Hydraulic and Manufacturing Co. He spoke as follows:

We read 5,600 meters in 36 hours by two men last month. From the time we started reading those meters until the bills were ready was seven days. The meters are located on the sidewalk. We have mostly straight meters, all set at the curbstone, with 24-in. covers on the meter boxes and 18-in. extensions on the meters. There is no box to open. There is a small center cap on a large top. We use a 20-in. concrete box. In the center of the large top there is a small cover 6 ins. square, and in reading that a small stick is used about 18 ins. long, with an iron shoe on the end of it. All that is necessary is to leave that center cap so that you can see the dial. We never leave the caps down, because they are protected by the iron top. It is generally a question of reading only one figure. We use the card system entirely for preserving the reading. These cards are distributed according to routes, and when they are taken out they are thrown in route form in their consecutive number, so that a man can carry these cards in his hand, just slipping one over the other so that he can easily refer to the former reading. We never read the odd feet. We just read the nearest hundred, so that on the ordinary house service it is generally necessary to read only one figure.

We are troubled very little with moisture under the glass. I think two years ago last February in the extreme cold weather we had a little trouble with moisture, but we carried an extra man, who broke the moisture loose. It possibly reduced the record a little at that time, but very seldom do we have any trouble with moisture.

Water Filtration and the Mechanical Washing of Filter Sand at Wilmington, Del.

The present article relates to certain unique or unusual features in the design and operation of the water filtration works of Wilmington, Del. The article is based on a paper by Edgar M. Hooper, Jr., and James M. Caird before the latest annual meeting of the American Water Works Association. The authors acknowledge their indebtedness to the notes of Capt. C. H. Gallager, Theodore A. Leisen and John A. Kienle, former chief engineers of the water department.

The source of supply is the Brandywine Creek. The water is conducted by means of a race from the dam to the preliminary filters, thence by gravity to the pumps, thence to the sedimentation reservoir. After about three days in this reservoir the water flows by gravity to the final slow sand filter bed, thence to the filtered water reservoir, and finally to the 48-in. steel distribution main supplying the high service system direct and the low service reservoir.

The preliminary filter plant consists of ten beds of 14 ft. 5 ins. by 100 ft. and a reservoir 105 ft. 9 ins. by 30 ft. 4 ins., inside dimensions, with a depth in each of approximately 8½ ft. The construction of the foundations and filter walls is of reinforced concrete, the superstructure being of brick, with limestone trimmings. After drawing the water from the race, it is fed to the beds from an influent gallery and, passing under the filter medium, through the inlet drains, is distributed by them over the area of the bed. The system of filtration is upward, the water rising from the drains through 6 ins. of gravel of various sizes, thence through 20 ins. of small coke, and finally through 20 ins. of sponge clippings, the later being held in place by cypress racks. After passing the bed, the water is delivered into an effluent gallery and conveyed thereby to the clear well.

The distinguishing feature of this plant is the method employed in cleaning the filter medium. Unlike other plants of its type (in which the sponges are removed) the washing of the coke and sponges is done directly in the beds with air and water.

For this purpose, there is installed in the bed under the filter medium, a system or manifold of air pipes, consisting in each bed of two 6-in. main headers with ½-in. laterals at 12-in. intervals. The latter pipes are plugged at the end and perforated with 1-16-in. holes on 3-in. centers. The flow of air is secured from a rotary blower, and is discharged through the bed under 5 lbs. pressure. Water is discharged through the bed under normal operating heads in either direction, up or down, and the dirty or wash water is carried off by suitable connected drains. This system of cleaning the sponge clippings and coke has been found not only satisfactory, but very effective, and is carried on at a greatly reduced cost of operation and maintenance, only one attendant being required at all times.

The preliminary filters were placed in operation in October, 1907, two years before the final or slow sand filters. These filters are operated at the rate of about 50,000,000 gals. per acre per day. During 1908-09 the average removal of bacteria was 51.69 per cent. During the same period the average reduction of turbidity was 62.35 per cent. Thus it will be seen that the turbidity removal exceeded the bacterial. After the prefilters were in operation the typhoid fever death rate decreased 32.2 per cent.

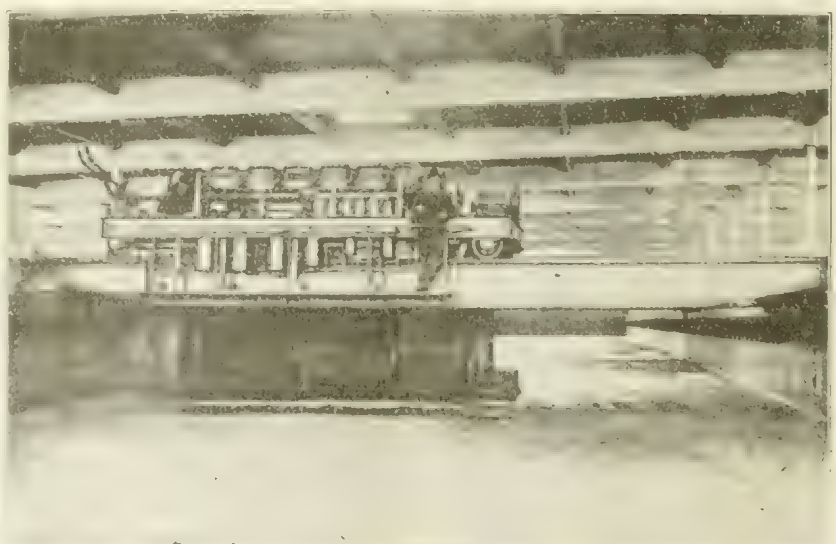
[illegible] FILTRATION PLANT.

Radical changes were made in the design of this plant, these changes involving a type of construction which is an innovation in this character of work, no similar plant having been constructed elsewhere. After much legal discussion together with financial difficulties, the contract for this work was finally executed on Dec. 9, 1908. Though the construction work was not entirely completed within contract time, it has been possible to operate the filter, and since Jan. 16, 1910, filtered water has been distributed throughout the city.

The plant consists of six covered sand filters, of one-third-acre each in area, a filtered water reservoir, a sand washing machine of the Blaisdell type, in addition to a control house, Venturi meters, valves and piping, and was designed for a daily capacity of 15,000,000 gals., although for present needs a 12,000,000-gal. capacity is ample.

The six filter units are superimposed upon a reservoir of 6,000,000 gals. capacity. At the extreme end of the works a dry gallery runs across the width of the six beds which contains the influent and effluent pipes and regulating devices, all visible and accessible.

One of the principal features in the design is that the filters and filtered water reservoir are combined in one structure two stories high. The reservoir or lower story has a groined arch roof supported on square piers, and this roof serves also as the floor of the filters, over which extends a roof of concrete slabs supported on I-beams. This form of construction effected a considerable saving in concrete and in excavation. If the two parts had been built separately a roof would have been necessary for each. There is also the obvious saving in land area. With such a grouping of the parts there is, of course, a far greater loss of head than in the case when the structures are built separately. Local conditions had a large influence in the selection of this type of structure, for the works are on high ground, and the loss of head between the filters and the reservoir is not a matter of any great consequence.



View of Blaisdell Filter Sand Washing Machine Installed at Wilmington, Del., Water Filtration Plant.

Another unusual feature is the shape of the beds, which are long and narrow; they measure 362 by 40 ft., and were built of this size in order to use the sand washing machine, later described.

The system of underdrains also is somewhat unusual, for the filter effluent collected by a tile pipe drain passes through holes in the floor into a discharge pipe hung up close to the roof of the filtered water reservoir.

This reservoir measures 364 by 252 ft. inside, and has a plain concrete floor of inverted groined arches. The piers, which are spaced on 14-ft. centers, are 24 ins. square and 6½ ft. high from floor to the springing line of the groined arch vaulting. The minimum thickness of floor and roof is 9 ins.

The filters which form the second story of the concrete structure are separated from one another by concrete walls, which are placed over every third pier across the width of the plant. These walls are 24 ins. thick and 9 ft. high, and along their side are bolted cast iron brackets, which support rails for carrying the trucks of the sand washing machine. In the top of each wall a channel is cast and will serve to convey the dirty wash water from the washer to the waste pipe connected to a sewer. The underdrainage system consists of one row of Brossman tiles, laid longitudinally down the center of each filter. These tile drains are 2 ft. long and have a section resembling a flat inverted V. Water may enter these through four openings—two on each side. They are bedded to the floor by a coat of grout, and are covered with a 15-in. layer of broken stone.

Under the tile pipe collectors at 14-ft. intervals along the longitudinal axis of each filter compartment holes are cast in the concrete floor and extend through to the reservoir below. The purpose of these holes is to afford an outlet for the filtered water, and at the same time serve as a means of supporting the effluent pipes, which are suspended under the roof of the filtered water reservoir, and extend the long way of each bed, running directly beneath the main tile collectors. The effluent pipes are of cast iron, increasing from 6 to 20 ins. in diameter. The effluent piping suspended at 14-ft. intervals for its entire length, connects with a main effluent pipe in the dry gallery at the end of the plant.

The lower courses of the filtering material are 14 ins. of crushed stone and graded gravel and on them rests a layer of sand 24 ins. thick. The sand was dredged from the Delaware River and had an effective size of 0.23 mm. and a uniformity coefficient of 1.83. The head of water on the sand is 4½ ft., and the beds are operated at a nominal rate of 6,000,000 gals. per acre daily for the present, although it is believed that a rate of 8,000,000 gals. can be used by taking precaution to sterilize the filtrate.

The roof of the filters is concrete with reinforced triangular wire mesh. It is laid in 3-in. slabs upon 8-in. I beams, and supported by I-beam columns resting on the walls dividing the filter beds. The roof has a slope from the center toward either side of ¼ in. in 1 ft.

At the southwest end of the plant, across which the dry gallery extends, the roof had to be raised a few feet in order to afford clearance for the sand-washing machine when it is transferred from one bed to another. A continuous bracket cast in each gallery wall carries the rails, along which a transfer carriage travels. This carriage carries a pair of rails spaced 38 ft. apart, the same gage as those of the filter beds, so that the machine may pass from the beds to the transfer carriage and then be carried by it transversely to any position in the gallery which may be desired.

At the northwest end of the pipe gallery there is a valve chamber to control the head of water on the beds. Water enters it by gravity through a 48-in. and 30-in. pipe from the sedimentation reservoir, which ends with a 30-in. hydraulic and hand-operated disc valve originally intended to be operated by a float in the chamber. By a simple and occasional adjustment of the hand wheel the water is maintained at a uniform level on the filters. From the valve chamber the water enters an influent main running the length of the gallery, reducing in size at each bed from 30 to 16 ins. in the total length of the gallery. This main has a 16-in. branch at each bed, and water from it discharges into a channel along the end of each filter and flows over the inner edge, which acts as a weir, and down through the sand. The filtered water, after passing through the pipes suspended from the roof of the filtered water reservoir, discharges through a Venturi meter on each bed into the main effluent collecting pipe and thence into the filtered water reservoir.

The final or slow sand filters were placed in operation January 16, 1910. The average number of bacteria per cubic centimeter in the raw, pre-filtered, settled and final effluent water for the years 1910 to 1913 were as follows: Raw, 27,263; pre-filtered, 16,025; settled, 15,875; final effluent, 464. The total percentage removal of bacteria by the pre-filters, settling basins and final filters was 98.3. The corresponding figure for B. coli was 88.52. The average turbidity in parts per million for the four years 1910 to 1913 was for the raw water, 67.2; pre-filtered, 37.0; settled, 27.8; final effluent, 0.8. The total turbidity removed was 98.82.

As a large portion of the turbidity is removed by the pre-filters, there is little work for the settling basin to do, the size of the particles passing these filters being very small, the amount that settles is very slight. There is no mechanical action to assist in carrying down the bacteria, hence the removal by the settling basin is small.

The color of the water has been determined only during 1912 and 1913. The average color for those two years, in parts per million, was. in the raw water, 23.0; pre-filtered, 22; settled, 22; final effluent, 8.5. The total color removed was 63.05. As no coagulant is used the removal of the color is not high or complete.

The final effluent during eight months of 1913 was treated with liquid chlorine. The liquid chlorine removed 83.12 per cent of the bacteria and 76.86 per cent of the B. coli. The amount of liquid chlorine averaged 1.56 lbs. per 1,000,000 gals.

During January, 1912, there was an epidemic of typhoid fever at Coatesville, Pa. (on the watershed). No notice of this condition was sent to the water department until the epidemic was well under way, and before an emergency hypochlorite plant could be installed and placed in operation the filters had failed to remove the bacteria and several cases of fever started in Wilmington.

When the condition of the watershed was learned an emergency hypochlorite plant was placed in operation, the hypochlorite being added to the water after it left the settling basin and before it was upon the final filters. This treatment killed the bacteria in the water and also those on the beds, and it was several months before the beds returned to normal working conditions. In the meantime a more permanent hypochlorite plant was placed in operation and the final effluent was treated.

The use of hypochlorite prevented a serious outbreak of typhoid fever in Wilmington. After this experience experiments were made with liquid chlorine, the results being so satisfactory that a permanent plant was installed.

MECHANICAL WASHING OF FILTER SAND.

The Blaisdell mechanical washing of filter sand is a method of recent introduction, and in consequence is not as generally known or understood as those systems and methods that have been heretofore applied for filter cleaning. Briefly, the washing machine may be described as a traveling crane spanning a filter unit and supporting a water-tight rectangular chamber containing the essential washing machinery, and further provided with means of lowering this chamber to the sand surface and traversing over the filter while the operation of washing the sand is proceeding. The washing chamber may be raised so as to clear the rim of the filter, and the machine removed to another unit by the transfer carriage.

As the width of each filter bed is 40 ft. the span of the main traveling crane supporting the washing chambers was made 38 ft. on rail centers. Forty-pound rails were used, supported on specially designed cast-iron brackets bolted through the division wall on 2½-ft. centers.

In the design of the machine the watertight washing chamber was made 20 ft. long by 4 ft. wide by 5 ft. 6 ins. deep. Approximately 2 ft. above the sand level or bottom of this chamber there is a plate or diaphragm subdividing the washing chamber into two compartments, the lower known as the suction chamber, from which the dirty wash water is withdrawn, contains five horizontal stirrer wheels supported on vertical shafts; the upper or dry chamber contains the driving mechanism for the stirrer wheels and also the pressure and suction pumps.

As the width of the bed is 40 ft. and the area washed in the forward movement of the machine is but 20 ft., the entire washing chamber is supported upon crane beams mounted upon wheels, for the purpose of moving it laterally across the width of the bed, on rails supported on the girders of the main traveling crane. This permits of completing the washing of the entire bed in one round trip of the machine—i. e., by moving forward over the entire length in one position a width of 20 ft. is cleaned, and by transferring the chamber to the opposite end of the main crane, which transfer is made without stopping washing process, the remaining area is cleaned upon the return trip, leaving the entire machine ready to be transferred to any bed without undue loss of time.

The stirrer wheels in the lower or suction chambers are supported by vertical shafts in a horizontal position 1 in. above the sand surface. Numerous teeth project from the lower face of the stirrer wheel rim and extend into the sand below the level of the chamber to any desired depth. Each vertical shaft passes through a stuffing box in the top of the chamber to an upbearing in the frame of the machine, and is revolved by a motor-actuated worm gearing at the upper end. Midway of the shaft there is a T-shaped sleeve, with stuffing boxes at the upper and lower ends, which serves as a water connection. From this sleeve downward each shaft and wheel, including spokes and rim, are hollow. The teeth are hollow and perforated for the purpose of creating a water jet action from the supply delivered by pressure pump. Water supply for the pressure pump is taken from the raw water of the filter, while at the same time a suction pump connected to the top chamber withdraws not only all the water the pressure pump supplies through the perforated teeth, but also an additional quantity made up from the filtered water stored in the filter sand. In operation the teeth scour the sand; the wash, or pressure water, by the jet action prevailing, drives the dirt up into the suction chamber; the clear water stored in the filter bed is drawn into the washing zone by the excess suction over pressure supply, and the wash water is withdrawn and pumped from the chamber before the machine passes a given point. The chamber is sealed to the sand surface by wide hinged shoes extending in advance and to the rear of the front and back plates, while the side plates form a cutting edge extending down into the filter sand.

The maximum speed of operation is governed by the rapidity with which the pressure pump water may be displaced by the filter bed supply, and as a result the speed of advance varies with the length of teeth used —or, in other words, the depth of the washing.

Due to the fact that there is less dirt at the lower depths of the filter than near the surface, it is necessary when washing to a considerable depth—such as 24 ins.—that teeth of proportionate length be fixed in the stirrer wheel, and the wash water is therefore introduced not only at the lowest point, but at several points above. This pressure water quickly rises through the disturbed sand zone and is displaced by the inrush of the clear water, the upward current of which occurs well toward the center of disturbance created by the teeth and covered by suction chamber. The sand grains are dislodged and forced apart by the passage of the teeth, and as they return to the cavity in the wake of the teeth the strong upward current of wash water creates a temporary suspension and churning action within the area of the suction chamber. During this time the dirt and light particles are swept to the surface and withdrawn with the wash water. The subsidence of the suspended sand takes place following the passage of this violent upcast and occurs during the more gentle upward passage of the clear water, so that when the sand comes to rest it is uniformly water packed and free from all air.

As each stirrer wheel revolves five times per minute and mounts 32 teeth, it follows that all sand grains are disturbed several times, both by direct contact with the teeth as well as by the working of adjacent sand and the horizontal water jets from the teeth.

The wash water may be controlled to enable hydraulic sizing if so desired, working all very fine sand to the surface, and if sand too fine for effective size is present it is possible to remove it from the filter by increasing the duty of the pressure and suction pumps to secure the desired upward velocity to hold the sand in suspension. At all times the excess suction demand is satisfied by the supply of clear water stored in the filter sand.

The dirty wash water from the machine is discharged to a gutter formed in the concrete party wall between the filter sand units, from which there are sewer connections at either end. This gutter is about 18 ins. deep, 10 ins. wide at the top and 6 ins. at the bottom, with a slight grade either way from the center of the beds to the drains at the ends, but the slope is not enough to allow any sand raised to proceed to the sewer system. Sand that collects in the gutter is removed with a spade and returned to the filter or taken away, as desired.

The party walls serve to support the brackets for the rails on which the washing machine travels, and also the steel columns supporting the roof over the plant. The wash water gutter is placed off the center line of the wall to enable the discharge spout from the machine to pass the columns.

All of the several operations of the mechanical features of the machine are actuated by independent motors controlled from the platform by one operator. There are six motors mounted on the machine with variable-speed controllers for operating, as follows:

	Capacity, HP.	Minimum HP.	Maximum HP.
Main traveling crane......	25.0	5.0	15.0
Washing chamber transfer	7.5	1.5	2.5
Washing chamber hoist	[illegible]	[illegible]	[illegible]
Stirrer wheel drive	[illegible]	[illegible]	20.0
Pressure pump drive	[illegible]	12.0	15.0
Suction pump drive	[illegible]	[illegible]	5.0
	90.0	32.0	63.0

Actual experience has demonstrated that but 54 H. P. is required for the worst conditions of operation, and 25 H. P. with short teeth and normal conditions. The quantity of current consumed in washing each bed of one-third of an acre in area has ranged from 80 to 175 K.W.-hrs., and averages throughout the year approximately 120 K.W.-hrs. The current consumed and power required are least when the shortest teeth are being used, and reaches a maximum with the longest teeth during the winter months, when the oil-gear boxes tend to congeal and the operation of the machine is hampered by ice on the filter beds.

With a five-unit machine cleaning a strip 20 ft. wide, one operator washes one-third of an acre in four hours, including transfer, and the bed is usually back in service within five hours from the time of closing valves for washing process. This question of time element naturally affects the reserve area of plant that must otherwise be constructed in order to maintain normal rates of filtration throughout. The experience of the department has been that the time out of service by this method per acre area ranges from 7.5 per cent to 10 per cent, while with the old method of hand scraping, ejecting and restoring, from 15 per cent to 25 per cent of time is lost. It is apparent, therefore, that the construction cost, either for reserve area or plant capacity, for low rates of filtration is materially reduced by this improved mechanical method of filter cleaning.

The method also had a material effect upon the cost of operating sand filters, effecting a reduction in our plant of approximately 50 per cent over the cost of operation of plants of the same capacity elsewhere, notably Philadelphia.

It was confidently expected when the plant was built that the operating cost would not be greater than $1 per 1,000,000 gals. exclusive of laboratory charges. We have not, however, been able to reach this low figure, the cost in the year 1910-11 being $1.08. As the operations of the plant require intelligent superintendence at all times, and as skilled mechanics must be employed continuously for operating the washing machine, notwithstanding that it is idle approximately 75 per cent of the available working time, it is evident that with a larger daily delivery in million gallons the above unit cost, which involves a considerable overhead or fixed expense, could be materially decreased.

When the turbidity of the settled or applied water is above 50 p. p. m. it is impossible to get a filtrate that is entirely satisfactory.

At one time during cold weather it was necessary to use dynamite to dislodge the ice from the filters.

The most serious trouble is from "air-bound" beds during the winter months, when the water is saturated with oxygen.

ROADS AND STREETS

Details of a Reinforced Concrete Pavement in Morgan Park, Ill.

(Staff Article.)

The details of construction of a reinforced concrete pavement built in Morgan Park, a suburb of Chicago, the method of constructing a difficult street intersection, and the organization and performance of the concrete crew used by the contractor in laying the surface, are described in this article.

The street on which the pavement was laid connects Tasso Place and Belmont Ave. Funds were provided under the special assessment law of the city of Chicago. While the general topography of the country is rolling the grading on this street was very light, although no previous improvement of the street had been made. One short fill was made; the remainder of the grading consisting of leveling and trimming. The soil, for the most part, is of a clayey nature.

Concrete curbs were set 20 ft. apart in the middle of the right-of-way, liberal park space being allowed on either side between the curb and property line. Where needed 6-in. drain tile was laid under the curb. Only a small portion of the curb, however, was underdrained.

The cross section, Fig. 1, shows clearly the various details of the surface. On account of the uncertain character of the foundations it was decided both to reinforce the concrete surface and to provide a cinder subbase. The surface was cut at 25-ft. intervals to provide for temperature changes. Additional expansion joints were provided along the curbs. In effect the pavement is divided into separate slabs 25 ft. long and 20 ft. wide, free to expand in any direction. Metal bound felt expansion plates were provided for transverse joints. Longitudinal joints along the curbs were of plain ½-in. felt. All joints used were manufactured by the Philip Carey Co. of Cincinnati, Ohio.

A total crown of 2½ ins. was allowed, which is a rate of ¼ in. to the foot for a half-width of 10 ft. The curb exposure was 4 ins., the center of the pavement being 1½ ins. below the top of the curbs. By this means ample waterway was provided to care for excess storm water. The average grade on which the pavement was laid was about 4½ per cent.

The reinforcement consisted of triangle mesh wire in 50-in. and 54-in. widths manufactured by the American Steel and Wire Company, and was designed to be so placed as to allow an alternate 2-in. lap. It was designed to follow the center line of the pavement cross-section. Concrete was of an unusually rich mixture, a 1:2:3 mix being specified. Aggregate consisted of washed flint gravel passing a 1½-in. screen and retained on a ¼-in. screen and clean Lake Shore torpedo sand.

An interesting design feature was the treatment of the intersection, Fig. 2, which was a junction of two descending grades. It will be noted that the distance between transverse expansion joints at this point is quite large, the length of slab being 59 ft. 9 ins. The joint on the intersecting street is broken, as shown in the figure. The distance from the curb to the point at which the break occurs is 20 ft.—the width of the street. The joint is at right angles to the intersecting curb line, and the break occurs at the intersection of the curb lines extended. The distance from the point at which the break is made to the curb is 9 ft. The crown at the intersection was increased to 4 ins. and is placed on the up-grade side of the point of intersection of the street center lines. It will be an interesting study to watch the action of this joint and note the effect of time and temperature changes.

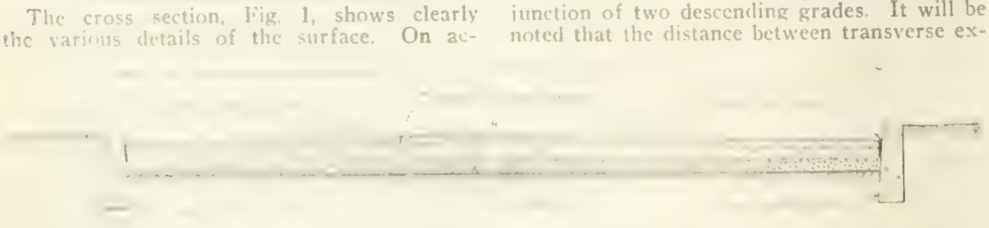


Fig. 1. Cross Section of Reinforced Concrete Pavement in Morgan Park, Ill.

Fig. 2. Method of Making Joints at an Oblique Intersection.

DETAILS OF CONSTRUCTION.

The curbs were placed throughout before beginning to lay surfacing, and sufficient time elapsed to allow the concrete to set thoroughly, so that the curbs were used as rails for the striking template. Fine grading was completed and allowed to settle. The cinder base was placed a convenient distance ahead.

Water was supplied through ¾-in. hose from the municipal supply, taps being made at convenient intervals. Some trouble was experienced from lack of water pressure.

Fig. 3. Method of Placing Reinforcement and Finishing Surface.

The ordinary procedure in mixing and laying was as follows: Aggregate and Universal Portland cement were piled at points convenient to a T. L. Smith 1 cu. yd. capacity mixer. The concrete gang consisted of 16 men and a foreman and was organized as follows:

Item.	Cost per 8-hour day.
Wheeling to mixer, 4 men at $3.20	$12.80
Wheeling from mixer, 3 men at $3.20	9.60
Loading mixer, 1 man at $4.50	4.50
Spreading concrete, 2 men at $4.00 and $3.60	7.60
Finishing, 1 man at $5.20	5.20
Mixer engineer, at $4.30	4.30
Setting joints and reinforcing and smoothing grade, 4 men at $1.60	6.40
Foreman (approximately)	7.00
Total daily labor cost of laying surface	$57.40

Average performance, six 25-ft. sections (150 lin. ft.), or 333 sq. yds. per 8-hour day.

Note.—The cost of superintendence charged at $7.00 per day should properly be prorated over the cost of hauling, grading, etc., in addition to mixing and placing concrete. The foreman is employed on a monthly salary plus a bonus basis.

A thin layer of concrete about 3 ins. thick was laid on the cinder base for a short distance ahead of the second layer. On this the reinforcing was placed and the second layer run and finished. Owing to the difference in thickness at the center and sides it was found difficult to place the reinforcing in the center of the slab, and on most of the work it was dropped below the center. It was also found to be much easier to handle the reinforcing by using this method of placing. The striking template used was made by P. Roughen, Fond du Lac, Wis., and was found very satisfactory, reducing the amount of finishing required and aiding in spreading. A braced finishing board straddled the pavement behind the template and from this the hand finisher worked. Figure 3 illustrates the method of placing reinforcing and finishing. Each block was placed continuously, no stop being made for any purpose until the joint was reached.

Fig. 4. Method of Setting Transverse Expansion Joint.

The plate joint used and the method of setting are shown in Fig. 4. The joint was set up on trestles and placed in the pavement at points marked on the curb. It was rigid throughout its length and supported at the curb by an arm which was removed after placing. The joints were placed rapidly, and no difficulty experienced either in handling them, or during the placing of the concrete.

The finished surface was sprinkled daily during construction and for at least two weeks after completion. At the present time the surface is in good condition.

PERSONNEL.

The contract price for 3,300 sq. yds. of pavement complete was $1.50 per square yard. About 1,300 cu. yds. of grading were taken at 33 cents per cubic yard. The contractors were the Wm. Ahlborn Construction Co., Hammond, Ind.; S. J. McDowell, local superintendent. Vertus B. Roberts, Blue Island, Ill., was engineer; W. J. Hill inspecting the work for the City of Chicago.

The Construction and Use of the Wooden Road Drag.

(Staff Article.)

The road drag is a device for maintaining those types of road surfaces which when wet soften and become rutted under traffic but upon drying out become firm. It may also be effectively used during construction for the purpose of smoothing out surfaces composed of earthy materials. Many different types of drags; built of both wood and steel are in use. However, the material of which constructed appears to have but little effect upon the quality of work accomplished. A possible exception is in dragging sticky, prairie soils where weight and both a cutting and smearing action are desirable. In this case an adjustable steel drag is to be preferred.

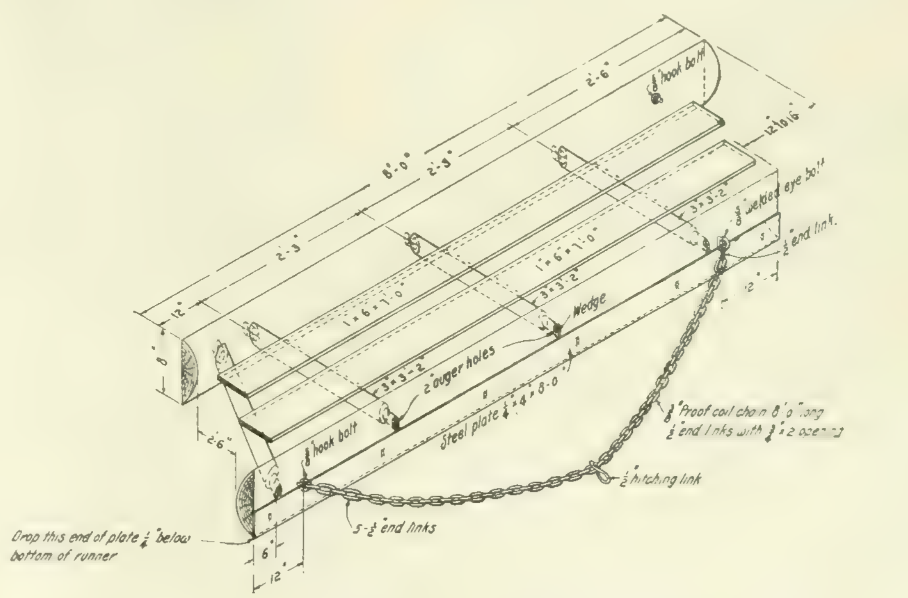


Fig. 1. Improved Type of Split Road Drag.

In a recent bulletin of the United States Office of Public Roads a modified form of wooden drag is described and instructions for its use given. The following article has been prepared from information contained in this bulletin and data collected from other sources.

METHOD OF CONSTRUCTION.

Log Drag.—A drag constructed of the halves of a split log is illustrated in Fig. 1. The log selected is preferably of a hard, tough wood to resist the grinding wear incident to dragging, and well seasoned to secure lightness. The diameter of the log may vary from 7 to 10 ins., depending largely upon the type of soil on which the drag is to be used. For a heavy, sticky soil in which ruts are apt to be deep a good thickness, or depth of face, and extra weight is desirable. On the other hand, in a sand loam soil or for use in the construction of sand-clay roads where a comparatively light weight drag is used, less thickness is satisfactory. It is not necessary that the log be the same thickness throughout its length. Some road builders even prefer to have one end thicker than the other, claiming that by an alternate arrangement of large ends on the front and rear logs better control is secured in operation.

A convenient method of constructing the drag illustrated in Fig. 1, the type recommended by the U. S. Office of Public Roads, is as follows: The log is sawed or split into halves and the better half selected as the front blade. Two-inch auger holes are bored

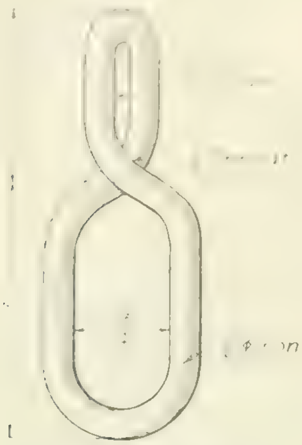


Fig. 2. Grab Link for Changing Hitch on Draw Chain.

to receive the cross-braces or rungs, care being used in boring to see that all the holes are plumb from one position of the log. The dimensions of the various parts are noted fully in Fig. 1. To secure better control and to facilitate emptying the drag while in operation the two blades are so arranged that when the tool is drawn at an angle the ends of the blades are in a line parallel to the center line of the roadway.

In framing a level surface is selected and the logs forming the two blades driven together gradually upon the cross-braces; the rear log having the round side for a front edge. In this way some rigidity is secured before wedging the ends. Wedges are driven across the grain of the timber. Two 1-in. by 8-in. boards laid across the top provide a platform on which the driver may stand in operating the drag.

The steel plate attached to the front log, Fig. 1, should always be provided if the drag is expected to accomplish much work and wear well. This plate should be drawn to a sharp edge on one side and tempered. An old road grader blade makes a very satisfactory cutting edge on account of the superior quality of the steel. Heads of bolts fastening the plate to the log should be countersunk.

Fig. 4. Split Log Drag King Type. Note the Hitching Link.

In case a road grader blade is used the spacing of bolt holes should be noted.

For hitching to the drag it is well to provide a stout chain. Five or more long links should also be provided at one end for convenience in changing the angle of draft. A length of about 8 ft. is usually necessary. Eye and hook bolts for hitching the ends of the chain are to be preferred both from the standpoint of convenience and security from vandals and durability of the drag.

A convenient type of hitching link is illustrated in Fig. 2. This type of grab link, a shaped ring one side of which is given a quarter turn, is frequently seen on timber wagons in logging camps. By its use a convenient method of changing the angle of draft is provided without changing the length of hitch, the full strength of the chain is obtained and a ring provided for either a hook or clevis. The ordinary clevis hitch without some form of ring is cumbersome. The weight of an oak drag complete (not including the draw chain) will vary from 500 to 700 lbs., depending on the seasoning of the wood. A pine drag is about 150 lbs. lighter.

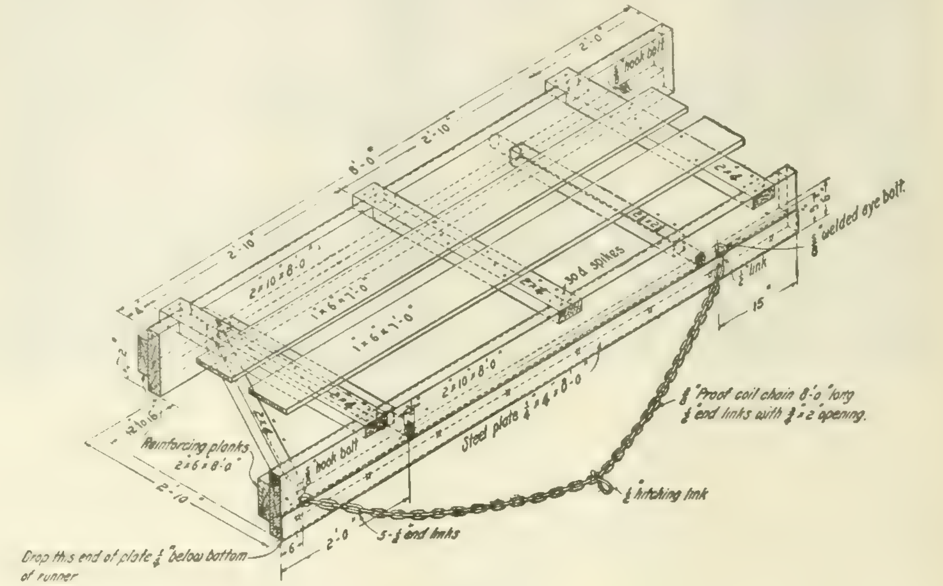


Fig. 3. Improved Type of Plank Road Drag.

The following is a list of the material needed for the log drag illustrated in Fig. 1:

No.	Item.	Size.
1	steel plate with holes as shown..	¼"x4"x8'-0"
2	hook bolts with nuts and washers.....	⅝"x5"
1	welded eye bolt with nuts and washers.	⅝"x5"
6	bolts with nuts and washers (countersunk)	⅜"x5"
¼	lb. wire nails	8d
1	hitching link	½"
1	proof coil chain....⅜" links, 6 ½" links,	8'-0"
	Lumber:—	
2	8"x8'-0" half round logs.	
3	3"x3'-2" cross pieces.	
1	2½"x3'-6" diagonal brace.	
2	1"x6"x7'-0" boards.	

ity without cumbersome cross-bracing. Otherwise, a 2-in. thickness of plank would be satisfactory. The top braces are quickly framed and add materially to the strength of the drag. Frequently two ⅝-in. rods (not shown in Fig. 3) placed within a few inches of each cross-brace, are used to bind together the two planks more securely. Other details are similar to those for the log drag, shown in Fig. 1. The weight of the plank drag illustrated in Fig. 3 (not including the draw chain) will vary from 225 lbs. to 350 lbs., depending on the seasoning of the wood, when constructed of pine, and 330 lbs. to 450 lbs. when constructed of oak.

The following is a list of the material needed for the plank drag illustrated in Fig. 3:

No.	Item.	Size.
1	steel plate with holes as shown..	¼"x4"x8'-0"
8	bolts with nuts and washers countersunk	⅜"x3"
1	welded eye bolt with nuts and washers.	⅝"x5"
2	hook bolts with nuts and washers......	⅝"x5"
⅓	lb. wire nails........	8d
3	lbs. wire spikes........	30d
1	hitching link	½"
1	proof coil chain...⅜" links, 6 ½" links,	8'-0"
	Lumber:—	
3	2"x4"x3'-2".	
2	2½"x2½"x3'-2".	
1	2"x4"x3'-0".	
2	1"x6"x7'-0".	
2	2"x6"x8'-0".	
[illegible]	2"x10"x8'-0".	

Fig. 5. Maintaining a Newly Surfaced Sand-Clay Road.

Plank Drag.—In Fig 3 is illustrated essentially the same type of drag previously described, except that sawed lumber is used for the blades instead of the halves of a log. Either 2-in. or 2½-in. lumber, reinforced by a narrow strip of 2-in. x 4-in. lumber may be used. A 4-in. depth of bearing for the cross braces is required to secure the desired rigidity without cumbersome cross-bracing.

Other Wooden Drags.—Various forms of wooden drags are in use designed to fit some special soil condition or local condition of labor or materials. Among these are the float, made of 2-in. by 10-in. planks framed together like clap-board house siding, which is used in very wet prairie soils; the sloping rear blade drag, in which the rear plank is tilted forward to increase the smearing action of that blade; and the hone, in which the cross-braces are set at 30° angles forming cutting blades (the long sides merely acting as runners), the hitch being made from the runner ends.

METHOD OF USE.

Smoothing an earth road surface with a drag while in a damp and comparatively plastic state accomplishes the following results: (1) By the intermingling and kneading together of porous sections of soil which drain well with more impervious sections on which water stands in puddles, a more uniform density of road surface is obtained after repeated dragging; (2) an immediate benefit is derived from the filling in of ruts, and low places that hold water, permitting the surface to dry out and pack down uniformly; and (3), the crown of the road is preserved. It is readily seen that dragging a sand road accomplishes nothing more than the filling in of ruts. This work is accomplished more economically —should it be desirable—with a 2-horse road grader.

In operation the drag is ordinarily drawn at an angle of 15° to 30° with the center line of the road, the nose being on the ditch side of the road. Further than this, instructions as to use are valueless, since the art of successfully operating a road drag may be learned only by experience. Suffice to say the operator will find that by alternately riding and walking, by shifting his standing position on the drag, by varying the length and angle of hitch and the use of a little common sense, he will be able to change a badly rutted surface into a smooth road—which is the end sought.

Practice varies in regard to the amount of dragging after each rain. Ordinarily, a trip up one side of a section of road and down the other after each heavy rain, or continued rainy period, is the method most commonly used. More dragging than this is usually uneconomical, except in special cases where other tools are not at hand.

The time at which the drag is most effective is difficult to determine in localities where many different soil types exist. Each soil type has a definite state of plasticity at which it may be handled to best advantage. For this reason it is quite desirable that maintenance dragging districts be either small enough to contain all one type of soil, or if large, soil types should vary in time of dragging requirements. The time to drag is best determined by close observation of each section.

Maintenance dragging systems have been instituted in many states, notably Iowa, Illinois and Kansas. ENGINEERING AND CONTRACTING, in Vol. XLI, p. 608, and Vol. XLII, p. 107, contains articles on methods and cost of operating dragging systems. In Bennington Co., Vt., the cost of dragging, as determined by the U. S. Office of Public Roads, varied directly with the price of labor and teams. When the road was well shaped, requiring only one trip in each direction, an average rate of complete dragging was one mile per hour. An experimental section of road 8 miles long, near Alexandria, Va., dragged for 3 years (24 draggings during first year) cost at the rate of $1.25 for each dragging; a dragging consisting of three round trips. The foregoing cost is below the average. In Iowa the average price of many dragging contracts ranges from forty to fifty cents per mile for each dragging. The length of sections dragged by one man varies from 3 to 6 miles.

It should be borne in mind that a road drag is primarily a maintenance tool. If ruts are deep and the road badly out of shape it is much more economical to use a road grader to put it in good condition before using the drag. The small 2-horse grader is convenient for this purpose. A road grader and drag are used for purposes that do not materially overlap; and when properly used together, earth road surfaces may be materially improved in an economic manner.

The Relation of Farm Produce Hauling to Permanent Road Improvements.

The economic factors entering into the character and extent of permanent road improvement are varied and important. Some of these factors are as follows: The relation of the extent to which the area surrounding a city contributes to its support and the bearing of improved roads upon the radius of a circle including this area, the relation of the type and extent of traffic on a road before improvement to the type and extent of traffic after improvement and the factors that contribute to a change of type, if any; the extent to which through traffic should control the type of improvement, the just apportioning of the cost of improvement, and the type warranted by economic considerations; the changes in the character of farm crops raised and in labor conditions induced by road improvement, and the relation of these factors to the justifiable expenditure of money desirable for present and future welfare.

A recent report of the Public Roads Commission of Ontario, W. A. McLean, chief engineer, considers some of the foregoing questions, which questions are fundamental to the consideration of the economic improvement of highways. Portions of this report are given here.

GENERAL CONSIDERATIONS.

Broadly, it is the general experience of all countries that roads for local traffic only and lightly traveled can be managed and financed by the local community alone; the function of a central government being, by well-advised legislation, to provide effective means of local co-operation, with government assistance only as a measure for stimulating and focusing local effort in the early stages of a country's development, and as an educative step.

On the other hand, roads which carry traffic from a considerable area (on which unites the traffic of numerous local roads, serving a township or group of townships) enter a field of construction which demands large outlay, scope of organization, and continuity of effort beyond the ability of local or municipal organization alone to carry out. This has been the experience of every country which has attained a system of good roads; and to the work of building and maintaining roads of more than local traffic, the control and financial co-operation of the central government is in all cases being extended. Great Britain, France, Germany, the United States owe their good roads, where they exist or are being built, to the active co-operation of their central governments. Equitable distribution of cost, an adequate scheme of finance, representation in expenditure, technical efficiency and business management all demand the grouping of roads according to a suitable classification. For this reason the roads are broadly divided into township roads, or those of local traffic; and county roads, those which carry united traffic, and to which the co-operation of the provincial government should be more permanently extended.

The maintenance of the well-built roads in particular demands that they should be grouped under counties, rather than left with the local roads under township management. Otherwise expensive roads, after being built, are apt to be starved and allowed to fall into neglect, in order that the local roads may be improved. When a main road has been built, the tendency is for residents on other roads to say to their township council: "Don't spend another cent on that good road until my road is in equally good condition." Many old toll roads, formerly good, have been permitted to degenerate after passing to township control from this cause.

CITIES AND SUBURBAN ROADS.

There exists about the city a belt of rural territory which is knit to it in the closest fashion. Much of the city's food is grown in this belt; more would be if the means of communication were better. Sundry industries, due to the presence of the city, are prosecuted in this area. The residents for some miles out are valuable customers of the city's shops. In every way the city stands to gain by the equipping of this belt with a system of roads able to carry a heavy traffic with speed and economy. The speed of the motor 'bus and motor truck would extend the city's influence; that is, the area from which it could draw food and direct trade. Opportunities would be afforded for a specially beneficial development, the rapid moving of workers out into the countryside after their daily task is over. It is understood that in Belgium one-third of the industrial workers live outside of the towns, cultivating small holdings of land, under conditions of health, which surpass those of residence in the crowded streets. From the standpoint of the city's food supply alone the improvement of the roads is of great importance to the town-dwellers.

Economically speaking, distances are measured by time, and if men trespass too much on the early morning hours in order to reach distant markets, nature makes her claim on them later on. If the constant, regular supply to city markets is limited to points, say, two hours therefrom, it would mean leaving the farm at 6 a. m. in order to be on the market at 8 a. m. It is easy to realize, therefore, that by cheap motors and good roads the supply area can be greatly enlarged, as compared with the present districts, in which supplies are sent into town by horse-drawn vehicles on indifferent roads. Further, the widening of the belt means enhancing the profits per acre, to the advantage of the farmer.

Again, the countryside has suffered for several decades from certain inevitable developments. Forty years ago a considerable amount of industrial work was carried on in nearly every small town, in nearly every village, and indeed in many rural communities too small to aspire to the name and style of village. This caused a wholesome diversion of industry, increased the interest of country life and was in most respects a beneficial social influence. The march of progress has swept that state of things away. The tendency of the age is towards centralization. Those small industries, which mean much to the small towns, have been absorbed into those operating in larger centers. The countryside must specialize in farming. Why, then, should cities, to a certain extent, built up by rural districts, which have lost taxable property to those cities, not be prepared to contribute to the road system?

In short, if the roads of this suburban belt should be brought up to a high standard, suited for heavy traffic, some of it carried on at considerable speed, and the principal beneficiary would be the city; on whom should the burden fall? The county and the township already spend as much upon these highways as strictly county and township purposes warrant; the wear and tear is inflicted by city people and by others whose residence in the suburban area is due to the propinquity of the city. These and other considerations have influenced the classification shown.

Along the interurban roads many persons will pass who do not live in the municipalities in which they are situated. This is perfectly natural; from time immemorial, the highway has been for the use of the traveler, regardless of his residence. It is necessary, of course, to see that the burdens of constructing and maintaining such a road are equitably adjusted, so as not to impose an undue proportion of them on the people of the locality. Indeed, if measures of this sort are not taken, the situation will work itself out, and disadvantageously to all concerned; for the motorists will search out and appropriate to their use the best stretches, and there will be motor routes which at once will give dissatisfaction to the motorists and inflict a sense of injury upon the farmers and tax payers along them.

SUGGESTED CLASSIFICATION OF PERMANENT ROADS.

The important roads of each county carry about 80 per cent of the traffic, and amount to about 15 per cent of the total mileage of all roads in the county.

Thus important roads called county roads have three functions to perform, and are consequently divided into three grades: Suburban shown on diagram, Fig. 1, by solid black line; interurban shown by double line; and rural shown by broken line.

All three classes are used by the farmers of the county, therefore all have a county function to perform and all should receive support from the county. The suburban road in addition has certain city functions, and the interurban has its additional urban functions outside of the suburban area as well as certain provincial, and in some cases, national services to render and is largely used by motors. The division for aid suggested is shown in Table I.

TABLE I.—TABLE SHOWING PROPOSED APPORTIONMENT OF THE EXPENSE OF CONSTRUCTING AND MAINTAINING ONTARIO ROADS

Classification.	Construction cost, part paid by County. Per cent.	City Per cent.	Province Per cent.	Maintenance cost, part paid by— County. Per cent.	City. Per cent.	Province. Per cent.
Rural	60	..	40	60	..	40
Suburban	30	20	40	33⅓	23⅓	33⅓
Interurban	33⅓	..	66⅔	60	..	40

*If cost exceeds $10,000 per mile, excess to be met by local improvement tax.

†To cost not exceeding $12,000 per mile, revenue from motor fees to supply one-half of provincial aid.

THE COST OF DISTRIBUTING FOOD PRODUCTS.

Highly productive farms can never be made profitable adjoining poor roads, and the first agency towards increasing the prosperity of the farmer is to create the cheapest outlet for his products to the market. Canadian statistics for 1912 indicate that one dollar will carry a ton of the average freight by railway 130 miles, and on water, 515 miles. The former is the average of high grade and low grade traffic, car lot and less than car lot, long haul and short haul, while the latter is relatively a limited number of commodities, moving in large bulk, on long water hauls. As to the cost of haulage per ton by horse-drawn vehicles, there is some general understanding that on very ordinary roads, four miles is the limit for one dollar. It can hardly be disputed that a team will haul three tons easier on a hard smooth road, than one ton on a bad road.

Motor haulage appears to be the solution of the problem. On good roads the motor will make 20 miles with a reasonable load as quickly as a team can cover 6 miles on good roads, or say 3 miles on poor ones. The "home market" of the farmer thereby may become enlarged to an extent he hardly realizes today. Give him a vehicle which can transport a load cheaply for 20 or 30 miles in the time now taken for 10, and you give him a choice of urban centers in which to do his marketing. Motors consequently will be a great factor in the development of the country, provided the roads are made suitable for them. The area of production of food stuffs will be increased, and the profits per acre increased to the farmers. In a very interesting investigation carried on by Clyde Lyndon King, Ph.D., for the Mayor of Philadelphia, in 1912, it was shown that the spread between what the farmer received for certain foodstuffs, and what the consumer paid, varied from 67 per cent to 265 per cent. These investigations of course were only carried on locally.

In dealing with these figures, in an article on the cost of distributing food products which appeared in the Annals of the American Academy of Political and Social Science, Dr. King, referring to what these costs mean to the consumer and farmer, in part stated as follows:

It is difficult for the imagination to grasp just what these costs of distributing farm produce mean in lower prices to farmers and higher prices to consumers. The consumers of New York City pay annually around $645,000,000 for food. This food costs at the terminal $350,000,000. That is to say, the people of New York City are paying over $150,000,000 each year to have their foodstuffs taken from the terminal to their kitchens. At a cost of 14 cents per meal per person for all classes in Philadelphia, high and low, rich and poor, Philadelphia citizens are spending $225,000,000 every year for food. Of this amount they pay something less than $75,000,000 each year in cartage and delivery costs and in retailers' profits.

Of the $140,000,000 paid annually by the people of New York City for milk, eggs, onions and potatoes, less than $50,000,000 was received by the men who raised these crops. For certain produce for which the eastern farmer last summer received $1, the Philadelphia consumer paid $2.35.

Is it not needless to point out other results of this method of distributing food products? Is it not clear that the interests of every farmer and every consumer point to that necessity for developing a cheaper method of food distribution, whereby at least much of the handling and the profits of a few of the middlemen may be eliminated? All are interested in cheaper costs of food distribution. The farmer is, of course, because it means higher prices. The consumer is because that is his only hope for lower prices. But so is the city; and the labor employer.

It is mainly in the development of direct shipments to relatively nearby markets that the farmer's returns can be increased and consumers' prices lowered. It is, however, not fair to charge all of the great spread, including transportation charges, as between producer and consumer to the middleman, as many are now doing when looking for the causes of the present cost of living. The residents of the cities, especially the property owners, are in a measure responsible for some of that spread, because they having joined the real estate man in taking $10 bills in the shape of city property, have committed forgery by coolly raising the bills to $100. That at once creates new conditions for all the city dwellers, owing to a return being immediately demanded on that watered stock.

The cartage of food products by the individual farmer from the farm to some market as is now the custom will always be the most expensive method. The time is coming when the motor will play an important part in co-operative marketing by farmers. That, however, must wait until proper road facilities exist for the motor. There is no class of the people who need co-operation more than our farmers. It will not alone benefit their pockets, but it will create new social conditions, making farm life more agreeable than at present. Important strides have been made in that respect in Europe, and while this subject may be regarded as having no place in a report on roads, still improved highways and good farming must go hand in hand. The doing of things, whatever they may be in this age of competition, must be reduced as nearly as possible to a science. At present it is practically impossible to get from the farmer clear and concise figures of haulage cost. This is not offered in any sense of criticism. If such figures were available it would be possible to compile most interesting statements, showing the annual loss or gain on roads of various grades, to each township, which could easily

Fig. 1. Chart Showing a Suggested Method of Classification for Highways.

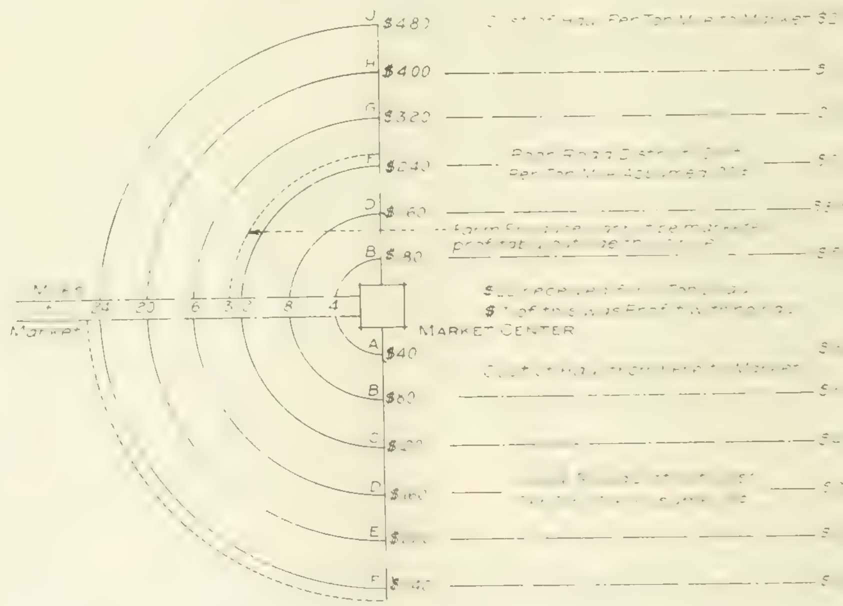


Fig. 2. Diagram Showing Relative Profitable Hauling Radius and Variation in Profit for a Given Load.

be determined owing to the total agricultural production being fairly well known.

SUPPORTING AREAS OF ONTARIO CITIES.

Cities are benefited by general rural development as brought about by better roads. Urban centers with good roads are especially benefited by the main roads in their immediate vicinity. It may in a general way be assumed that each city has a special interest in an area immediately surrounding it, sufficient to provide a food supply for the city, and the population within such area.

It is not advanced that the Suburban Area referred to in the classification of roads should necessarily coincide with the area of food supply or Supporting Area. Such an area is probably too great to meet conditions of actual practice; but it is of use as representing an extreme standard of measurement.

Before further explanation is entered upon, attention should be called to one or two preliminary points. It is well known that cities are not, even in the matter of home-grown products, supported altogether by their immediate neighborhoods. Thus, for example, while potatoes are an excellent crop in Ontario, New Brunswick potatoes are quoted throughout the year on local markets. The reason for this, of course, is that in point of time and cost, the New Brunswick potato-grower is actually nearer the Ontario markets than are the Ontario growers themselves. Under present conditions farmers within comparatively short distances of large consuming centers, are unable to take up diversified farming to the extent that they would, if they could market their produce readily as it ripened. Thus farms in close proximity to centers of population may be found devoted to grain crops only, because the farmer cannot afford to risk the growing of crops requiring immediate marketing, or because he finds that the time consumed on the road to market and back makes the cost of production on these classes of foods relatively higher than it is on other crops which keep longer and can be marketed when his time is worth less. Thus, while it is true that our cities do not, at present, draw all their food from immediate territory, it would seem that the most potent influence, in preventing such an arrangement, has been the heretofore inadequate means of local transportation in marketing.

Improvement in the facilities would, however, induce the abandoning by nearby farmers of low priced crops, which have heretofore carried the bonus of cheap marketing, for high priced crops upon which marketing charges will decrease as the farmer is brought closer to his market.

Then, again, it should be pointed out that some districts are specially adapted to the production of certain products, such as fruits, and they should therefore be properly expected to specialize in the production of these commodities. This factor has its effect in altering any general calculations that may be made for cities and their suporting area as a whole. Taking the case of any one supporting area, however, while a certain proportion of farmers would be producers of some special commodity for distant consumers (e. g. apples for the northwest), on the other hand, a certain proportion would be relieved from producing for the local centers by the fact of imports from distant producers (e. g., grain from the northwest) and, while it is not held that these two proportions would balance each other, yet they would tend to make any discrepancy in the general calculation less noticeable.

Still another point arises with the calculation of a large supporting area, such as that for the city of Toronto. A number of towns of various sizes are found within the area. In this instance, therefore, a special calculation has been made. In the case of the smaller cities, however, this difficulty is not incurred.

In all, calculations have been made for the twenty-one largest centers in Ontario. The results appear in Table II.

The results given in Table II are based upon calculations in which both the general items of food entering into the dietary of the average family and the yield of these items in the various districts respectively, for which estimates were made, have been taken into account. The "average family" was taken as consisting of five members. There was then worked out the acreage required to supply the various food items. The total area required for the support of 50 people for one year was thus found to be 109.14 acres. It is to be noted that this acreage provides only the amounts of each kind of food grown locally and consumed by the unit of 50 people in one year and no account is taken whatever of other foods, such as imported fruits, etc., which are consumed in addition. The 109.14 acres thus represents the area required to provide home-grown products only. It is to be further noted that this acreage represents only the net area required, and this whole area of land would need to be cultivated to provide the required amount of food. In the case of each area for which a calculation was made, therefore, account was taken of the proportion between cultivated or producing land and total acreage.

TABLE II. POSSIBLE SUPPORTING AREAS OF ONTARIO CITIES.

Name of city.	Population	Total supporting area, sq. miles.	Radius of circle or part of circle of total supporting area, miles.	Radius of area of immediate support, miles
Toronto, city only (census 1911)	376,538	1,676.8	....	32.7
City, 1913 (assessment figures)	445,575	2,591.9	...	35.3
With country (census)	458,432	2,225.4	37.6	....
With country, 1913 (assessment)	533,411	2,591.9	40.6	....
With country and towns, 1913 (assessment)	573,723	2,905.9	43.0	....
Ottawa	87,062	621.25	19.8	16.8
Hamilton	81,969	730.84	15.9	11.8
London	46,300	326.18	10.1	8.3
Brantford	21,132	161.82	7.2	5.5
Kingston	18,874	126.77	8.9	7.8
Peterboro	18,360	148.49	6.9	5.6
Windsor	17,829	119,98	9.6	7.7
Berlin	15,196	104,28	5.8	4.8
Guelph	15,175	97.95	5.6	4.8
St. Thomas	14,054	79.98	5.1	4.4
Stratford	12,946	75.71	4.9	4.3
Owen Sound	12,558	99.67	5.9	5.1
St. Catharines	12,484	101.76	8.0	6.4
Chatham	10,770	60.35	4.4	3.8
Galt	10,299	68.04	4.6	4.0
Sarnia	9,947	80.21	7.8	6.7
Belleville	9,876	63,88	6.3	5.4
Brockville	9,371	73.21	6.8	5.8
Woodstock	9,320	53,69	4.2	3.7
Niagara Falls	9,248	69.56	6.6	5.1

The Construction of Gravel Roads in Iowa.

It has been found in recent years that a well-constructed gravel road will resist destructive automobile traffic better than a macadam road. Furthermore, the cost of construction, repair and maintenance is less than that of a macadam road. An interesting discussion of the methods of building gravel roads best suited to Iowa conditions, by T. R. Agg, professor in charge of highway engineering at the Iowa State College, presented in a recent official publication of the Iowa Highway Commission, is given here in part.

A complete description of the standards and methods used by the Iowa Highway Commission to which reference is made in this article will be found in the issue of ENGINEERING AND CONTRACTING for July 15, 1914.

PROPERTIES OF GRAVELS SUITABLE FOR USE IN ROAD CONSTRUCTION.

The ideal gravel for road construction is hard and durable, well graded from about 2½ or 3-in. pieces down to sand, and possesses good bonding properties. Such a gravel is rarely encountered in Iowa, but nevertheless it is possible by proper selection and mixing to make serviceable roads out of any of the gravels found in the state. Good wearing qualities are, of course, desirable, and most gravels that are available are made up of fairly hard particles and will wear reasonably well. The Deval test may be applied to get the relative durability of gravels, but this is usually unnecessary. Good bonding properties are also very desirable and in the Iowa gravels the bonding material is usually clay. If the gravel in the pit does not contain sufficient clay it can be added, as will be explained later. The amount of clay should be not less than 10 per cent, nor more than 25 per cent, dry measure.

One of the principal sources of gravel is the terrace deposits. These gravels vary greatly in quality and grading, but are, in general, deficient in coarse pieces. Not only do the various deposits differ materially, but the individual deposits also lack uniformity. By careful selection of material from the pit it is usually possible to secure a material that will serve admirably for road work. For example, the upper portion of the pit may contain much more clay than the lower portion and consequently have better bonding properties. In that case the better bonding material should be used for the surface of the road. In other cases the upper portion of the pit may contain too much clay, but in loading the material the material in the upper part of the pit can be allowed continually to fall down and be loaded with the material in the lower face of the pit. Thus a mixture may be obtained that is very well adapted to road purposes.

A great deal depends upon care in the selection of materials, and while any of those usually available will make durable road surfaces, they will not prove equally so. The quality of the work may be greatly improved in many cases by an intelligent manipulation of the available material.

Washed gravel or gravel pumped or dredged from stream beds or bars is often the only material available for road work. Usually these gravels are of very good quality and reasonably well graded, but in general they are all lacking in bonding material. This can be remedied by adding clay when the road is built and the writer has constructed many miles of excellent road from gravels of this character.

PREPARATION OF ROADBED.

Before a gravel surface is placed all grade reduction work should have been completed and the road should have been under traffic for a sufficient time to thoroughly compact the fills. If the road to be improved with a gravel surface requires but a small amount of shaping to bring it to the proper cross-section, that work should be done a sufficient time in advance of graveling to permit the earth to become well packed before the gravel is placed.

A gravel road depends upon the solidity of the earth foundation for its stability. If the foundation is once solid the gravel covering will shed water to a sufficient extent to prevent the sub-grade from softening much. If, however, the gravel is placed on a foundation of newly placed earth, neither the gravel nor the foundation will pack readily, and the road will rut badly under traffic and in wet weather will likely get into very bad condition.

PLACING THE GRAVEL.

Trench Method.—When the road upon which gravel is to be placed has a high crown so that the gravel cannot be placed on the surface without making an excessive crown to the completed road a trench about 6 ins. deep and about 8 ft. wide is cut in the surface of the road and the material thus removed is graded off to form shoulders to hold the gravel in place. The trench should be crowned about 4 ins. and should be neatly made, having straight shoulders and a uniform subgrade or roadbed. If a roller is available for use on the roadbed, all the better; but if it is not, a tractor should be used to pack any loose material that is left after shaping. The material removed, together with the depth of the trench, will form shoulders about 8 ins. high after the loose earth has packed down. The width of the trench should for all ordinary roads be 8 ft. Into this trench the gravel should be dumped at the rate of about two loads (2½ cu. yds.) to the rod (16½ ft.). The gravel should be spread in a uniform layer having a crown of about 4 ins. After the gravel has been placed and spread it should be harrowed thoroughly so as to bring the heavier particles to the top and to secure uniformity in the layer. The harrow should be heavily built with teeth about 1 in. in diameter and 8 ins. long below the frame, the weight on the harrow being about 10 lbs. per tooth. Traffic, including gravel hauling, should be put onto the road as soon as possible so as to pack this lower layer of gravel as rapidly as possible.

For a single-track road the second layer of gravel should be spread about 10 ft. wide, or 2 ft. more than the lower course. To do this the loose shoulder material is pushed back with the grader the proper distance. The second layer of gravel is dumped between these shoulders at the rate of two loads per rod and spread with a crown of about 3 or 4 ins. Traffic will gradually push the edge of the gravel out on the earth shoulder, increasing the crown slightly and widening the road to a width of about 13 ft.

A road constructed as described will have, after settling, a thickness of about 10 ins. for the middle 8 ft. and of about 4 ins. at the edges. This gives a good, serviceable layer where traffic uses the road most, and a thinner layer at the edges which will be adequate to carry the traffic it receives due to vehicles turning out to pass each other.

If a double-track road is desired the lower course should be placed as for a single-track road, but the upper course should be placed 14 ft. wide. This will require the use of four loads (5 cu. yds.) of gravel per rod (16½ ft.), and the loads should be dumped in two rows. The gravel will eventually spread to a width of about 17 or 18 ft., and will have ample thickness to sustain the heavier traffic encountered where a double-track road would be required.

Surface Method.—The surface method is followed where the earth road is flat enough to permit the additional crown obtained when the gravel road is placed on top of the existing earth road. This is the common method followed, and is cheaper than the trench method, and probably better because the hard earth surface need not be disturbed in order to place the gravel, and consequently will be harder.

In following this method earth shoulders are provided by drawing material from the ditches with a grader. It is essential to have these earth shoulders in order to prevent excessive spreading of the gravel, particularly during the first few months the road is used.

The gravel is placed in the same manner as in the trench method, and the same widths and quantities used.

FINISHING THE SURFACE.

The best gravel roads are obtained when the lower course is placed in the fall or winter and kept under traffic until the next year, before the upper course is placed. The lower course must be dragged frequently to keep it smooth, and if bad ruts or depressions show up these should be filled to prevent traffic cutting entirely through the layer of gravel. Considerable mud will work into the gravel, but that does no harm to good coarse gravel and is very necessary with fine gravel. In fact good surfaces have frequently been secured with gravels scarcely coarser than good sand when this method is followed.

The upper course may be placed any time the year following the placing of the lower course, and when it has been dumped and spread the construction of the surface has really just begun. As traffic uses the road ruts and uneven places will form, and the road must be smoothed repeatedly with a blade grader to prevent these uneven places from becoming permanent. After a time the gravel will pack so that a grader will not move any material, and when that stage is reached the road drag should be used. The drag will be of service only when the road is very wet after heavy rains. Careful dragging at that time will fill all the hollow places and cover the surface with the finer material (mostly mud), which will cement the particles of gravel together and maintain a perfect surface. Too much emphasis cannot be placed upon the importance of maintenance during the first year a gravel road is used. At the end of that time the gravel will be so thoroughly compacted that it will be difficult to remove any unevenness in the surface. A gravel road must be hard, but for comfort to traffic it should also be smooth.

If the gravel is deficient in bonding properties and packs very slowly on the road a small amount of black dirt or clay, free of vegetable matter, should be spread over the surface. This can be done by a careful grader man by grading on a small amount of loam from the earth side roads. It is probably better to push up a ridge of loose earth with the grader and throw it on the road with shovels. If the loose places appear only in spots on the surface, then earth should be thrown on the loose places until they pack. All of this requires care and the work may have to be done two or three times before it is satisfactory.

If the gravel contains too much clay and the road gets sticky when wet, sand should be added to the surface until the stickiness disappears. Do not put on too much at first. An inch is usually enough, but if it appears after a time that more is needed the necessary addition can be made.

When river gravel is used the lower course should be placed, and then covered with about 2 ins. of clay or loam. The clay or loam will disappear under traffic, but when the road gets wet it will work up into the voids and bond the gravel. Meanwhile the surface must be frequently smoothed with a grader or it will get hopelessly out of shape. When the lower course is well packed the upper course should be placed in the same way. If it appears that insufficient clay has been used add more, a little at a time, until the surface is hard and smooth. Keep the surface in shape by frequent smoothing with the grader or drag.

Gravel roads may be placed in one layer and if they are kept smooth will finally become hard and serviceable, but the two-course method is much better.

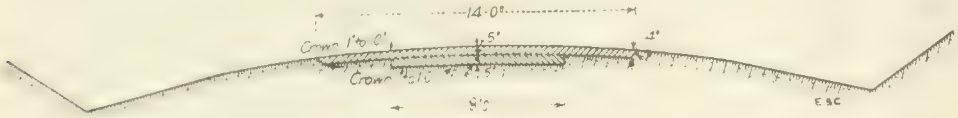


Fig. 1. Cross Section of Typical Gravel Road Suitable for Iowa Conditions.

A gravel surface 10 ins. thick at the middle and 4 ins. thick at the edge will hold up well and be adequate for all ordinary traffic on any of the soils encountered in the state.

If the road carries heavy traffic the gravel road will hold up, but the surface will wear rapidly. This will necessitate frequent dressing of the surface and even then the road is apt to become uneven. The surface may be maintained by incorporating a good quality of asphaltic oil with the gravel. The oil used should be one which has good bonding properties, and which has sufficient body to hold the surface together.

REPAIRING WORN GRAVEL ROADS.

When a gravel road becomes uneven, as it will under continued use, the surface may be restored by adding a small amount of new material. If, however, new material is added to the surface when it is dry, and consequently hard, the new material will not bond with the old surface, but will be scattered by traffic and do little good.

To get good results it is best to do the repair work early in the spring when the road surface is soft after the long-continued wet weather of spring. The surface can at this time be roughened by means of a heavy harrow so that the new material will bond to the old surface. Enough new material should then be added to restore the shape to the surface. The new material must be shaped with a grader and be kept smooth with a drag until it is well packed. Repairs made in this way will restore an old road and the work is not expensive. The surface obtained is as smooth and durable as that of a new road.

Fourth American Road Congress, Atlanta, Ga.—Mayor Harrison of Chicago has been requested by the Hon. A. B. Fletcher, President of the Fourth American Road Congress and State Highway Engineer of California, to name three delegates to attend the sessions of the Congress at Atlanta, Ga., during the week of Nov. 9.

Forty-seven organizations are taking part in the Congress under the leadership of the American Highway Association and the American Automobile Association. In his letter to the Mayor, President Fletcher calls attention to the fact that practically every state highway commissioner will be present and take part in discussing the problems of road construction and maintenance, and that some of the foremost men in public life will devote their attention to the question of Federal Aid to road improvement, in an endeavor to work out a policy which may be submitted to the Congress of the United States with the support of the Road Congress. An important move bearing upon state legislation will be made at the session to be held under the auspices of the American Bar Association, at which a joint committee, appointed at the 1913 Congress, will report progress in compilation and suggested revision of state road laws. The creation of a commission participated in by each state to work out a revision of the road laws will be urged. The National Civil Service Reform League will hold a session on the merit system in road administration.

President Fletcher calls attention to the exhibits to be made by the United States government, the states, and more than a hundred of the leading manufacturers at the Congress, which will illustrate every known method, material and equipment for road construction and maintenance. He urges that the city and county be officially represented, as the Congress is in reality a training school where a very great amount of useful information can be obtained through attendance at lectures with leading specialists in road and street work, and the collecting of the many instructive bulletins which will be available for distribution.

The headquarters of the Congress are in the Colorado Building, Washington, D. C., in charge of I. S. Pennybacker, executive secretary, and the exposition is in charge of Charles P. Light, business manager.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design and to Methods and Cost of Construction

Volume XLII. CHICAGO, ILL., SEPTEMBER 2, 1914. Number 10.

The Vital Details of Rapid Sand Filters.

The typical article on the design of a rapid sand water filtration plant recites the conditions indicating the desirability of filtration, describes the general layout of the works, faithfully describes the various reinforced concrete boxes which, assembled, form the bulk of the plant and, in a brief paragraph, summarizes certain matters of detail pertaining to the controlling apparatus and its auxiliaries. Little has been published about the economic and service considerations which should govern the designer in choosing or proportioning these features of detail. In view of this condition we are confident that the article entitled: Some Features of Detail in the Design of Rapid Sand Water Filtration Plants, contributed by Mr. George W. Fuller and published in the water works section of this issue, will be of great interest and value to water filtration engineers generally.

The article relates to the selection of filter sand and gravel, to the design of overflow troughs, strainer systems, chemical mixing tanks, and to the selection and proportioning of hydraulic valves, conveying machinery, chemical solution pipe lines, water pumps and air blowers. The success or partial failure in operation of a filter plant depends primarily upon the manner in which these features of detail perform their functions. For this reason they are matters of first importance to the designing engineer even though, as Mr. Fuller says, they are often entrusted to the experienced engineers of the filter companies.

Concerning Future Building Codes.

The lack of uniformity in the provisions of the building codes of our principal cities has undoubtedly tended to retard the progress of building construction. The designer and the builder alike have often found it difficult to engage extensively in building work due to the wide differences in the provisions of the building codes of various cities. In many cases too much is left to be decided by building commissions, making it difficult to determine in advance whether or not certain materials and types of construction will be permitted. To be most effective building codes should contain provisions which point out as fully as possible to the designer and builder how he can best insure the acceptance of his work. On account of the great number of interests which are concerned it is too much to expect any code to be sufficiently complete as not to require some explanation and interpretation. On this account it is essential that men of broad judgment and public interest be selected as members of building commissions. Engineers and architects in the past have been too little concerned as to the personnel of such commissions, although they have been most vitally affected by its rulings. Even though the provisions of a building code are satisfactory, to insure good building construction, it is necessary that a sufficient number of competent inspectors be employed. The number of inspectors employed by many cities is far too small to do effectively the work required of them.

The need of a basic building code has been recognized by the American Institute of Architects as a guide in the framing of specific codes, and steps have been taken to investigate fully this subject. A sub-committee, appointed by the Committee on Contracts and Specifications, has recently submitted a report which emphasizes the importance of the preparation of a basic building code. This sub-committee, which consisted of A. O. Elzner, Cincinnati, chairman; R. E. Schmidt, Chicago; Thomas Nolan, Philadelphia; Edward Stotz, Pittsburgh; Ernest Flagg, New York; Norman Isham, Providence; and L. C. Holden, New York, in its report, proposed that the Institute should plan, outline the general scope, provide proper places for details, and invite other societies specializing in the respective branches to co-operate in the preparation of such a code. A financial scheme was outlined and the following program of action suggested:

> In regard to a definite program for the conduct of its work it is recommended that the president of the Institute appoint a special committee of five on "Basic Building Code." This committee should be charged with the task of making a thorough investigation of the entire subject of building codes and should thereupon prepare a definite general scheme for a basic code and submit such scheme to the various chapters, through duly credited committees of each chapter, for consideration, and return report with suggestions. After such a scheme shall have been finally perfected and approved the special committee shall nominate a suitable advisory expert to be engaged by the Board of Directors, who shall be charged with the work of compiling the details of the general scheme. These details, having been classified, shall then be transmitted to the respective societies which may be invited by the standing committee to co-operate in the work. The advisory expert shall collate the various reports on details and submit them to the special committee, which in turn shall submit them to the chapter committees for consideration and approval. Finally, the entire code shall be edited by the expert, subject always to the approval of the Board.

The report of the sub-committee was adopted by the Board of Directors and the president of the Institute appointed the following as members of the special committee on "Basic Building Code": A. O. Elzner, chairman; E. J. Russell, Edward Stotz, Thomas Nolan and R. F. Almira. This committee was instructed to carry on its work as indicated in the report of the sub-committee and to report at the annual meeting of the Board.

The importance of the work of such a committee will be appreciated by engineers, and it is highly important that the engineering features of this basic code be given full consideration. Moreover, engineers should see to it that they have proper representation on the building commissions of our cities. The legal right of cities to enforce the provisions of building codes, designed for the safety and welfare of the people, has been definitely established. Based upon broad police powers in their enforcement of laws governing the safety and welfare of the community, building commissions may be expected to exercise greater authority than in the past. Our future building codes will undoubtedly contain more provisions designed to protect the rights of the community, even at the expense of individual interests.

A Suggestion to License Sewage Plant Attendants.

During the year 1913 the State Board of Health of New Jersey made 384 inspections of sewage disposal plants. These inspections were only for the purpose of supervising operation since all additions to and other structural changes in plants are covered by the Engineering Department of the Board. At the end of the year there were 130 sewage disposal plants in active operation in the state. Thus we see that on the average each plant was inspected, with respect to its operating conditions, three times during the year. Such thorough inspection in a state so far advanced in the treatment of sewage as is New Jersey should lead to interesting conclusions relative to the competency of those in charge of plant operation. It is, therefore, exceedingly interesting and illuminating to note the following comments on this matter by Mr. Francis E. Daniels, Director of Water and Sewerage Inspection for the Board, as published in the annual report of the Board for 1913:

> The experiences of the past year and the difficulties encountered in the field have served further to demonstrate the need of frequent expert supervision of the operation of sewage disposal plants. It is unfortunate that the majority of the men in charge of disposal plants do not have enough knowledge or pride in the plants under their charge to keep them in proper condition. Very frequently it is found that an attendant will permit his plant to run on with the least possible amount of labor, and by such men our assistants find themselves regarded more as policemen coming around to censure and complain than as co-operators ready and willing to advise and help.
>
> The experiences of the past year have also served to demonstrate further the desirability of having sewage disposal plant attendants registered or licensed by the State Board of Health. If such a license was necessary it would prevent much of the trouble now encountered, due to attendants who are absolutely ignorant of the fundamental principles of sewage disposal. The need of intelligent attendants is particularly necessary in disposal plants fitted with complicated dosing or alternating devices. Such arrangements at best require the attention of a careful and capable man, and much of our trouble in the field is due either to the improper construction of the dosing device or to the fact that the attendant fails to understand or comprehend the mechanical contrivances and their operation. It might be well at this time to state that too much reliance should not be placed on these automatic alternating and dosing devices, for they easily get out of order, and when out of order usually require the services of some one thoroughly familiar with their construction and operation.
>
> A great deal of trouble is frequently experienced with new plants shortly after they are first placed in operation, owing to the fact that the designing engineer, upon the completion of the plant, leaves without giving any operating instructions, and gives the subsequent conduct of the plant little if any attention. It would be well if the engineer could be required to supervise the operation of the plant for a period of one year after its completion. This would result in the plant being run as intended, and defects in design would be avoided in subsequent plants.

Few topics have been more widely discussed before engineering societies, in recent years, than the neglect of sewage treatment plants. The incompetent and indifferent operation of such works, or even their absolute neglect, is a condition well understood and widely recognized by engineers. There is little to be gained at this time by the continued reiteration, in engineering society discussions, of a matter so familiar to the entire profession. Definite action to correct the abuse of sewage plants should be instituted and carried forward by engineers. Mr. Daniels proposes a plan and it should be seriously considered.

In states where a central authority has the power to pass upon designs for works, the power to supervise operation should also be granted to the same authority. This is clearly a state function, owing to the clash of intercommunity interests where incompetent operation exists. The central authority being thus empowered to inspect operation conditions should be given sufficient additional power to secure competent operation. Experience has demonstrated that proper operation is not secured where the city authorities are permitted to choose the superintendent as they please or to make no provision for operation at all. This has been repeatedly demonstrated in every state in the union. If the central authority had the power to pass upon the competency of plant operators the existing abuse would speedily be corrected. The issuing of a license to the would be operator, following a suitable examination to ascertain his qualifications, seems to be the most logical solution yet proposed for this troublesome matter.

Engineers will do well to help secure this additional power for state boards of health or similar bodies now having control of plant design. Their active co-operation will greatly aid the central authority in securing the greatly to be desired additional power. They have much to gain in taking this action since the average sewage treatment plant is now foredoomed to at least a partial failure due either to incompetent operation or total neglect. This condition is prejudicial to the engineer's business interests. Moreover he has an important public duty to perform in this matter.

The concluding paragraph of the above quotation also brings out a point upon which we have several times commented in these columns. The designer should keep in closer touch with his plants, both to aid in their proper adjustment and operation and to observe their shortcomings and thus avoid the repetition of his mistakes. If operators are licensed there seems to be no need for requiring the designer to supervise the operation of his plants for any stated period of time, but to safeguard his own reputation he should prepare instructions for the operator's guidance and should occasionally visit his plants to see how they are working.

The Superintendents' Day at Water Works Convention.

In the early days of our large water works associations the membership consisted chiefly of superintendents of water works. Many of these men had worked their way up through various positions in their departmental organizations until, finally, they become superintendents. They were called "practical" water works men. These men were at that time chiefly concerned with the physical problems connected with the collection and distribution of raw water and were practically untroubled by the considerations of quality of supply and strict economy in construction and operation which have since been brought to bear upon water supply problems by chemists, bacteriologists and consulting engineers. In recent years the technical experts have rather monopolized the time of the water works conventions and the superintendents, with the exception of those having technical training, have usually played only a thinking part at the conventions. This condition was not entirely satisfactory to the superintendents and, sensing this dissatisfaction, the program makers have endeavored of late to appeal more directly to the interest of the superintendents by giving over a certain part of the convention's time entirely to a consideration of the problems of immediate daily concern to the superintendent.

We have previously commented upon the success of the "superintendents' day" at the latest annual meeting of the American Water Works Association. This initial session of this kind was so successful that hereafter at every convention of the American Association, Thursday of the convention week will be devoted to the discussion of questions pertaining to the practical management and operation of plants.

The New England Water Works Association has taken up this plan, and at the coming annual convention of the Association, to be held at Boston on Sept. 9-11, two sessions will be of particular value to water works superintendents. The afternoon session of Thursday, September 10th, and the forenoon session of the next day, will be known as the Superintendents' Sessions and will be given over entirely to the reading and discussion of short papers of "live interest to those responsible for the daily maintenance and operation of water works." There will also be demonstrations of the cleaning of water mains, of pipe calking by machine, of the cleaning of service pipes by paper plug and pressure pump, of gate operation by attachment to motor truck, and other contrivances of interest to superintendents.

The creation of the superintendents' day is the most important development in recent years in the conduct of water works conventions. These sessions should prove very attractive to the water works operator and undoubtedly will greatly widen the sphere of wholesome influence and usefulness of the water works associations. The "steering" committees of the associations have now given the superintendent the recognition which he deserves and which he has sought, and he should now do his part to make these sessions highly successful. His part is to join at least one of the large associations, attend its meetings and take part in its proceedings.

"Safety First" in Road Building and Traffic Handling.

The nation wide propaganda to reduce the number of accidents, well summed up in the slogan "Safety First," has lately been spreading into the highway field. The idea that country roads would ever become dangerous to travelers, except possibly from highway robbers, would have amused our forefathers. But the transition from the Scylla of highway robbery to the Charybdis of excessive speed of vehicles and heavy traffic has required but a short time. That danger exists and is worthy of consideration may be quickly seen by only a cursory survey of current newspapers—although at the present time fatalities from other causes are more numerous.

What constitute safety measures in the layout of country roads? Statistics show that most fatalities on country roads occur at grade crossings. Another prolific source of serious accidents are sharp and unprotected curves on heavy grades where the traveler is unable to see any safe distance ahead, or where a high fill at that point is unprotected by a guard rail. Defective bridges, the approaches of which are unprotected by guard rails and culverts that are either in a poor state of repair, or are not sufficiently long to extend across the entire widths of traveled way, are the causes of other accidents. On paved roads and on some earth roads slipperiness of surface has resulted in disaster to vehicles. These and other causes, of not quite so frequent occurrence, have led to injuries of more or less importance. The conditions mentioned result from the design of the road. They are familiar not only to engineers but to all users of the road. The remedy is obvious, the only question being the extent to which the increased cost of eliminating sources of danger will be favored by the people as a whole.

There are, however, other causes of serious accidents that may be eliminated by co-operation on the part of the road users themselves. Of what should this co-operation consist? The answer lies in the education of the traveling public in the use and application of the rules of the road. Disregard of traffic signals on a railroad often results in a disaster, the occasional disregard of similar rules on a high road may not result in an accident, but it leads to their frequent disregard and ultimate disaster follows. The point which needs the greatest emphasis is that traffic rules should never be violated; and that such violation results not merely in danger to the violator, but others who are observing the rules are endangered to a greater extent than if there were no rules of the road.

The cause is a worthy one and deserves the encouragement and active support of every road builder. More can be accomplished by enlisting the support of the users of the road than by the prosecution of offenders. An appeal for the observance of the rights of other people undoubtedly have greater effect on the reckless and thoughtless driver—who is usually the offender—than a threat or a fine. The very fact of his recklessness makes any punishment largely ineffective. But by securing his support this surplus of daring may be made to serve a useful purpose in influencing others to further the cause of safety first.

BUILDINGS

Some Details of the Reinforced Concrete Building of the Ford Motor Co., Chicago, Ill.

(Staff Article.)

The assembling and service building of the Ford Motor Co., Chicago, Ill., has a frontage of 164 ft. on 39th St. and extends 232 ft. along Wabash Ave. The structure is of reinforced concrete construction, the columns being [illegible] for the center bay of the building where the column spacing is 35 ft. The exterior concrete walls are faced with brick with terra cotta trimmings, except on the alley side. The architectural treatment is especially effective, and the lighting facilities are excellent as approximately 60 per cent of the exterior of the building is glass. In addition, the building [illegible] monitor with a glazed glass roof surface.

[illegible]

The reinforced concrete columns have spread footings resting on a bed of sand, the footings being designed for a soil pressure of 3,500 lbs. per square foot. Under the street sides the footings form a continuous girder 5 ft. 6 ins. thick and varying in width from 7 ft. 4 ins. to 8 ft., the top of the footing being 15 ins. below the floor line. On the alley side and at the south end of the building the typical exterior columns have footings 14 ft. 3 ins. square. Those for south end of the building were designed for full panel loads, as it is the intention later to extend the building southward. The footings for the interior columns are stepped and have bases varying in size from 13 ft. 6 ins. square to 17 ft. 9 ins. square.

The floors are of the "Akme" type of flat slab construction and have a thickness of 11 ins. They are designed for a live load of 150 lbs. per square foot. The first floor rests directly on the ground.

The octagonal reinforced concrete columns have outside diameters varying from 24 to 40 ins. The capitals are also octagonal, with a 45° flare, and carry a cap 7 ft. square by 9 ins. thick.

A railroad track runs through the center of the building, the elevation of which is sufficient to bring the floors of the cars on a level with the second floor of the building. Figure 6 shows a view of a portion of the center panel and indicates the adjacent floor and column construction. This view also shows a portion of the roof monitor. The track, which is in the center of the 35-ft. bay, is placed in a concrete trough 5 ft. deep by

16 ft. wide. The main track girders are 16 ins. wide by 7 ft. 3 ins. deep, between which there is a 21-in. slab supporting the track. One of the track girders is carried by the main

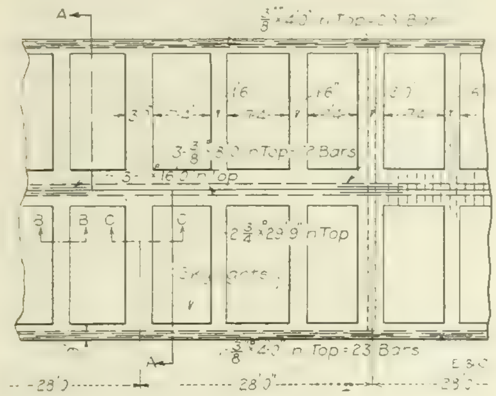


Fig. 1. Plan of a Portion of the Roof Monitor of the Ford Motor Co. Building, Chicago, Ill.

columns, while a parallel girder is carried on independent columns which are spaced 28 ft. on centers. This construction is designed for Cooper's E-40 loading, with 25 per cent added for impact.

By referring to Fig. 6 it will be noted that the center bay is open from the second floor to the roof, except for the floor cantilevers. This open space is served by a traveling crane, which runs the entire length of the building. By means of the cantilever balconies all floors are served by this crane. These balconies are extensions of the floor slabs and project 8 ft. 6 ins. beyond the column centers.

Figure 1 shows a plan of a portion of the roof monitor over the 35 ft. center panel. The trusses are spaced 28 ft. on centers, there being two intermediate rafters in each panel. Figure 2 (a) shows a cross section of the roof and side walls of the monitor and gives the general dimensions and sizes of the members. The left-hand portion of this section shows the roof truss and the right-hand portion, the rafters. The rails of the crane runway are spaced 33 ft. 6 ins. on centers, being carried on reinforced concrete girders. Figure 2 (b) shows a cross section of a rafter and gives its dimensions and reinforcement; while Fig. 2 (c) shows a cross section of the upper chord of a truss. The monitor was designed for a live load of 35 lbs. per square foot.

Figure 3 shows details of the ridge beam, the lower chord of the truss, and the tie which supports the lower chord at its center, and Fig. 4 shows a detail of the vertical wall of the monitor and its connection to the truss. Figure 5 shows an elevation of the end of the monitor and gives the sizes and arrangement of the rods and angles used to reinforce the 12-in. end wall.

The crane girder for the 28-ft. span has a width of 2 ft. 3 ins. and a depth of 3 ft. 1½

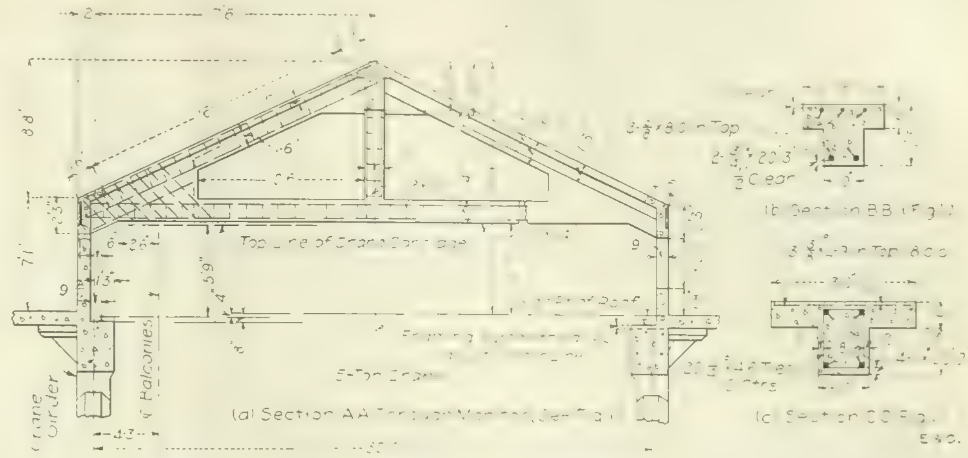


Fig. 2. Cross Section of Monitor and Beam Details of the Ford Motor Co. Building, Chicago, Ill.

ins. Figure 6 (a) shows an elevation of this girder, and Fig. 6 (b) shows a cross section of it and of the portion of the wall below a window opening. The crane girder was designed for a 5-ton crane. The wheel concentrations (including impact) were 12,000 lbs., the wheels being spaced [illegible] ft. 6 ins. on centers. The rails were fastened to the concrete girder by means of the American Bridge Co.'s clamp No. A24, the clamps being spaced 2 ft. on centers.

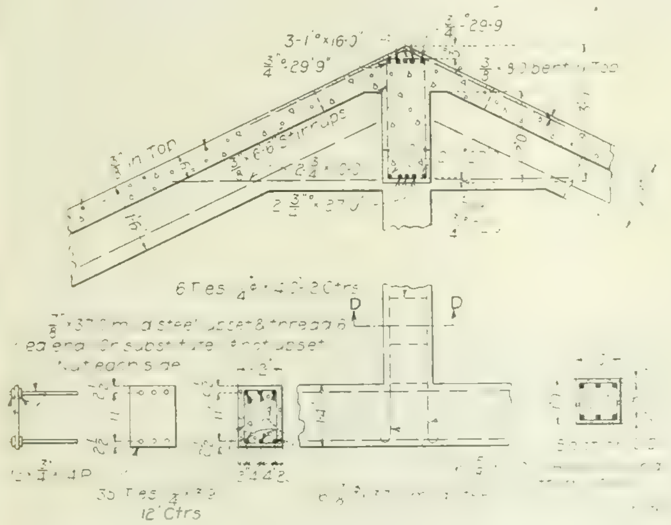


Fig. 3. Section Through Peak of Monitor and Details of Trusses, Ford Motor Co. Building.

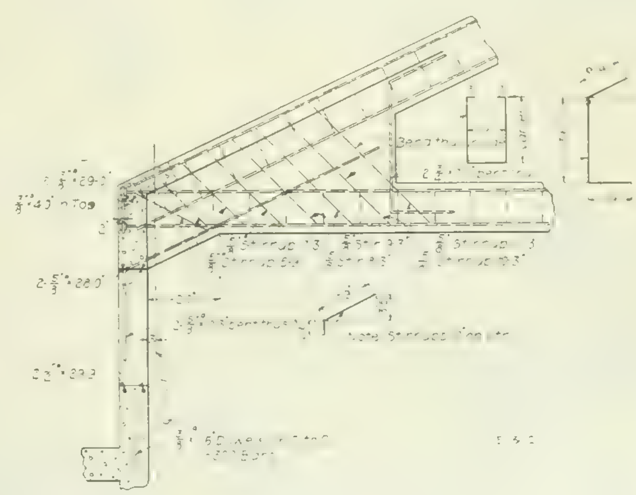


Fig. 4. Details of Side Wall of Monitor and Connection Between Wall and Truss, Ford Motor Co. Building.

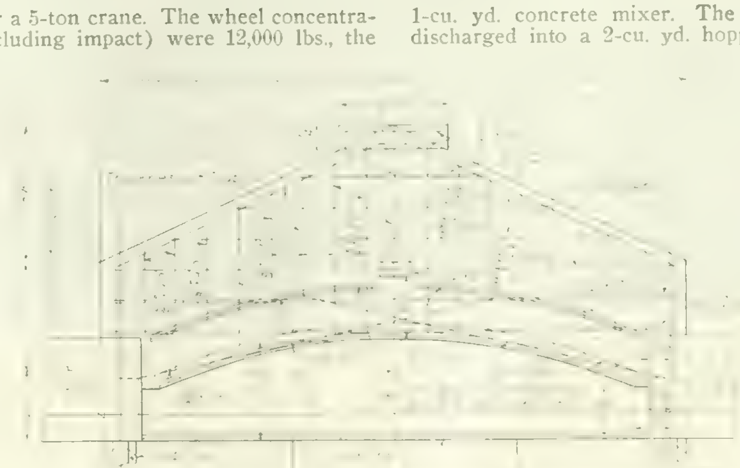


Fig. 5. End Elevation of Monitor of Ford Motor Co. Building Showing General Dimensions and Reinforcement in End Wall.

CONSTRUCTION FEATURES.

The concrete materials were brought to the site in cars, over an elevated trestle, and were dumped into storage bins located under the trestle just south of the building. By means of a belt conveyor the sand and gravel were delivered to bins which were placed above the measuring hoppers, the latter being above the 1-cu. yd. concrete mixer. The concrete was discharged into a 2-cu. yd. hopper which fed [illegible] bucket [illegible] tower.

The concrete, which was a 1:2:4 mix, was spouted into place, except that for the monitor.

The chutes consisted of a trough of sheet steel placed between two 8-in. channels. These channels had bottom lacing and were spaced 12 ins. back to back. By means of struts and cables a truss system was formed, the channels constituting the top chord of the truss. The concrete was chuted from the main tower, which was placed in the south end of the building, to a shorter tower placed near the center of the structure, from which it was chuted to all points of the site. The chute which extended from the second tower was provided with gates at about 10-ft. intervals, through which the concrete could be drawn through flexible pipes.

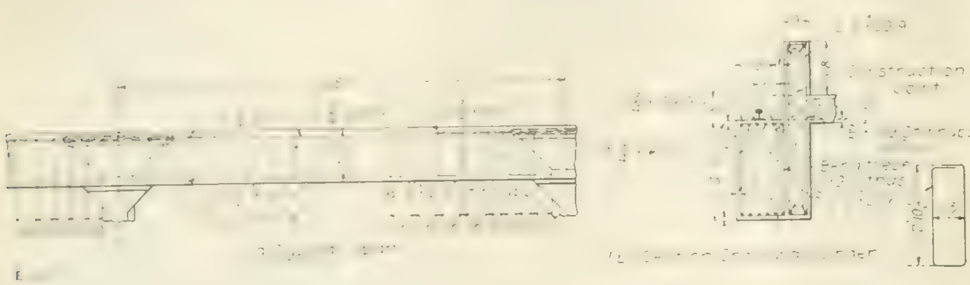


Fig. 6—Elevation and Cross Section of Crane Girder, Ford Motor Co. Building.

Figure 7 shows a test load in place on one of the cantilever balconies. The balcony tested was 42 ft. long and extended 8 ft. 6 ins. beyond the center line of the columns. A test load of 310 lbs. per square foot, in addition to the weight of the slab, was applied and allowed to remain for 24 hours, at the end of which time the measured deflection at the edge of the slab was found to be $\frac{5}{32}$ in.

Figure 8 shows a view of the front and side of the completed building. The effective architectural treatment of the structure is well illustrated by this view.

Fig. 7. View of Center Bay of Ford Motor Co. Building, Showing Floor Cantilevers, Roof Construction and Track.

Fig. 8. View of Completed Building of Ford Motor Co., Showing Architectural Treatment.

The design of the building was in charge of John Graham, supervising architect for the Ford Motor Co., the structural design being made by the Condron Co. of Chicago. E. L. Schneidenhelm, of Chicago, was the general contractor.

The Writing of Building Codes: Some Specific Data and Recommendations.

It is highly important that the commissions having in charge the revision of building codes secure the co-operation of engineers and architects; and for the interests of all concerned it is essential that competent engineers and architects be chosen as members of such commissions. That many building codes are unsatisfactory is due mainly to the fact that those in charge of building design and construction have not interested themselves sufficiently in the framing and revision of such codes. The following data on the writing of building laws were abstracted from a paper by John A. Ferguson, engineer, Bureau of Building Inspection, City of Pittsburgh, in the Proceedings of the Engineers' Society of Western Pennsylvania. The data apply specifically to a building law for Pittsburgh, although most of the provisions are applicable to American cities in general.

ORGANIZATION AND ADMINISTRATION.

The administrative portion of a code should come first. It should establish the status of the inspector and of all assistants; it should name salaries which will attract industrious and capable men, and should outline the work to be done and the records to be kept in sufficient detail to give a capable man the proper instructions. It also should possess such flexibility that improvements in methods are not prevented but are given every advantage.

Figure 1 gives what the writer considers to be a suitable organization for a "Bureau of Building Inspection," for such a city as Pittsburgh. In explanation, it should be said that under the present state laws of Pennsylvania (which are being revised) other bureaus have been established which should be brought into the organizations here suggested as divisions. The idea is to state simply what would make an efficient organization; to show how the work could be arranged to minimize the overlapping of inspections; to save effort on the part of applicants for building permits; and so to arrange and co-ordinate matters that all building inspection work would be under one organization. One head of each division, who remained most of the time at the office, could, by keeping an effective system of reports, have control over his particular division, being held responsible to the superintendent for its efficiency. For each sub-division one of the inspectors should be regarded as "senior," his district being located downtown; and it is important that this district be not too extended in order that the senior inspector shall have time to confer frequently at the general office with the head of his division.

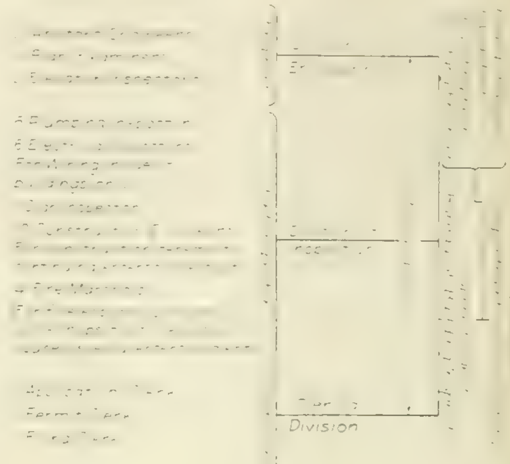


Fig. 1. Diagram Showing Organization of a Bureau of Inspection for such a City as Pittsburgh.

A permanent "Board of Appeal," consisting of one representative of each of the principal divisions of the work to be covered should be provided for in order to decide appeals from the decisions of the building inspector, to pass upon rulings of the bureau, and to act as a permanent commission for the advancement of the building laws.

In addition to the organization, the general manner of arranging the work should be laid down. The reports to be made to the city and to the state should be described so that an efficient head of the office would be able to catch the correct idea and do the work as desired. The general manner of procedure in taking out permits, in serving notice of violations of the laws, penalties for violating the laws, regulating permits required for change of occupancy and all such matters should have careful consideration.

It will be found to be of the greatest advantage to regulate the matter of building permits in general as follows: The permit given the contractor, the engineer, or the architect should be for construction only. The application for permit should state, in addition to other information, the intended use of the building, leaving the classification of structure to the building inspector, under the law itself. Upon completion of the building operation the owner should receive notice as to whether or not it is accepted by the inspector and a permit issued for a specified occupancy. Successive permits should be required for each change in kind of occupancy. This control over occupancy would require very careful study, but could be worked out.

Another detail which rarely is found to be handled satisfactorily in building codes is the plans for a structure. In Cleveland duplicate sets must be furnished to the inspector, and both sets must be marked "approved" before a permit for construction will be issued. One set is kept for the files of the inspector's office and the other is returned to the contractor to be kept by him for the inspector's use. The inspector can thus be assured that when he goes upon a job he has the particular plans for which a permit has been issued. This procedure would add largely to the efficiency of the inspector's work, and two principal results should be noticed immediately, one of which relates to the bureau's record in the files. Contractors and architects would then have to file, in the office of the inspector, all plans which cover changes in construction as determined upon after work has begun. This will make the file of plans in the inspector's office complete and accurate. Moreover, the inspector will have added interest in checking the construction from the plans, as he will be assured that he is working with the approved plans.

DEFINITIONS.

The part of the code which relates to organization and administration should be followed by a complete list of definitions of every name used. These definitions should be *brief* but *accurate* and should not contain specifications.

CLASSIFICATION OF STRUCTURES.

The section relating to the classification of structures is one of the most important sections of a building code. There are many different opinions as to details, and it is of interest to note that a perusal of the criticisms offered to the Pittsburgh commission on the steel and concrete ordinances shows that not one person offered suggestions on the general and broader aspects of the plan. Each individual correspondent read the ordinances and picked out a few details which he praised or criticised as his fancy dictated.

A carefully planned and well worked out building law will not, as some fear, supplant the work of the high class engineer, architect, or contractor; but on the contrary the general public will be led to appreciate the work that the really good professional men are doing, and this will lead to their sure employment because the need for such experienced services will become apparent.

The following classification of structures was arrived at after considerable work on the inspection of buildings and a careful study of the laws of other cities, gleaning the best from each and writing it into this classification. Classification is made according to the requirements for resistance to fire hazard. Grade is given as definition of kind of occupancy.

Class I.—Buildings in which all structural parts carrying loads or resisting stresses are made of incombustible materials protected by fire resisting materials, and all walls, partitions, doors, trim and sash are made of fire-resisting materials, all of which will fulfill the requirements given in Table I, and in which all glass in windows, doors and transoms or skylights are made of wire glass. All openings through floors for fire stairs must be made of fire-resisting enclosures, as given in sub-classes *F*, *G* and *H* (Table I). All floor openings other than fire stairs, such as elevators, escalators and ordinary stairways must be completely surrounded with fire-resisting enclosures, as given in sub-class *H*, they must be provided with doors made of incombustible materials and wire glass, and must have automatic fire doors for every opening. In other respects all partitions and enclosures to be as specified for occupancy. Occupancy should then be defined, and the description of the requirements for each kind should be given under the heading "*Occupancy.*"

Class II.—Buildings in which all structural parts carrying loads or resisting stresses are made of incombustible materials protected by fire-resisting materials, and in which all walls and partitions are made of fire-resisting materials which fulfill the requirements given in Table I. All doors, wall trim, window sash and floor finish may be made of combustible materials. All glass may be ordinary glass. All openings through floors for fire-stairs must be made of fire-resisting enclosures, as given in sub-class *N* (Table II). All floor openings other than fire stairs, such as elevators, escalators and ordinary stairways must be completely surrounded with fire-resisting enclosures, given in sub-classes *O* and *P*, and must be provided with doors made of incombustible materials and wire glass or have automatic fire doors for every opening. All partitions and enclosures otherwise to be specified for "*Occupancy.*"

Class III.—Buildings in which all structural parts carrying loads or resisting strains are made of incombustible materials protected by fire-resisting materials. All walls

TABLE I.—STANDARD TABLE FOR FULL PROTECTION.

Type of construction.	Sub-class.	Duration of test, minimum.	Average temp., deg. F.	Thickness of material, ins.	Area of test panel, feet.	Water test. Time, min.	Water test. Nozzle diam., ins.	Water test. Pressure, lbs. per sq. in.	Water test. Flooding.	Remarks.
Protection of structural frame	..	240	1700	2 or more	....	10	1⅛	60	..	Metal fully protected from corrosion by coating.
Fire resisting floors	..	240	1700		100 sq. ft.	10	1⅛	60	‡	See notes * and †.
Fire resisting roofs	..	180	1700	3 or more	....	5	1⅛	30	‡	
Partitions, fire division	..	180	1700	5 or more	9½×14½	5	1⅛	50	..	For dividing stores and important floor areas.
** Partitions important	A*	120	1700	5 or more	7¼ × 9½	5	⅞	50	..	For office buildings and storage warehouses.
** Partitions important	B†	120	1700	4 or more	9½ × 14½	1½	1⅛	30	..	To separate office from operating room—factories.
** Partitions—minor	C	90	1700	4 or more	9½×14½	1½	1⅛	30	..	Non-bearing, length = 3 × height.
** Partitions—minor	D	60	1500	4 or more	9½×14½	1½	1⅛	30	..	Non-bearing, length = 2½ × height.
** Partitions—minor	E	45	1500	4 or more	9½ × 14½	1½	1⅛	30	..	Non-bearing, length = 2 × height.
Fire resisting enclosures	F	240	2000	6 or more	9½×14½	5	1⅛	50	..	To separate fire exit from main floor in workshops.
Fire resisting enclosures	G	120	1700	5 or more	9½×14½	5	⅞	50	..	To separate fire exit from main floor in stores.
Fire resisting enclosures	H	90	1500	4 or more	7¼ × 9½	2½	⅞	50	..	To separate fire exit from main hall in tenements.

*Standard specifications of National Board of Fire Underwriters.
†Standard specifications of American Society for Testing Materials.
Note that floors and roofs to be designed for loadings to suit use of building.
**Note that all partitions are to be constructed entirely of materials that will not support combustion, and to be securely keyed to floor slabs, the attachment to withstand test.
‡Flooding optional.

TABLE II.—STANDARD TABLE FOR PARTIAL PROTECTION.

Type of construction.	Sub-class.	Duration of test, min.	Average temp., deg. F.	Thickness of material, ins.	Area of test panel, feet.	Water test. Time, min.	Water test. Nozzle diam., ins.	Water test. Pressure, lbs. per sq. in.	Water test. Flooding.	Remarks.
Protection of structural frame	..	180	1700	1½ or more	....	5	1⅛	60	..	Metal fully protected from corrosion by coating.
Fire resisting floors	..	180	1700		100 sq. ft.	5	1⅛	60	‡	
Fire resisting roofs	..	180	1700	3 or more	100 sq. ft.	5	1⅛	30	‡	
Partitions, fire division	..	180	1700	5 or more	9½×14½	5	1⅛	50	...	For dividing important fire areas.
Partitions—important	I	90	1700	5 or more	9½×14½	1½	1⅛	30	..	Bearing length = 3 × height
Partitions—important	J	60	1700	4 or more	7¼ × 9½	1½	1⅛	30	..	Bearing length = 2½ × height
Partitions—minor	K	90	1700	4 or more	9½×14½	1½	1⅛	30	..	Non-bearing length = 3 × height.
Partitions—minor	L	60	1500	4 or more	9½×14½	1½	1⅛	30	..	Non-bearing length = 2½ × height.
Partitions—minor	M	45	1500	4 or more	9½×14½	1½	1⅛	30	..	Non-bearing length = 2 × height.
Fire resisting enclosures	N	180	1700	6 or more	9½×14½	2½	1⅛	50	..	To separate fire exit from main floor in schools.
Fire resisting enclosures	O	90	1500	5 or more	9½×14½	2½	⅞	50	..	To separate fire exit from main floor in theaters.
Fire resisting enclosures	P	60	1500	4 or more	7¼ × 9½	1½	⅞	30	..	To separate fire exit from hall in office buildings.

Note.—All partitions are to be constructed entirely of materials that will not support combustion, attachment to be secure and withstand test; floors are to be designed to suit use of building; elevator enclosures and doors are to withstand same test as fire division partitions.
‡Flooding optional.

[illegible] materials which will fulfill the requirements given in Table II; all other parts to be the same as in buildings under Class II.

Class IV.—Buildings in which all structural parts carrying loads or resisting strains are made of incombustible materials unprotected by fire-resisting materials. All walls and partitions being made of fire-resisting materials which will fulfill the requirements given in Table II; all other parts to be the same as in buildings under Class II.

Class V.—Buildings in which all the structural frame, including walls, columns, beams and girders are made of incombustible materials unprotected by fire-resisting materials. All floors and partitions to be made of fire-resisting materials which will fulfill the requirements given in Table III. All doors, finish, trim, window frames and glass to be constructed of combustible materials and ordinary glass. All openings through floors for fire stairs to be fire-resisting enclosures, as [illegible] and [illegible] as specified for *"Occupancy,"* and to be provided with automatic fire doors of the same fire-resisting quality as the enclosures or else doors made of incombustible materials. All partitions and enclosures otherwise to be constructed as [illegible] *"Occupancy."*

nent stage upon which stage scenery and theatrical apparatus is employed.

Division "E": Detention Buildings.—Includes all public or private hospitals, reformatories, prisons and police stations.

Division "F": Public Utility Buildings.—Includes all other buildings owned or used by the general public but not classified in the foregoing divisions.

SECOND GRADE: Quasi-Public Buildings.—Includes all buildings used for public shelter either for purposes of business, or for temporary abode or habitation.

Division "A": Hotels.—Includes all hotels, public inns, or any building or part thereof designed to be used for supplying food or shelter to residents or guests and having a public dining room, cafe or office or either. A public lodging house or a building used only for the shelter of residents or guests will be classified as a hotel.

Division "B": Office Buildings.—Includes any building designed or used for office purposes in the conduct of general business, but may have a store or sales rooms on the ground floor. No part of such a building shall be used for residence except by the janitor and his family.

Division "C": Store Buildings.—Any building designed or used for the sale of merchandise or objects of utility or general supplies.

THIRD GRADE: Industrial Buildings.—Includes all buildings designed or used for the manufacture or storage of merchandise.

Division "A": Factory Buildings.—Includes all buildings designed or used for the manufacture of merchandise by machinery. All printing estiblashments will be classified as factory buildings.

Division "B": Work Shops.—Includes all buildings designed or used for the manufacture of merchandise by hand.

Division "C": Mill Buildings.—Includes all buildings designed or used for the manufacture and storage of heavy machinery, structural steel shapes or bars, or castings, and machine shops.

Division "D": Warehouses.—Includes all buildings designed or used for the storage of general merchandise or food or other supplies.

Division "E": Garages.—Includes all buildings designed or used for automobile livery or storage, where five or more automobiles, carrying tanks containing fuel, any volatile or inflammable material, are kept.

Division "F": Slaughter Houses.—Includes all buildings designed or used for the slaughter of animals, the curing, drying or preparation of meat or the by-products thereof, or the rendering of fat or manufacture of soap, bone-dust, fertilizer or other animal product.

FOURTH GRADE: Tenements.—Includes all buildings containing suites or apartments used for the permanent habitation by more than two families living independently of each other.

Division "A": Apartment Houses.—Any building or any portion thereof designed or used as a residence for more than two families living independently of each other, and in which every family or household shall have provided for it a kitchen, set bath tub and water closet separate and apart from any others. A store on the ground floor will be classed as an apartment for one family.

Division "B": Club Houses.—Includes all buildings used or intended for use by an organization or society for mutual entertainment or recreation having a common kitchen, dining room and other rooms of utility and recreation and containing lodging apartments for the use of the members of the organization only.

Division "C": Tenement Houses.—Includes all houses, buildings or portion thereof which are designed to be used or occupied as a home or residence of more than two families living independently of each other and doing their cooking upon the premises or by more than two upon any floor so living and cooking, but having a common right in the halls, stairways, yards, or water closet. A store on the first or ground floor will be considered the same as a home for a family.

FIFTH GRADE: Dwellings.—Includes all buildings which are designed or used as the home or residence of not more than two separate families in which not more than ten rooms shall be used for the accommodation of boarders, no part of which is used as a store or for any business purpose.

TABLE III.—STANDARD TABLE FOR TEMPORARY PROTECTION.

Type of Construction.	Sub-class.	Duration of test, min.	Average temp., deg. F.	Thickness of material, ins.	Area of test panel, feet	Water test. Time, min.	Water test. Nozzle diam., ins.	Water test. Pressure, lbs. per sq. in.	Water test. Flooding.	Remarks.
Protection of structural frame......	..	90	1700	1 or more	... .	2½	1⅛	30	..	Metal fully protected from corrosion by coating.
[illegible]		90	1700	... .	100 sq. ft.	2½	1⅛	30	‡	
[illegible]		60	1700	... .	100 sq. ft.	1½	1⅛	30	‡	
[illegible]		90	1700	3 or more	100 sq. ft.	2½	1⅛	30	‡	
[illegible]		90	1700	1 or more	9½×14½	2½	1⅛	30	..	For division walls between suites in tenements.
Partitions—important		60	1500	1 or more	9½×14½	1½	1⅛	30	..	Bearing-in.
	.	45	1200	3 or more	7¼×9½	1½	⅞	30	..	Non-bearing, in residences.
Partitions—minor	.	45	1500	3 or more	7¼×9½	1½	⅞	30	..	Non-bearing—length = 2½ × height.
	.	30	1200	3 or more	7¼×9½	1½	⅞	30	..	Non-bearing—length = 2 × height.
Fire-resisting enclosures	..	90	1700	5 or more	9½×14½	2½	1⅛	30	..	To separate fire exit from main floor in nickelodeon.
	..	60	1500	4 or more	7¼×9½	1½	1⅛	30	..	To separate fire exit from main floor—assembly hall.
	..	45	1500	3 or more	7¼×9½	1½	1⅛	30	..	To separate fire exit from main floor—hotel or lodging

Note.—All partitions except "minor" are to be constructed entirely of materials that will not support combustion, attachment to be secure and to withstand same test; elevator enclosures and doors to withstand same test as fire division partitions; all floors are to be designed for loadings to [illegible]

‡Flooding optional.

Class VI.—Buildings in which all structural parts carrying loads or resisting stresses are made of combustible materials, but which fulfill the requirements given and specified for slow combustible construction. All outside walls to be made of fire resisting materials which will fulfill the requirements given in Table I. All fire enclosures and partitions to be the same as in buildings under Class II.

Class VII.—Buildings in which all structural and other parts are made of combustible materials.

FIRST GRADE.—All buildings devoted to the use of the general public for purposes of [illegible]

Buildings designed to be occupied by state, county or city administration offices, court rooms, libraries, museums, art galleries or council chambers.

Division "B": School Buildings.—Includes all school, college or other buildings, containing class, drawing or lecture rooms or rooms for the purpose of education or instruction. If any such building has an assembly room of greater seating capacity than four class rooms such assembly room will be considered as an assembly hall and subject to the requirements therefor.

Division "C": Assembly Halls.—Includes all churches, convention halls, auditoriums, exposition buildings, music halls, railroad departments, or any part of a building containing an assembly room holding more than 100 people.

Division "D": Theaters.—Includes all theaters, opera houses, play houses, pavilions, or any assembly hall designed or used for the entertainment of spectators having a perma-

Two or more such dwellings may be connected on each story when used for boarding purposes provided the halls and stairs of each house shall be left unaltered.

Dwellings built in terraces more than three stories high having two distinct families living independently, or a family on upper floors with a store below will be classed as a tenement house.

SIXTH GRADE.—Includes all buildings used for the shelter of animals or vehicles.

Division "A": Stables.—Includes all buildings designed or used for horse livery, boarding or private stables or barns, carriage houses, sheds, pens, coops, stockyards with attendant slaughter pens or any building for the feeding or sheltering of animals or fowls.

Division "B": Garages.—Includes all garages designed or used for the shelter or storage of automobiles for private use only where not more than five automobiles are kept.

SEVENTH GRADE: Towers.—Includes all structures designed or used for water storage, for sprinkler systems or other purposes. A water tank for any purpose, placed in or near a building whether supported by separate tower or not will be classed under this grade.

EIGHTH GRADE.—Includes all buildings or structures not classed in grades one to seven. For the purpose of this law, fences, bill and sign boards and all signs shall be classed as structures of this grade.

OCCUPANCY

Following the classification of structures the name and definition of every kind of use or occupany should be given, naming the class and grade under which they must be constructed and specifying all the additional requirements necessary in order to provide for the hazards occurring in each. This will cover such structures as tenement houses, theaters, nickelodeons, assembly halls, office buildings, factories, warehouses, etc. These additional requirements will cover electric wiring, plumbing, etc., as well as general sanitary conditions and many others. This is the longest, most wordy and tedious of the various portions of a building law and must be passed over very briefly in such a discussion as this. Mention, however, must be made that all the requirements for each kind of occupancy should be exhausted under its heading, even if there might be some duplication of requirements for other occupancies. This is advocated simply as a method to help to make the laws clear and easy for reference for the convenience of those who must use the code daily. It will be found that such an arrangement renders it less easy to make mistakes in planning a structure, or in enforcing the laws.

CLASSIFICATION OF MATERIALS AND METHODS OF CONSTRUCTION.

Following the classification of structures should come a careful description and specification for the materials of construction. This should be accomplished in a great measure by classifying special requirements such as: *"A"*, Fire Resisting Properties; *"B"*, Resistance to Stresses; *"C,"* Permanency of Construction; *"D,"* Maintenance and Preservation; *"E"*, Sanitary Condition, etc. All classifications should be worked out properly to suit the risks involved. Requirements more severe than necessary to bring about proper application of construction to balance evenly the hazard should not be advocated. Short descriptions should be interwoven with the technical specification in order to popularly summarize the reasons for their use to correspond with the hazard. This last will do much toward removing popular prejudice against the requirements and to prove that the specifications are not more severe than necessary. All classifications should be worked out with the end in view of placing the use of all materials and methods of construction on a basis of merit alone. It should specify results only, since the result desired is the end in view. It should not contain an endless amount of agglomerate details setting forth how the result must be obtained, but it should leave all this to the inventive genius of engineers and to the results given by testing apparatus. Requirements placed against the physical testing apparatus should never be more severe than obtain under the actual conditions of service, and observation should be carefully made under these conditions.

A classification of materials, covering fire-resisting qualities, is now receiving careful consideration by the "National Fire Protection Association" and the "U. S. Bureau of Standards" at the Butler St. laboratory in Pittsburgh. It received its impetus at the "International Fire Prevention Congress," held in London in 1903, by the "British Fire Prevention Committee," and at the "National Fire Prevention Convention," held in Philadelphia in 1913.

Tables I, II and III are built up around the information gained from study of the requirements laid down by the foregoing organizations and differ from them only in two or three cases where experience has suggested that a sub-class might be added to provide for conditions which are less severe than those called for by the requirements. In addition to this, the information given under the column heading "Remarks" and at the bottom of the Tables as notes has suggested itself to the writer as a brief way to explain how the different parts of a structure may be construed to suit the requirements.

Classification of materials from other standpoints has been given careful study by the writer. However, this is a difficult matter to cover completely in a short time and the work is not in shape to present at the present time. The progress achieved so far by the writer in this work has come from requiring those interested in various building materials to conduct physical tests as rapidly as possible and to submit reports of all tests, giving information relative to their use, and to place in writing specifications for their use and the manner of conducting calculations for proposed installations. This has been made to include several materials for non-bearing partitions, several systems of reinforced concrete construction and waterproofing compounds for concrete. The subject of preservative coatings for steel has been brought to the writer's attention quite recently.

When the information is at hand those points now covered by law are separated from those which are not and a decision is made as to the manner in which the inspector will check the work in order to make the proposed construction as safe as other types covered by ordinances. The principles of good engineering practice are strictly adhered to. When a decision has been reached and agreed to by those proposing to use a new type of construction, or building material, a specification covering its use is written and sent to the interested party. All information, tests, reports and correspondence are then filed, together with the specification covering its use. The advantage of this procedure is that users of a given material will know exactly what will be passed by the bureau and they can then proceed with confidence. This method of procedure also insures uniformity in the decisions made by the bureau, thus avoiding the oft-repeated claim that one man is given advantages which are refused another. It also aids in correctness of procedure.

In the tests made of various methods of applying steel reinforcement to concrete strain-gage tests readings are required on steel and concrete. These are to be taken at the time the applied load (which equals the live load for which the structure is designed) is so placed on the structure as to produce the maximum stresses which are likely to occur. However, conditions are never required which will be more severe than those which may be imposed in practice. When this information has been correctly obtained, the assumptions and the theory must be worked out to a sufficient degree of completeness to insure that an estimate of strength may be made with sufficient accuracy to come within the limits set by the uniformity of construction and the behavior of materials.

CONCLUSIONS.

Although a building code need not assume the proportions of a text book, nevertheless considerable information is needed by a designer to enable him to do his work correctly and to be assured that it will pass the requirements of the bureau without unnecessary annoyance and delay. This makes it necessary that all requirements for safety be clearly stated. It should be remembered always that a building law—not a specification—is being prepared. The preparation of building codes should be placed in charge of the broadest minded and most vigorous men in the community, and these men should receive suitable pay for their work. The provisions of building codes should be arranged and properly indexed in about the order which is followed in planning a structure, each division being fully treated before the next one is considered. They should also provide for the constantly progressing state of the science of engineering and the art of construction.

WATER WORKS

Some Features of Detail in the Design of Rapid Sand Water Filtration Plants.

Contributed by George W. Fuller, Consulting Hydraulic Engineer and Sanitary Expert, New York City.

The construction of mechanical filters of the modern type has become common and many interesting plants of first-class construction have been described in the technical press during the last dozen years. The general principles of design are as a rule well brought out.

It is not uncommon, however, for the designing engineer to leave much of the detailed work to be selected and installed by the experienced filter company which builds the plant. While in many cases this may be the most desirable plan to adopt and will give results most satisfactory to the engineer's client, it is well worth while for the designing engineer to consider fully all of the details which make up the filter construction. This article is written to bring before the interested public a few of the factors which go to make up a proper design of these details.

FILTER SAND AND GRAVEL.

Selection of Sand.—At the time of the origin of mechanical filters, when this type of filter was used almost entirely for industrial service, coarse sand was commonly used. As these filters became adapted to municipal service with a higher requirement of bacterial efficiency, finer sand was gradually substituted. In the course of more recent years the general practice has swung back somewhat toward the original practice. Having in mind the possibility of meeting the bacterial requirements by effective sterilization after filtration, it is considered economical for some conditions to use a relatively coarse sand. The particular size selected will vary mainly with the amount of head available and to some slight extent with the character of water which is to be filtered. For ordinary conditions where the available head used for filtration will be about 8 ft., a sand with an effective size of about 0.4 millimeters is most commonly used. The uniformity co-efficient of the sand for mechanical filters is a rather important item, mainly because the reverse washing to which filter sand is subjected causes a grading of the sand bed with the coarse sand at the bottom and the fine sand at the top. To keep this separation within reasonable limits it is customary to specify that the sand shall have a uniformity coefficient of about 1.65.

In the design of the Jerome Park filters for the Croton supply of New York City the normal head for filtration was 4½ ft., and the sand layer was composed of 30 ins. in depth of sand having an effective size of .60 to .70 millimeters and a uniformity coefficient of 1.65.

It is common to require that there shall not be very much fine material present; for instance, not more than 1 per cent shall be finer than 0.25 millimeters. This is not a factor which need be considered in the purchase of the sand, as this extremely fine material may readily be washed out of the sand bed by repeated washings at a high rate, and if necessary scraped from the surface of the sand bed after it has been hydraulically graded by rewashing.

Dirt, clay and organic matter are usually excluded by the sand specifications. But as these likewise can readily be removed by rewashing, the strict enforcement of this clause should be called for only after a certain amount of washing has been done by the contractor.

Limestone is somewhat soluble in ordinary water, and for this reason it is common to limit the amount of limestone allowed in the filter sand. The usual specification is that not more than 1.5 per cent of lime or magnesia calculated as $CaCO_3$, will be permitted; and not more than 5 per cent of the sand shall be dissolved by digesting powdered sand for 24 hours in warm hydrochloric acid.

Sand Testing.—In order to test the grading of the sand submitted for approval, a Riehle sand shaker or its equivalent is commonly used. This consists of a series of pans 5½ ins. in diameter, placed one over the other, the pans having their bottoms of screens or wire mesh in sizes varying from coarsest at the top to the finest at the bottom. One hundred grams of the sand to be tested are placed in the uppermost pan and the nest of screens is shaken 120 times, or some other number of shakes corresponding to the screen calibration, until each screen contains that portion of the sand too large to pass through it and small enough to pass through the screen above. The sand remaining on each screen is weighed. A sample card for plotting the results of a sand test is shown in Fig. 1.

In calibrating a sieve, the sieve is shaken to say 110 times, and after the sand which has passed through is removed, the sand passing through the sieve during the last ten shakes is collected, weighed, and the number of sand grains counted. It is assumed that these last sand grains are a measure of the separation of the wires. The equivalent diameters of these sand grains calculated as spheres is obtained from the relation

$$d = \sqrt[3]{\frac{6w}{\pi G}} = 1.24\sqrt[3]{\frac{w}{G}}$$

where d is the diameter of the sand grain,
w is the weight, and
G is the specific gravity.

The percentage of sand passing through each size of screen is marked on the line opposite the screen number or screen size on this chart and a curve is drawn through these points. The intersection of this curve with the lower of the two cross lines opposite the 10 per cent horizontal line will read the effective size of the sand. The uniformity coefficient will be given by a ratio of the size of the 60 per cent line to the size of the 10 per cent. The lines plotted on Fig. 1 show the allowable limits of the sand specified for the Croton filters.

Filter Gravel.—The sizes, thicknesses and arrangement of the gravel used will depend somewhat upon the type of washing method adopted. Where the low-rate washing method is used, with approximately 8 gals. of wash water per square foot of filter surface per minute, a thickness of graded gravel of 8 to 9 ins. is most common. This will be placed in four layers of about equal thickness, the uppermost layer being graded from No. 20 mesh to $^{3}/_{16}$; the second, $^{3}/_{16}$ to $^{3}/_{8}$; the third, ⅜ to ⅝; and the fourth ⅝ to 1 in. If high-rate washing is used; that is, 15 gals. of water per square foot of filter surface per minute, and if, as is customary in present practice, there is no wire mesh screen placed between the gravel and the sand, it is common to make the gravel about 14 ins. thick. This gravel thickness is the amount experience has shown to be necessary to deflect the water jets and diffuse them over the area of the filter at such low velocity that the finer gravel at the top of the gravel bed will not be disturbed or mixed with the sand when washing.

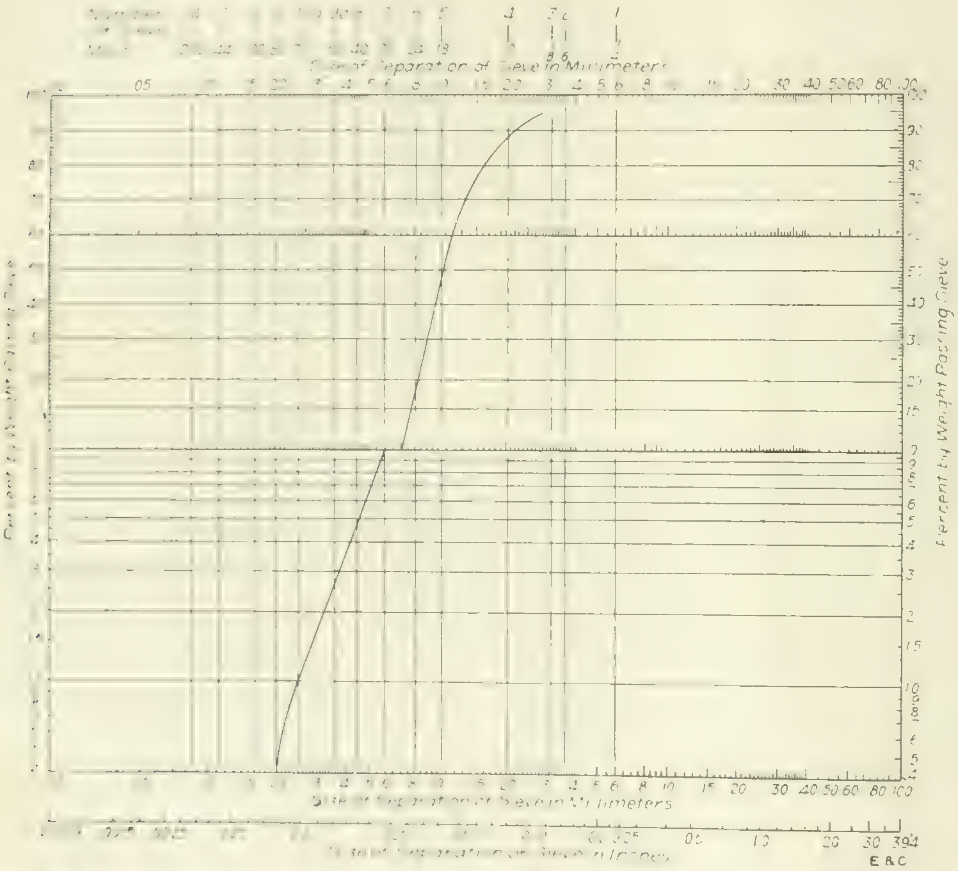


Fig. 1. Sample Card for Plotting Results of Test of Filter Sand—Lines Plotted Show Allowable Limits of the Sand Specified for the Croton Filters.

OVERFLOW TROUGHS.

The troughs to carry wash water may be built either of cast iron, sheet iron or concrete. If of cast iron, they are usually from ½ to ⅝ in. thick, with rounded bottom; if of sheet iron, they are usually ¼ in. thick; if of concrete, 2 to 3 ins. Care must be taken that all the upper overflow edges of these troughs are straight and exactly on the same level.

These troughs are usually placed about 7 ft. on centers. The size may be determined on the basis of a flow velocity of 2½ ft. per second at the largest cross section with the normal amount of wash water passing into the trough.

Some special forms of trough have been devised, having a sand catcher or other appurtenances for special purposes. There has been no evidence presented that any such special features have any marked value.

STRAINER SYSTEMS.

The strainer systems of mechanical filters can be divided broadly into two classes; (1) those consisting of a series of pipes or channels arranged as multiple conduits, allowing an approximately uniform loss of pressure from the filter to the outlet valve in all parts of the filter; (2) those having a false bottom separating the filter from a large size collecting chamber, this false bottom being fitted with suitable individual strainers over its area. The former type is the more common, and nearly all mechanical filters have been built this way; the latter type has been used at a number of plants, such as Panama, Alliance, O., and Birmingham, Ala.

Washing by reverse current is effected at varying rates up to about 15 gals. per minute per square foot of sand area. The form of construction differs for the various rates only in the areas being suited to the volumes of water.

The higher rate of wash systems has in the past more often than not been built of a series of concrete ridges in the bottom of the filter, forming channels which are covered with perforated brass plates. Such a type of design was used by the writer for Evanston, Ill., as shown in Fig. 2. In this type of design the velocities throughout the system are usually kept very low and the velocity through the perforations in the brass plates is made approximately 10 ft. per second so as to require sufficient loss of head through the openings to equalize the supply of water to the various portions of the filter bottom.

TABLE I. — JEROME PARK FILTERS. STRAINER AND AIR SYSTEM, WORKING VELOCITIES, FEET PER SECOND

	Filtration.	Wash.	Air (free.)
Type A—			
Through sand	0.0043	0.02	0.0668
Strainer holes, or air holes	1.50	6.75	260.0
Strainer neck	2.25	10.00	......
Laterals	.71	3.18	86.0
Manifold	1.11	5.00	79.0
10-in. supply	1.46	6.60	......
18-in collector	1.80	8.15	......
24-in. main	2 03	9.17	......
16-in. air main	....	....	68.8
Type B—			
Through sand	0.00443	0.02	.0668
Strainer holes	1.55	7.10	23.4
Strainer neck	2.15	9.70	360.0
Laterals	.58	2.60	8.68
Manifold	1.11	5.00	8.32
10-in. supply	1.46	6.60	......
18-in. collector	1.80	8.15	......
24-in. main	2.03	9.17	......
16-in. air main	....	....	68 8
Type C—			
Through sand	0.00443	0.0333	......
Strainer holes	1.41	10.8	......
Laterals	.26	1.98	......
Manifold channel	.40	3.08	......
14-in supply	.73	5.60	......
24-in. collector	.99	7.61	......
24-in. main	2.00	15.2	......

It is not essential, however, for high rate of wash that the concrete ridge system be used, as it is quite practicable if desired to use a pipe system. Such a pipe system would consist of cast iron manifolds, with laterals either of brass or iron, these laterals being perforated with holes, or fitted with strainer cups or bushings. A design made by the writer for the city of Baltimore, Md., but not used, showing the pipe system for high rate of wash, is shown in Fig. 3.

These lateral pipes can be made of brass at moderate cost, but still cheaper and very satisfactory construction can be obtained by the use of cast or galvanized iron. Such construction has been used at Burlington, Vt.; Botany Worsted Mills, Passaic, N. J.; Harrisburg, Pa.; Montreal, Canada, and Torresdale, Philadelphia, Pa.

For the lower rates of wash systems likewise either the concrete ridge system or the pipe system may be used. A design for a concrete ridge system for low rate of wash used in the Queen Lane filters, Philadelphia, Pa., is shown in Fig. 4. A design for the pipe lateral system intended for the Croton water filters, New York City, under the writer's supervision, is shown in Fig. 5. With this low rate of wash there is often built an air distribution system, such as is also shown in Fig. 5. A modified form of the low rate wash air system consists of the use of the same lateral pipes for both air and water. In this case the outlet to the lateral pipes must come from the bottom of the pipes, either by having a hole in the bottom of the pipe, or by using the Williamson strainer, as at Little Falls, which is a strainer cup screwed into the top of the pipe with an elongated neck projecting downward to the bottom of the pipe.

An idea of the sizes of the various pipes and openings used for the various designs of

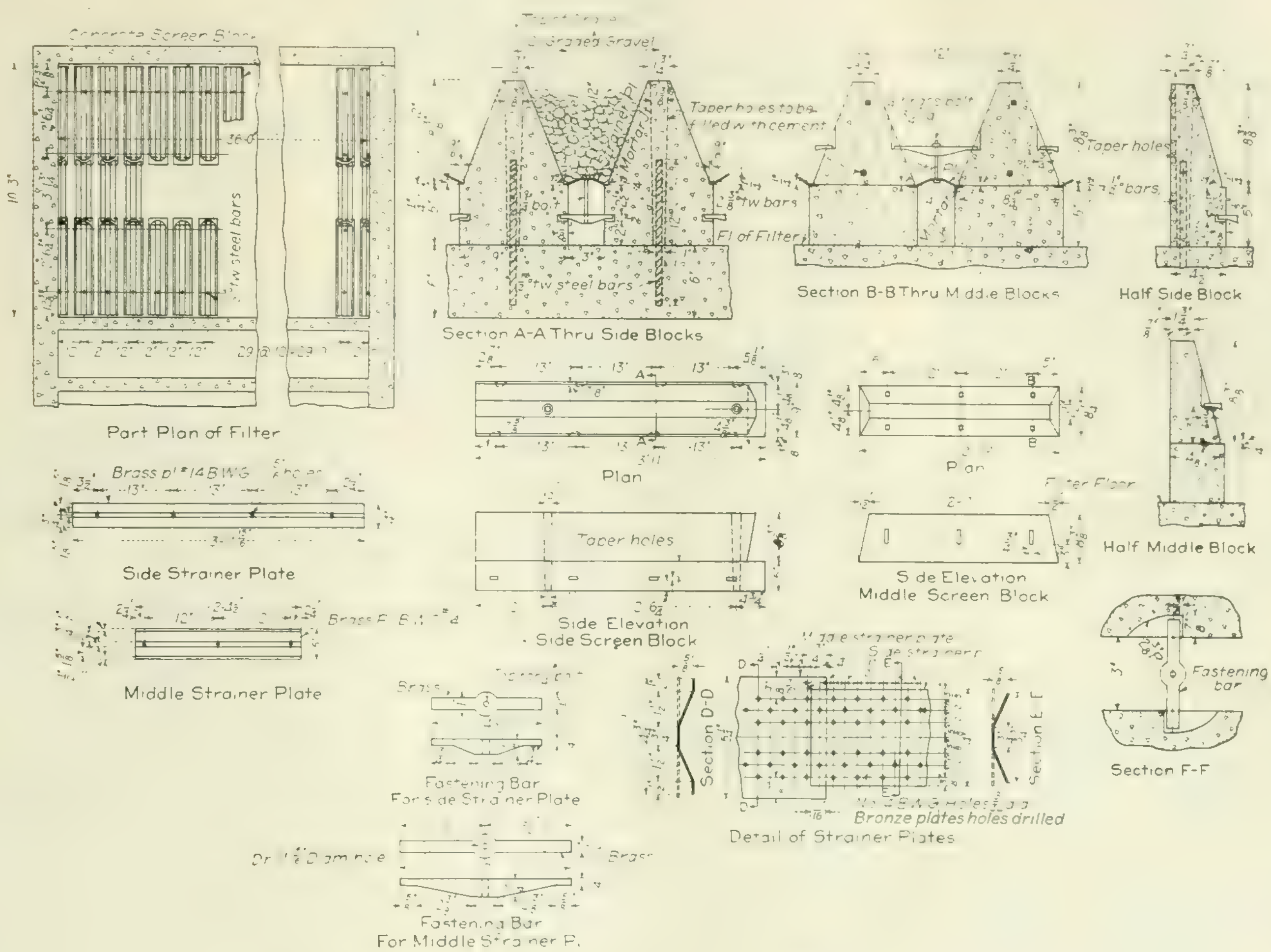


Fig. 2. Details of Strainer System of Evanston, Ill., Filters, Designed for High Rate of Wash.

strainer systems will be obtained from Table I, which gives the velocities under various conditions for three types of design. Type A in this table is for the design shown in Fig. 5, the low rate of wash with the separate air system. Type B is a similar design of low rate wash system using the Williamson strainer. Type C is a high rate of wash system without any air supply.

HYDRAULIC VALVES.

Under present practice the various filter operating valves used for controlling the operations of the filters are almost invariably hydraulically operated. For this service it is rather desirable to specify a double-gate parallel-seat valve, as wedged-seat valves for this type of service are more likely to stick. Valves should be bronze-fitted throughout, and the operating cylinders should be bronze lined. Operating pistons can to best advantage be fitted with double-cup leathers. In determining the size of the operating cylinder, good service may be obtained by making the cylinder of such size that under normal conditions the total effective moving force on the piston is equal to the total pressure on the valve disc, assuming this in no case to be less than 30 ft. This is roughly equivalent to the assumption that the coefficient of friction of the disc, including the friction of stuffing boxes and piston packing, is approximately 100 per cent.

It is very desirable that the rate of wash water shall be capable of adjustment as determined by trial and that a limit stop to the wash water valve be provided. A design by the writer of such a limit stop is shown in Fig. 6. In some places such stops may to advantage be replaced by a rate controller in the water supply line.

CHEMICAL MIXING TANKS.

Chemical mixing tanks are usually constructed of concrete for sulphate of alumina, iron sulphate, or hypochlorite of lime, and of steel plate for lime and soda. The size will be determined from consideration of the amount of chemical to be handled, the strength of the solution, and the length of time for which the tank is desired to serve. For instance, assume a plant of 10,000,000 gals. daily capacity, with sulphate of alumina used at the rate of 1.5 grains per gallon. Assume that the solution will be kept at a 2 per cent strength under normal conditions and that there should be provided two tanks each of which will run four hours. The sulphate of alumina used in four hours will amount to 358 lbs. and the tank must have a working capacity of 2,150 gals. If a little more attention is considered permissible and more frequent changes of tank, the tank may be accordingly made smaller for the shorter run.

Stirring apparatus must be provided, hand-operated for very small tanks, and power-operated for very large tanks. For these larger tanks the stirrer can to advantage be a small screw propeller at the bottom of the tank, with a suitable deflecting funnel operated by a vertical shaft and a small Pelton wheel. It is rather desirable, and not particularly costly, to provide an individual Pelton wheel drive for each solution tank or other piece of apparatus of this nature. For stirrers for lime tanks, best satisfaction is obtained by providing in the bottom of the tank

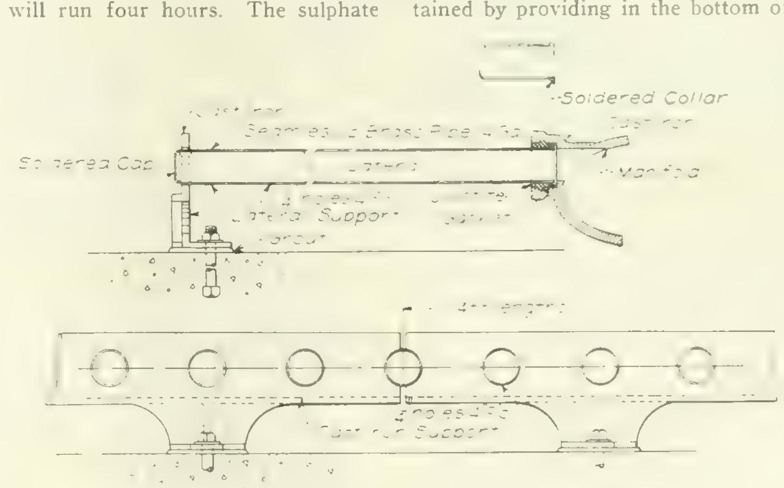


Fig. 3. Study for Strainer System of Baltimore Filters. This Design, Although Not Used, Shows Pipe System for High Rate of Wash.

a number of runs of iron pipe perforated with small holes to blow in a small amount of air through the solution. A design for a hypochlorite stirrer, used by the writer for Montreal, is shown in Fig. 7.

For dissolving hypochlorite of lime, which has a tendency to stick, special mixers must be used. A design by the New York Continental-Jewell Filtration Co. is shown in Fig. 8.

Where lime is used in the operation of the filters, it is usually received in the form of quicklime and must be slaked before being mixed with large quantities of water. To get efficient results it is necessary to slake with a relatively small quantity of water and even this water is sometimes preheated to permit the slaking at a sufficiently high temperature. The slaker may be in the form of a tank with suitable agitating or mixing devices. A concrete mixer of standard design can be used for this service. One type of slaker designed for the Croton filters is shown in Fig. 9.

CONVEYING MACHINERY.

The handling and conveying of the chemicals used is a big subject in itself and is only one branch of the large problem of material handling. No single form of machinery is suitable for all conditions and there are fully half a dozen different styles which will give the best of satisfaction when properly installed. However, for small plants there is not much doubt that an ordinary platform elevator with storage for barrels or boxes on the floor is the most efficient and satisfactory arrangement. For larger plants economy demands that the chemicals be stored in bins and handled in bulk. For such large plants clam shell buckets, traveling cranes, elevators, scraper or bucket conveyors, screw conveyors, and other forms, may all be used if proper attention be paid to details. The general preference should always be, where feasible, for the simpler forms and particularly for those that are self-contained and offer a smaller number of parts for deterioration. It is worthy of note, too, that power consumption is in practice a negligible item, and that simplicity and reliability of service and freedom from repair must be the controlling factors in the selection of the material-handling machinery.

The dust question is a serious one in handling such materials as lime, soda, or sulphate of alumina, as the dust is very disagreeable to the workmen engaged in the place. A certain amount of dust is inevitable, but the design of the machinery should be such as to minimize the dust that may be spread, and it is essential that suitable ventilating systems be provided to furnish free air to remove the dust. The ventilating system should be capable of changing the air in the general chemical rooms where chemicals are freely handled not less than six times per hour, and in special places a liberal additional supply should be provided.

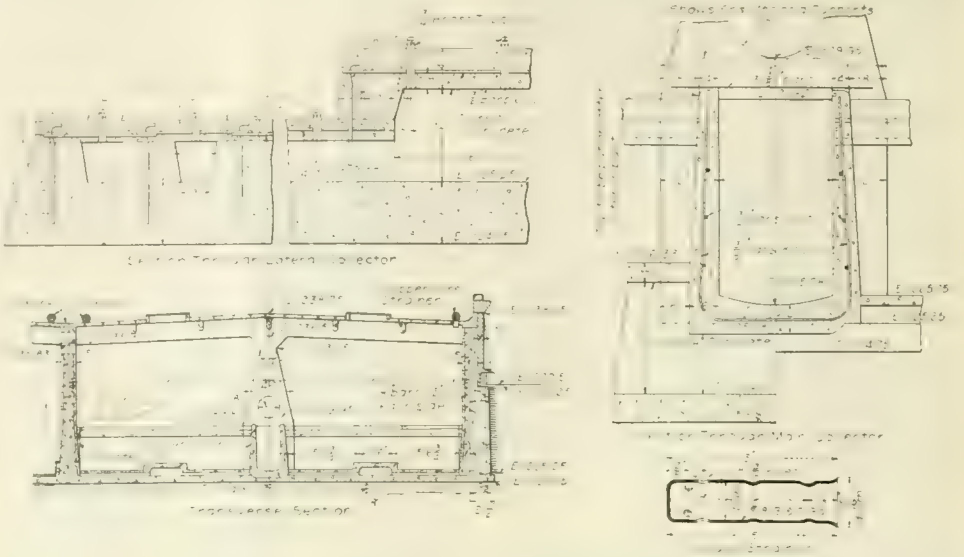


Fig. 4. Transverse Section of Pre-Filter of Queen Lane Filters, Philadelphia, and Sections Through Main and Lateral Collectors. Detail of Brass Strainer. Illustrates Concrete Ridge System for Low Rate of Wash.

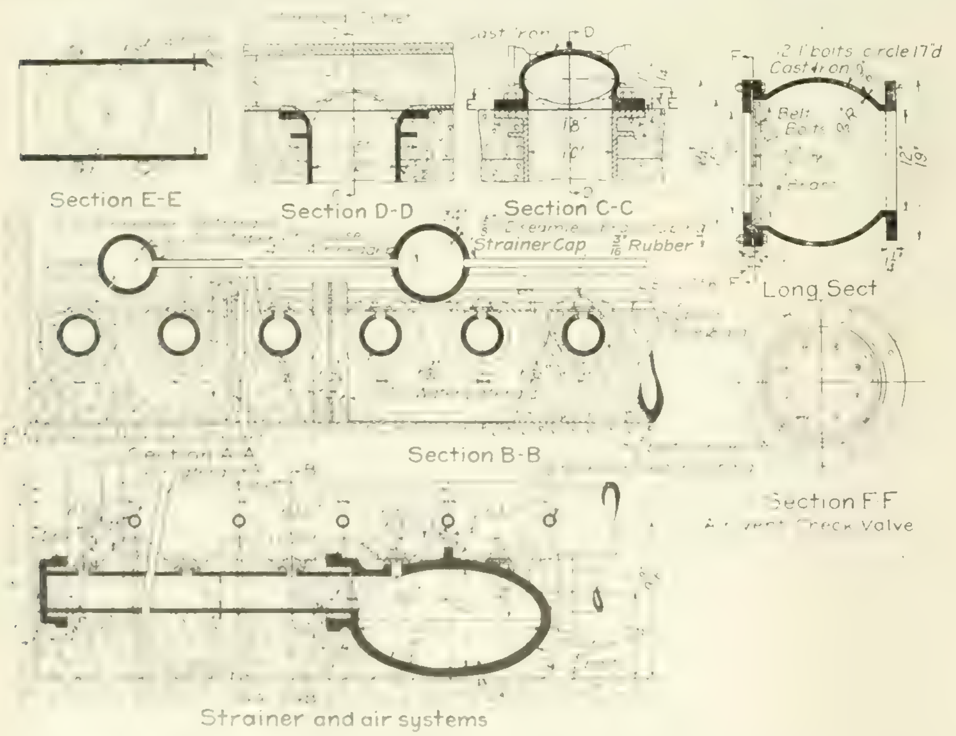


Fig. 5. Details of Strainer System with Separate Air System Designed for Jerome Park Filters, New York City—Designed for Low Rate of Wash.

CHEMICAL SOLUTION LINES.

The handling of chemical solutions to the point of application of the chemicals is probably one of the most difficult features in filter operation. Many of the chemicals, such as sulphate of alumina, sulphate of iron, and hypochlorite of lime, are corrosive, except to special materials which are high priced. Others, such as lime, have a very strong tendency to form incrustations or scale on the walls of the pipes, which will rapidly reduce the diameter of the pipes to a point where insufficient material can be discharged. For the handling of such materials as sulphate of alumina, it is desirable to use pipes either of hard-rubber composition for the first choice, or of lead or special bronze. There has recently been put on the market a new material known as "Duriron," which is an especially hard form of cast iron found in practice to be practically unaffected by sulphate of alumina. It is, however, so hard that it cannot be machined, and the proper running of a line with fittings is accordingly rather expensive. In the design of such chemical lines, it is essential that all changes in direction should be made by means of plugged tees or crosses and only straight runs of pipe be permitted between these points, so that all pipes can be readily cleaned. Ample connections for flushing with clean water and for draining must also be provided at many points. For the handling of lime, ordinary iron pipe can be used, as this is unaffected and even protected by the action of the lime. It is not practicable to prevent scale forming on the inside of the pipe, and all pipes must be duplicated and arranged so as to be easily accessible for renewal and cleaning. It will be found that if the lime solution be kept as cold as possible the troubles from deposition of lime scale will be sensibly reduced.

In some cases it is necessary to use chemical pumps, and these are subjected to the same annoyances as chemical solution lines. Sulphate of alumina pumps will be made to best advantage of Monel metal.

In many cases it will be found to advantage to use water, eductors or ejectors for the handling of chemical solutions. These eductors have the double advantage of being of ideal simplicity of form and of providing additional

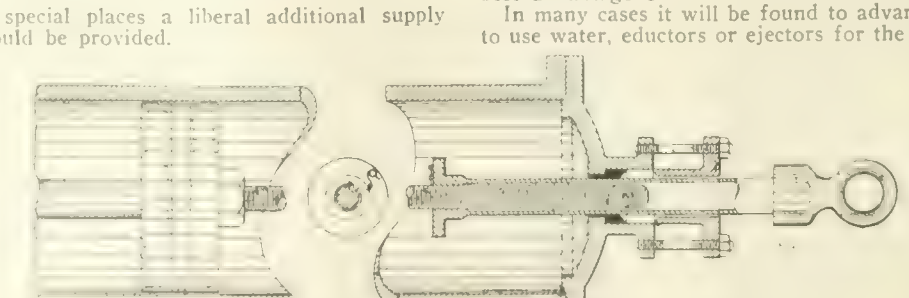


Fig. 6. Details of Adjustable Stop for Wash Water Valves.

diluting water in the chemical pipes which will reduce the objectionable action of the chemicals. The proportions of the eductors can be varied practically at will for handling

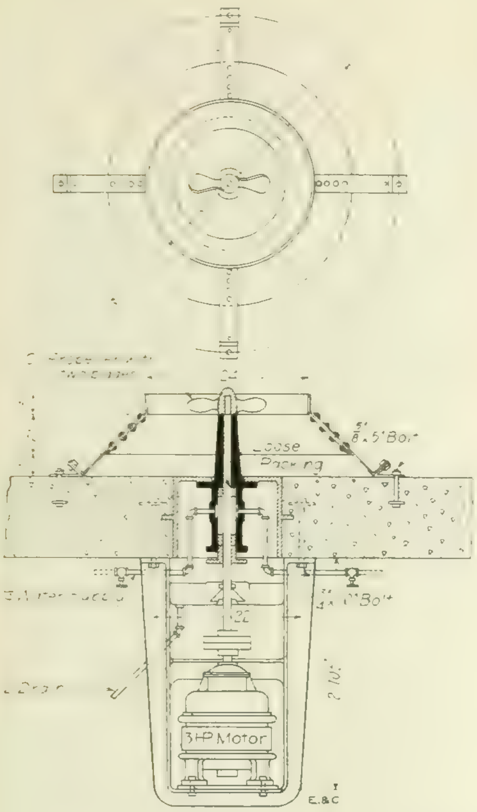


Fig. 7. Details of Agitator for Hypochlorite Solution Tanks, Designed for Montreal, Quebec, Filtration Plant.

solutions against different heads and with different working water pressures. The relations between these variables can be expressed by the formula

$$\frac{q}{Q}=\frac{\sqrt{H}}{h}-1$$

where q equals the quantity of solution to be pumped, h the head to which the solution is to be pumped, Q the quantity of pressure water used, and H the available head of the pressure water; thus, if the pressure water is available at 200 ft. head, and the solution is to be pumped against 40 ft. head, one volume of pressure water will handle 1.22 volumes of solution.

WATER PUMPS.

Pumps used in connection with filtration plants are almost invariably of the centrifugal type and are nearly always used in pumping raw water to the coagulating basin. Other pumps are used for pumping wash water. The most desirable form of pump has practically become standardized in the shape of a horizontal split casing, of as few stages as possible, according to the pressure. Because of the uncertainty and frequent variability of the head against which the pump is to work, it is considered best to use the plain volute type of pump instead of the turbine type, as better adapted to a varying range of heads. Impellers are made of hard bronze, shafts are bronze covered where in contact with water, and all stuffing boxes are double with water seals and are bronze-lined. As a matter of efficiency it is usually desirable to select a speed as high as possible. The possible speed will, of course, vary according to the patterns and designs of the manufacturer. A rough way of getting preliminary information as to a reasonable speed of a pump may be the following:

The peripheral speed of the impeller may be approximately represented by the relation $v=kx\sqrt{h}$, where v equals peripheral velocity of the impeller per second, k the constant ranging from 7.5 to 11, h the dynamic head in feet. For ordinary filter conditions of low-lift pumps, k may be taken approximately as 10. Given, then, the head, the peripheral speed is determined and from this a rough estimate of the rotative speed can be made. And to put this into figures, assume a pump is to deliver 10,000,000 gals. daily against a head of 36 ft. Such a pump, if of the double-suction type, would have a discharge opening of probably 18 ins., and each suction opening handling 5,000,000 gals., may be 14 ins. A 14-in. suction opening and an 18-in. diameter impeller may be about as small as it may be desirable to build. This will have a peripheral speed of 4.7 ft. for each revolution per second. The peripheral speed for 36 ft. of head will be 60 ft. per second, and from this we can roughly deduce that the pump should make somewhere in the neighborhood of 12.8 revolutions per second, or about 750 revolutions per minute.

BLOWERS.

Where the air wash system is used, it is necessary to provide an air blower. This will usually be designed to deliver air against 4 lbs. pressure and to deliver 4 cu. ft. of free air per square foot of filter surface. In other words, 1,440 cu. ft. per minute of free air for a 1,000,000-gal. filter. The blower is usually of the positive rotary type, having two geared impellers, cam-shaped, so that the varying space between the impellers and casing discharges the air. As these blowers usually are to give intermittent service, it is fully warranted to run them at relatively high speed. It is reasonable to figure approximately 85 per cent mechanical efficiency for these blowers and 85 per cent volumetric efficiency for small sizes, although these efficiencies may be somewhat increased for the larger sizes.

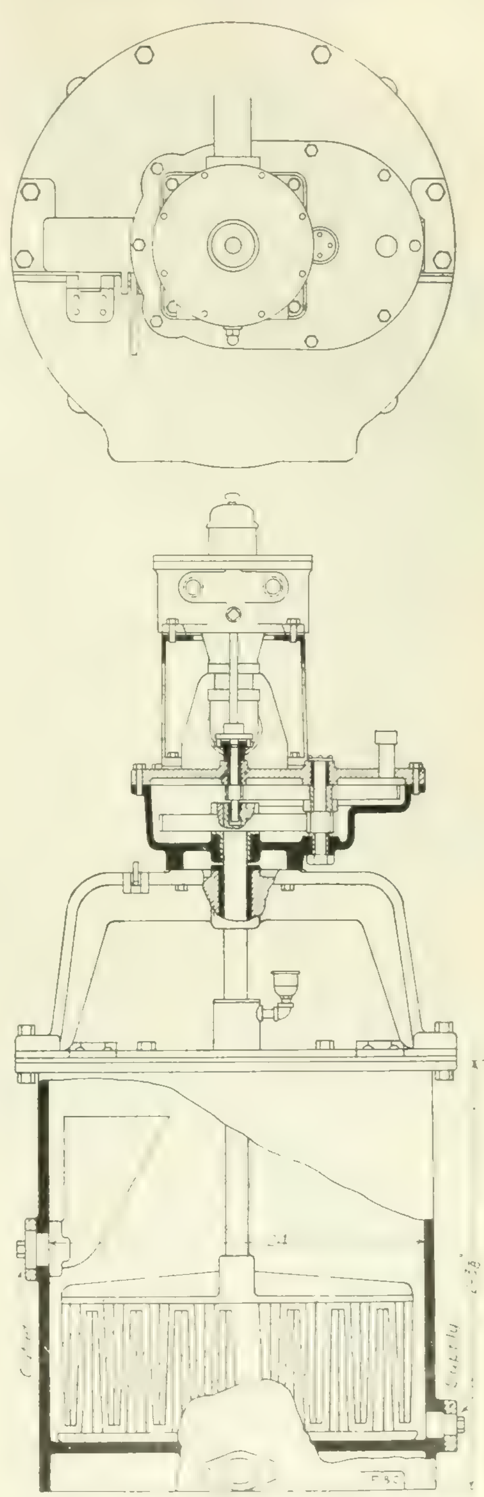


Fig. 8. Details of a Design of Special 24 in. Motor-Driven Mixer for Dissolving Hypochlorite of Lime.

It has been found economical at some plants, such as at Wilkes-Barre, Pa.; Trenton, N. J., and Montreal, Canada, to use a storage reservoir of the gas-holder type to hold both wash water and washing air, thus permitting the use of small, continuously-operating pumps. At other places, such as Columbus, O., and Panama, pressure tanks for air under high pressure are used.

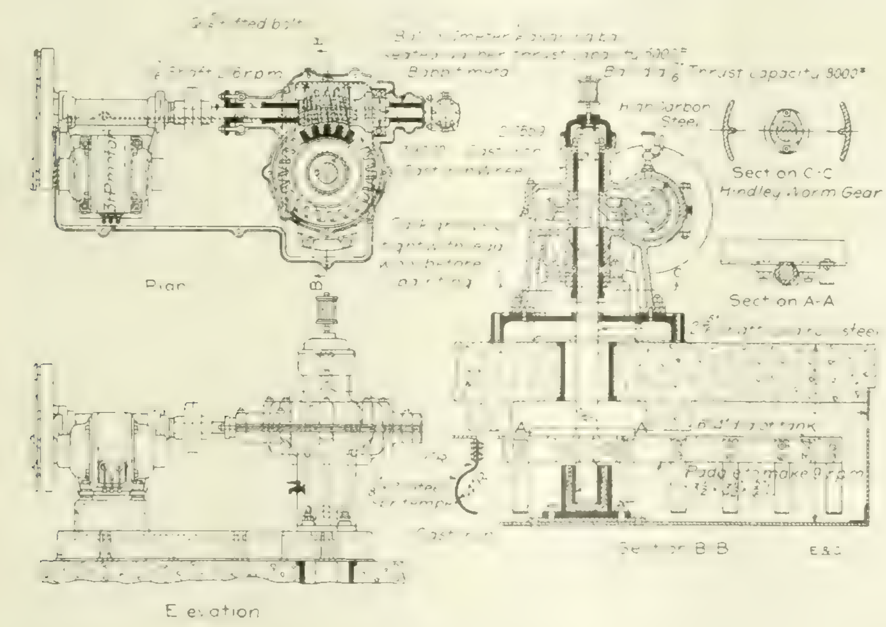


Fig. 9. Details of Stirrer for Lime Slaking, Designed for Jerome Park Water Filters, New York City.

Probable Utilization of Privately Owned Water Works at San Francisco in Connection With New Municipal Supply.

In former articles published in this journal relative to the new water supply of San Francisco it has been stated that the city will acquire the works of the Spring Valley Water Co., which is now supplying the city with water, and augment this supply with the new one from Hetch-Hetchy. This article relates primarily to the ultimate utilization of the present privately owned works in the new scheme for the water supply of the city, and is based on a recent address by the city engineer, Mr. M. M. O'Shaughnessy, before the Engineers' Club of San Francisco.

At intervals since 1873 the city has endeavored to acquire the properties owned by the Spring Valley Water Co. The present city administration has been trying for two or three years to place the acquisition of these properties before the citizens.

Many difficulties attend the supplying of water to a city situated as is San Francisco. The city lies at the head of a very narrow peninsula to which water must be brought a considerable distance. Possibly the only city similarly situated and confronted with the same physical conditions is New York. But New York is fortunate in having in its immediate vicinity a country with an average annual rainfall of 45 ins. whereas the average rainfall in San Francisco is less than 22 ins. and therefore the conservation of large bodies of water in storage reservoirs to overcome dry periods is an imperative necessity in San Francisco. This condition, of course, also makes the delivery and distribution of water more expensive, the retail prices of water in San Francisco now averaging about 21 cts. per 1,000 gals. This compares favorably with New York where the retail rates are 16 cts. per 1,000 gals. and where so far, they have gone only a comparatively short distance to get the present supply of 350,000,000 gals. per day. They are now constructing works 100 miles long to the Ashokan system to get an additional 500,000,000 gals. per day.

Up to the time of the earthquake and fire, San Francisco was possibly the most prosperous municipality in the United States, having practically no bonded indebtedness. It, however, owned no public utilities such as a water supply, street railways, municipal docks or wharves, its only properties being schools, fire and sewer systems. Since that catastrophe, the city has been compelled to incur the enormous outlay of possibly $25,000,000 to $30,000,000. Restoring the destroyed streets cost about $8,000,000, restoring the sewer system and school and fire houses; purchasing a civic center; building a city hall; and constructing the high pressure fire fighting system, which cost $6,000,000 and is acknowledged by all critics to be the best in the world. This burden has been heavy and besides the catastrophe of fire and earthquake San Francisco also had the misfortune of being run by an undesirable lot of politicians, so that a house cleaning has had to take place, causing with factional conflicts a still heavier burden on the city. The city, however, never lost sight of the necessity of acquiring a water supply. San Francisco is the only large city in the United States, with the exception of Denver, which does not own its own water supply. The authorities are duly impressed with the great importance of solving this question, and besides making provision for the future water supply they are also endeavoring to make provision for the present immediate water supply.

Since the new charter of 1900 when water rates were reduced, the Spring Valley Water Co. has made little extension to its system. It has ceased laying large distributing pipes through the city and the pipes now laid in the outlying district are less than 2 ins. in diameter. In some portions of the city due to lack of water pipes, the condition is serious, such as in the Richmond section where a population of 10,000 people is fed [illegible] from [illegible] pipe. There are miles and miles of asphalt streets in that section with property worth hundreds of dollars per front foot dependent on one water service pipe. There are always two sides to every question. The claim of the citizens is that the water company has been charging too high rates. The water company's contention is that they have not been getting enough revenue; and possibly, there is always a middle ground which is the right ground, but the practical result of this controversy compels the tax payers to pay the burden of legal expense annually incurred over rate contests.

In Washington last year, when the city's representatives were appealing to the national government to give the city the Hetch-Hetchy grant for a future source of supply, assurances were given Congress by the city officers that we propose to acquire the Spring Valley system. In pursuance of this policy, a condemnation suit was filed against that company last December to acquire all of its property that was necessary and useful in connection with a future Sierra supply.

With that end in view I compiled a report at the request of the supervisors, delineating all the properties that were necessary for a municipal supply, and that included about 1,000 acres of land in the city and county of San Francisco, and about 67,000 acres outside of the city. This list includes 823 acres around the shores of Lake Merced, and the latter, together with eleven other reservoirs comprise the service reservoirs within the city. The list also includes all the lands, rights of way, pumping plants, and reservoirs in the county that are useful for a future supply. Certain lands have been excluded, such as the Coyote Creek and Searsville, which are positively of no value to the city, and the effect of excluding those lands has been to lower the purchase price of the Spring Valley properties so that the attorneys for the city were able to get the attorneys for the Spring Valley to agree to accept $34,500,000 for all the properties described in this condemnation suit. This is about $3,000,000 less than the price last demanded by the company. The company asked $37,500,000, about 18 months ago, and the city offered $37,000,000. Since that offer, about 30,000 acres of land have been excluded and the Spring Valley offered a portion of it to a water company in San Jose for about $150,000, but it is absolutely of no value to San Francisco for a future water supply.

Sixty-seven thousand acres of land outside of San Francisco has very substantial value—possibly $100 an acre is a fair average to put on it. Some of the lands inside of the city, such as the Ingleside tract of 43 acres, is worth at least $3,000 an acre, as well as the sites of the University Mound, Holly Park, and Laguna Honda Reservoirs, so that the lands which the city is acquiring are of a very substantial value.

At the present time the water system is developed up to 40,000,000 gals. per day. About 20,000,000 gals. is taken from the peninsula system of San Francisco, comprising the three reservoirs of Crystal Springs, San Andreas, and Pilarcitos with a combined capacity of 30,000,000 gals. The Crystal Springs dam can be increased at a moderate expense to nearly double its reservoir capacity by raising the dam about 50 ft. in height. The constructed portion of the dam is of sufficient base to be able to stand that raise without any great expense. The capacities of Alameda, Pleasanton, Calaveras and San Antonio sources are now 20,000,000 gals. daily.

At the present time the company has under construction a hydraulically filled dam at Calaveras intended to be 200 ft. high. The base is about 1,310 ft. wide and is now constructed, and an outlet culvert 20 ft. in diameter has been built to carry away the freshet waters so that the dam will not be destroyed during construction. The city practically assured the company a return on this item of outlay provided they went ahead and built this dam. About a year ago last August the Spring Valley started and made good progress until about January or February of this year. Since that time they have been proceeding slowly, due perhaps to the condemnation suit. If this proposed purchase is agreed upon the Spring Valley will probably proceed actively with the construction. Naturally, they are a timid corporation, susceptible to external influences. With the completion of the Calaveras dam, that reservoir with a capacity about equal to the combined capacity of all the three peninsula reservoirs, nearly 30,000,000,000 gals., will add to the present supply about 20,000,000 gals. a day more; so that with a comparatively moderate expense at least 25,000,000 gals. per day can be taken out of the undeveloped portions of the Spring Valley.

The revenue list of 1912-1913 shows that the total revenue of the Spring Valley was $3,292,000, operating expenses $840,775, and taxes $116,000, making a total of $1,221,000, which makes a net revenue of $2,071,000 to meet interest on bonds, stock dividends, and depreciation. At $34,500,000 the proposed purchase price, on a basis of 4½ per cent the interest charge would be around $1,552,000 yearly, and deducting that from the net of $2,071,000 would leave $518,000 for a profit, including the depreciation. The depreciation may safely be estimated to be $250,000 to $300,000 a year on a plant of this magnitude, so that even at the present prices from the present figures and under city administration I believe a small profit can be made at the present sale price. I figure about $218,000 a year.

The entire distributing system of Spring Valley inside the city is about 568 miles, of which 424 miles, or 74 per cent is cast iron pipe which has been in for 22 years, 11 per cent is wrought iron riveted pipe which has been in for 27 years, and 133 miles small pipe with an average of 10 years, which confirms my statement that the policy of the Spring Valley for the past ten years has been to put in very small pipes so as to get the commodity to the consumer at a minimum of expense and defer capital expenditure for larger pipe. Of course, this situation would be changed under a city ownership, as the city would put in pipes of a larger diameter, none less than 6 or 8 ins., and map out a more generous policy toward consumers.

There are two alternatives at hand for the city. One is to buy Spring Valley and the other is to cease all negotiations and go ahead at once with the complete construction of the Hetch-Hetchy system. All estimates, including Mr. Freeman's, figure about $40,000,000 to the city boundary line and with the necessary distributing reservoirs, land, pipe lines, and pumping stations, inside the city, will cost $12,000,000 or $14,000,000 more, making $54,000,000 or $55,000,000 for the water supply and its delivery to the consumers in the city, exclusive of interest during construction.

If this alternative is adopted the city will have to pay interest on the entire $55,000,000 until the day of first delivering water. Eighty thousand taps now exist and the change from the private company to the public company will be slow and prolonged and there will be a tremendous economic waste during the transition.

The policy of the city administration is, however, a broad one; and it is endeavoring to solve this question on its business merits and treasure no resentment on account of past controversies.

The plan is to acquire the company for about $34,500,000, spending about $8,000,000 on the completion of the Calaveras dam and advancing construction towards the Hetch-Hetchy system, reducing the total cost and gradually developing the present system. Forty millions of gallons is to 65,000,000 of gals. as $3,200,000 is proportioned to $5,200,000 so that the purchase of this system on this plan and developing it progressively, using receipts from sales of water, will make no burden on the taxpayers of San Francisco. In the meantime the city will proceed to develop the mountain water supply, build the Hetch-Hetchy dam, 20 miles of aqueduct and develop 30,000 HP. as a first unit of hydroelectric power which will help to operate municipal railways. The whole purpose of this policy is to keep the tax rate down and help acquire other public utilities and proceed in a sound and reasonable manner with the financial policy of the city.

The tax rate in San Francisco is $2.30 on a

50 per cent valuation; Los Angeles, $3.19; Oakland, $2.97; San Diego, $3.95; Sacramento, $4.17; Pasadena, $3.90; Stockton, $4.04. In fact, the tax rate of San Francisco is but one-half of what it is in the other cities of the state. This will show that, notwithstanding our political administration, San Francisco is handling its business matters in a sound manner.

ROADS AND STREETS

Methods and Cost of Road and Trail Construction in Alaska.

I.

The development which will come in Alaska with the construction of railways will bring the necessity of extensive construction of roads. It is evident that the conditions affecting this road construction will for a long time be quite different than those in the States and that there must be corresponding differences in methods of construction and in structural features. These facts lend exceptional interest to an article on road and trail construction in Alaska which appears in "Professional Memoirs" for April-May, 1914. The author of this article is Maj. F. A. Pope, U. S. Engineer Corps, who was a member of the Alaska Road Commission. His description is unusually complete in matters of outfitting forces for Alaskan construction and in matters of methods and costs of work in high altitudes.

The different kinds of construction undertaken have been classified by the Board of Army Engineers in charge, under four general heads: Wagon roads, winter sled roads, trails, and staked trails. In this case the greater does not necessarily include the lesser. Wagon roads are not necessarily available as sled roads or even as trails under some winter conditions. The term "wagon road" is applied to roads intended for use by double teams and wagons carrying considerable loads the year round. They are given suitable grades and are crowned, ditched, drained, and corduroyed or planked where necessary. Winter sled roads are for use of double teams and sleds in winter. They are cleared and graded where necessary, but not surfaced or drained. Trails are of two kinds—for single-horse sleds, which differ from winter sled-roads only in width and in the care put in their construction, and pack trails. Staked trails are either temporarily staked each winter with poles or laths and cloth streamers, or permanently marked with iron stakes to point out the trails and keep people from getting lost in treeless and windy places where trails become quickly obliterated by the drifting snow.

The standard of wagon road adopted was a good earthen highway, planked or corduroyed where necessary. Although such roads are apt to become badly cut up in rainy weather or during the spring thaws if traffic is heavy, this was the most that the board felt justified in attempting on account of the vast extent of territory to be covered, the expense of construction, the very limited funds available, and, in most places, the limited traffic. At the best, it has been impossible, from lack of money, to build roads in many places where they are badly needed. In some cases the standard was departed from where traffic was exceptionally heavy or exceptionally light.

Nearly all work was done by day labor, as it was almost impossible in most cases to do it in any other way. Few men could be found who had the experience, money, and inclination to do the work by contract. On account of depending on taxes collected from time to time, and on uncertain appropriations, it was difficult to arrange for work in advance. The character of construction was such as to make it impossible, as a rule, to determine quantities and conditions with sufficient accuracy for a contract. On account of the shortness of the working season, the locating parties were often followed closely by the construction crews in order to do the work the same year.

The following general instructions were issued for guidance in location work:

WAGON ROADS.

2. The locator should most carefully cruise and examine the ground, covering all possible routes, select the best, and then proceed to lay out his line on the ground. Preliminary surveys will be made only in exceptional cases.

3. The first consideration must always be to select the line giving the cheapest road that will fulfill the purpose. Frozen or swampy ground must be particularly avoided. Side hill cuts on very steep slopes and lines involving considerable rock work will be avoided where possible. Deep cuts and fills will not be laid out.

4. Grades. The grades should be kept as low as possible, without making the road unnecessarily expensive. As a general proposition, however, it will be found that low grades involve heavy cost of construction, on account of either extensive grading or wet ground encountered. The maximum grades allowable

Grade lines should follow the natural slope of the ground, taking advantage of favoring benches, etc.

Grades should never be guessed at, but must be measured.

5. Curves. Sharp curves should be avoided if possible, but where they can not be avoided the line must be level. The minimum curvature is of 50-ft. radius. Unless the radius is at least 100 ft., the maximum grades should not be used. Curves need be laid out only where the line crosses spurs or ravines, or at the approaches to bridges, or to sharp, steep grades, etc. As a rule, in crossing small ravines, etc., it will be preferable to use a straight, steep grade rather than a sharp curve.

As stated above, the line will in general follow the natural curve of the ground, without reference to tangents and circular curves.

6. In general, in addition to the above requirements, wagon roads should be laid out

Fig. 1. Map Showing Roads and Trails in Alaska.

will be used if by so doing the cost of construction will be reduced.

The following are the maximum grades:

Short grades, 100 ft. or less in length, with easy grades at both ends, may be 15 per cent (9 degrees) if straight and with straight approaches.

Medium grades, 300 ft. or less in length, with easy grades at both ends, may be 10 per cent (6 degrees) if there are no sharp curves either on the grade or approaches.

Long grades, over 300 ft. in length, uphill for loaded wagons as freight is usually hauled, should be kept down to 6 per cent (3½ degrees) if possible, with reasonable expenditure. They may be as much as 8 per cent (5 degrees), but no more, if a less grade can not be obtained with reasonable expenditure.

For heavy traffic downhill, or for light traffic in general, the grade should be kept to 8 per cent, if possible with reasonable expenditure. It may be increased to 10 per cent, but no more, if a lower grade can not be constructed at a reasonable cost.

with a view to their winter use; that is, they should have proper protection against drifting snow. (Paragraph 10.) There are cases where, however, the wagon road will not be followed in winter, as, when winter travel will cross some level swamp; and in any event, the line for the wagon road should not be placed on wet ground for considerable distances merely to fit it for winter travel.

7. Marks. The location of the line will be marked by stakes set every 100 ft., and mile-posts will be established at the end of each mile. Mile-posts should be measured from the generally accepted center of the town of departure.

8. Record. The Board is required by law to prepare maps of each road. To this end, the locator will take the necessary magnetic bearings and distances, with such side notes as may be necessary, and record them in such manner that they may be plotted without further explanation from him. At that point of beginning and at every 20 miles thereafter, he will make

and record an observation to determine the variation of the compass.

SLED ROADS.

9. Grades, etc. The grades on sled roads will depend on the traffic, to a more considerable extent than on the wagon roads.

If the traffic is expected to be heavy enough to keep the trail well broken, the grades should be at least as low as the maximum specified in paragraph 4 for wagon roads, and as much lower as possible. Easy grades should never be sacrificed to put the line on dry ground.

If the traffic is expected to be light so that difficulty is to be anticipated on account of snow, the grade may be increased to as much as 15 per cent to keep out of drifts. This will be particularly applied in crossing divides, where the main line should be located as to cross with the least side-hill cutting possible.

10. The first requisite for sled roads is proper protection from wind and drifting snow. With this end in view, they should be located in timber if possible, but never close to the edge of timber. High, bare ridges should be avoided, but the tops of such ridges are preferable to a location just below the crest of the slope. The foot of steep, bare slopes is a particularly bad place for a sled trail.

11. While giving first attention to protection. at the same time as dry ground as can be had should be selected for the line.

12. The line of the sled road will be staked, not blazed, and mile-posts will be established. It is not necessary, however, to set the stakes at 100-ft. intervals.

13. The located line will be surveyed and recorded by the locator in the same manner as a wagon road (Paragraph 8), but with less detailed notes.

PACK TRAILS.

14. Pack trails should be laid out to secure dryness of ground, easy crossing of streams, and easy construction. Grades up to 25 per cent or 30 per cent may be freely introduced to gain these results. The location of pack trails will be recorded simply by general compass bearings and estimated distances.

These instructions were necessarily varied somewhat in special cases, and it was found that the best location for a wagon road was not often best for a sled road, and vice versa. Paragraph 6 was therefore changed accordingly. These instructions are given here complete, as they involve conditions not met with in the "States."

SOUTHEASTERN ALASKA.

As conditions in southeastern Alaska are quite distinct from those in other parts of the territory, a brief description of the work in this section will be given separately.

The conditions do not vary greatly from those in some parts of the "States," the main difference being due to the extremely heavy rains. With few exceptions the whole country is heavily timbered and with dense underbrush. It is very hilly and rocky, with few level valleys or benches. Rains are excessive in summer and snow in winter. Wherever level country exists it is apt to be swampy, if not properly drained. The surface of the ground is usually covered with a thick layer of decaying vegetation, often several feet thick. However, supplies can be delivered by boat within a very few miles of any part of this whole district. Freight rates are rather high on account of dangers to navigation from fog or snow, but land transportation is over such short distances that the cost of transportation in general is not very serious on road work. Away from the roads, of course, the country is almost impassable, but this does not affect the cost of road construction greatly, although it increases largely the difficulty in preliminary surveys and location. Labor in general was paid about $2.50 per day with board. Although supplies and labor are much cheaper, owing to the great amount of work, the number of bridges to be put in, the cost of clearing and grubbing, and the excessive rains, road work in southeastern Alaska is, as a rule, the most expensive in the territory. Maintenance is also very expensive. To quote an example of the conditions under which work must be done here in the summer of 1906, on a road on Prince of Wales Island, out of 126 working days during the season, it rained hard 100 days, with a slight drizzle and cloudy 10 more days, leaving but 16 days of clear weather for work. Of course, under such conditions men work in the rain, but their efficiency is much reduced. There are but few features of the work in this district of interest and a description of only a few roads will be given.

The Prince of Wales Island portage road referred to above is the most expensive road built in Alaska. It consists of 3.9 miles of wagon road and 7 miles of pack trail. The wagon road portion was cleared 24 ft. wide. Corduroy and culverts were made 13 ft. wide. It was finally found necessary to plank a very large portion of the road, the excessive rains making it almost impossible to retain any other kind of surface. Planks made from timber shipped from Seattle were found to be much cheaper than corduroy cut on the ground; this was partly due to the difficulty of getting timber of suitable size. The wagon road portion cost approximately $7,915 per mile, due to the extremely bad conditions already described. Rains were almost continuous throughout the working season. Timber cost $14 to $14.50 per thousand delivered on the shore at either end of the road. Corduroy cost $2.18 per foot. Planking, $1.26 per foot. Clearing, $700 per mile. Grubbing and grading in earth, $1,895 per mile. These costs are approximate only and do not include superintendence or surveying. Labor was about $2.50 per day with board. Supplies were purchased in Ketchikan and brought over in small launches.

A road connecting Haines with the Chilkat River at Hindestucki—3.04 miles—was advertised for contract, but as no bids were received it was built by day labor. It was graveled throughout and would compare favorably with the ordinary macadamized roads built in the more settled portions of the "States." The remainder of the road is a specially good earthen road, although parts are graveled. Supplies were carried up the river and delivered at the various camps in war canoes with Chilkat Indian boatmen.

In general, the width of clearing through the timber was made 40 ft. The clearing was classed as heavy where the prevailing timber was spruce or cottonwood, and light where the prevailing timber was alder or brush. The roadway was graded to a width of 24 ft.; all moss and decaying vegetation being removed from the surface, and stumps and roots being removed to a depth of 6 ins. below the surface. Culverts were constructed where needed; they were built of sound spruce logs at least 6 ins. in diameter at the smaller end, except floor logs which were allowed to be somewhat smaller. Abutments were made of log cribbing notched together with tie logs running into the embankment at least 5 ft., and the sill sunk deep into the ground to prevent undermining. Three stringers were placed with 5-ft. centers. The width of the waterway was required to be at least 6 ft., and the height 3 ft. The flooring was constructed of poles or split logs at least 4 ins. thick at the small end, hewed or adzed to a level surface. The width of culverts was 12 ft. Logs were placed over the outer stringers and pinned to them, each with two wooden pins 20 ins. in diameter wedged in by drift bolts ⅝ in. in diameter. Many of these culverts were placed, but it was found that they usually became filled with solid ice during the winter and that unless the ice was chopped out in the spring before the thaw they would not be of much benefit.

Bridges were constructed where necessary across small streams. The abutments were built as for culverts. Piers were made of spruce log cribbing, triangular in plan, pointing against the current 18 ft. in length and 8 ft. in width at the larger end. The cribs were filled with rock. Stringers were of spruce, three in number, and of the following dimensions: Spans 10 to 15 ft. wide, 10 ins. middle diameter; spans 15 to 20 ft. wide, 12 ins. middle diameter; spans 20 to 25 ft. wide, 14 ins. middle diameter; spans 25 to 30 ft. wide, 16 ins. middle diameter. The flooring was the same as in culverts. This, of course, does not include the large truss bridges built over rivers.

On wet ground which could not be properly drained, corduroy was used. The corduroy was made of sound logs or poles not more than 10 ins. in diameter and not less than 2 ins., and at least 12 ft. in length. They were laid close together in one or two horizontal layers, depending upon the diameter of the poles, at right angles to the direction of the road, and covered with soil, not muck, from the ditches to a depth of 12 ins. at the center and 6 ins. at the sides. When, by reason of the scarcity of large trees, it was found necessary to use saplings of 2 or 4 ins. diameter, the longitudinal layer was first placed upon the ground and the cross-wise layers placed upon top of this. Log cribbing retaining walls were built of spruce at least 6 ins. in diameter at the tips. Face logs were laid in horizontal courses with a batter, depending upon the special conditions. Tie logs running at least 6 ft. into the embankment were set at intervals of not to exceed 10 ft. between every two courses. The top course was pinned to the top logs as for culverts. The height of cribbing was limited to 8 ft. In some cases plank was imported from the States and used in place of logs in bridge or culvert work.

The unit prices under the contract portion were (1908) as follows: Heavy clearing, per acre, $125; light clearing, per acre, $100; grubbing, per acre, $50; earth excavation, per cubic yard, 38 cts.; rock excavation, per cubic yard, $1; corduroy, per linear foot, $1; cribbing, per linear foot, $1.50; bridges, per linear foot, $5; culverts, $10 each.

A bridge built across the Chilkat River at Wells consisted of two 108-ft. trusses with approaches 100 to 300 ft. in length. No new or unusual difficulties presented themselves in its construction. Work was done by contract at a cost of $8,620.08. The material was brought from Seattle by boat to Haines and hauled up the road for a distance of 24 miles to Wells.

A foot suspension bridge built at Sitka is of interest chiefly because it was broken by tourists crowding on it and swinging it for amusement.

The roads constructed in southeastern Alaska are necessarily, on account of the country being cut up to such an extent, local roads; except the road in the Chilkat Valley from Haines, which, in conjunction with Canadian roads leading from its terminus, forms one of the main routes to the interior.

The main part of Alaska lying west of the 141st meridian contains the great mass of roads built by the board. It is also the part which differs most from other countries and in which most of the new and unforeseen conditions differing from those met with outside were encountered. Of course many things, such as the high cost of labor, freight and supplies, and some of the difficulties of transportation, exist in all frontier countries.

The conditions along the coastal strip differ but little from those in southeastern Alaska. Labor and supplies cost some more, but the work is the same. The only work of importance in this section is the coast portion of the Valdez-Fairbanks road, which will be described later.

Road work around Cook Inlet is, as a rule, the cheapest in Alaska, as the extreme conditions met with in the southeastern part and in the interior are lacking. There is no permanent frozen ground as in the interior, and the excessive rains common to the coast portion are lacking. A good deal of the country is fairly level; the underbrush is light, and while there is a fairly good stand of timber it is not as heavy as in the southeastern part. Where there is no timber a good growth of grass is usually found, except, of course, in the high mountains above the timber line. Kenai Peninsula is mountainous, but, as a rule, there are good passes for the construction of roads and trails. In the mountainous parts of this section travel during the spring thaw, or even in the winter, after a series of warm winds from the ocean, is apt to be dangerous on account of avalanches; the mountains being, as a rule, very steep, and the valleys quite narrow. The snow fall in the mountains is very heavy.

Supplies are brought to Seward by boat the year round or to the trading posts on Cook Inlet or Susitna River in summer. They are usually distributed by sled in winter. Travel in summer is usually on foot and sometimes horseback, and in winter by dog-sled or snow shoes. Freight, when hauled in summer, is usually taken in with pack horses. A number of roads and trails have been constructed, but they offered no special difficulty.

On the Moose Pass sled road, constructed in the Kenai Peninsula, grubbing and clearing cost about $137 per acre; side hill grading 40 cents per yard, or about $1,300 per mile; corduroy from 50 cents to 80 cents per foot. The width of the road was 8 feet. This cost was about the lowest in the whole country.

A number of roads and trails have been constructed from Cook Inlet, Seward, and the Alaska-Northern Railroad toward the interior, and to different mining camps in the vicinity. These, however, will not be described, as they offered nothing particularly difficult or unusual in their construction.

One of the main roads to the interior leads from the end of the Alaska-Northern Railroad through Knik, Susitna, Rainy Pass, and the mining camps on the Iditarod River. As most of this route lies in the interior, it will be described later.

OUTFITTING.

In all that part of the "North Country" to the north of the Coast Range conditions are in some ways similar, or, to be more accurate, I should say that although they differ widely in different sections, these differences are so small compared with the difference between what one finds in any one section and what one is accustomed to on the "outside," that they appear very similar. It is in this district that the conditions peculiar to an Arctic or semi-Arctic climate are met with. The climate becomes gradually colder to the northward and more raw to the westward, but there is no decided break like that met in crossing the Coast Range.

Traveling in the cold in the North, while not by any means pleasant, is very far from being accompanied by the extreme hardships and danger which one is led to believe in from the harrowing tales of would-be "heroes," told for the edification of their friends or for publication in the newspapers. With proper clothing and food there is little danger if reasonable precautions are taken. The sufferings, sometimes resulting in death even, of Polar expeditions are almost invariably due to starvation or improper food on account of getting so far away from their base of supplies. Of course, accidents are more apt to occur where the ground is covered with snow or ice than on solid ground, but to travel an equal distance away from your base, even in a temperate climate, if absolutely dependent for food and fuel on that base, would be difficult and dangerous. If properly clothed and fed almost any degree of cold can be endured if there is no wind. I have suffered much more from cold at a temperature of 10° below zero with a light breeze blowing off of the Valdez glacier than from 63° below in the interior with no wind. The chief difficulty below—50° is in breathing the cold air. The sensation at —60° is a good deal like that in breathing the rarefied air at an elevation of 14,000 ft. These extreme temperatures are rather uncommon, although the temperature sometimes drops to —70° or lower. Below —50° the difficulty in breathing makes it troublesome to move rapidly or to keep warm by violent exercise. Personally, I should prefer traveling in winter to enduring the mosquitoes and gnats and wading through the tundra in summer.

Food should be rich in carbon and plentiful, as appetites are enormous. For a trip bacon, hot cakes and syrup and beans are best, particularly the first two, as no other kind of food appears to have so great a sustaining power. Tea is carried partly because it is lighter and easier to make than coffee, but particularly because it is much better for prolonged exertion, especially in the cold. To drink alcoholic stimulants either before starting or while on a trip is simply to commit suicide, as no man is strong enough to resist both the cold and liquor at the same time. The hardest drinker in the north would about as soon take poison as alcohol under such conditions.

Clothing in winter has to be carefully regulated. Too much clothing will make one perspire and then freeze almost as quickly as too little, if exercising. Bathing should be fairly frequent if possible, to keep the pores of the skin open, thus helping the natural heat of the body to keep the surface warm. The face should be kept shaven, as otherwise ice from the moisture in the breath will form on the beard and freeze the face. The following is a list of the clothes I wore in winter, which is a fairly good guide, although the requirements of different people differ and each must judge for himself; underwear of medium weight wool; flannel shirt, service or equivalent; medium weight winter suit; if on foot, mackinaw shirt instead of coat and vest; light khaki trousers as overalls; fur cap with silk top, as solid fur is too warm; heavy woolen mittens or gloves with leather mittens or gloves on the outside to break the wind; three pairs woolen socks, medium weight, next the foot, outside of these an extremely heavy pair, outside these a still heavier pair of German socks; moccasins, or, if riding, felt shoes with overshoes; if riding, a fur overcoat, otherwise a parki of khaki cloth with fur-lined hood. Fur should be wolverine, as it is the only kind that will not collect moisture from the breath and freeze. A parki is practically a large shirt worn outside the trousers and extending to the knees, as a wind shield. For some distance back from the Bering Sea and north of the Arctic mountains, where the climate is more raw and penetrating, fur parkis are used. Stiff collars and leather shoes are never worn except in town. Moccasins should be provided with a stiff sole, unless one is accustomed to them, as the arch of the foot, ordinarily supported by the sole of a leather shoe, will otherwise soon give way and become very painful on a rough trail. Colored glasses should be carried to protect the eyes from the glare of sunlight reflected from the snow.

Summer clothing is light, as the summers are fairly warm. I wore canvas, service leggings to protect my trousers from the underbrush and to keep out mosquitoes, and leather gloves to protect the hands from mosquitoes and gnats. After many experiments, I finally adopted the U. S. quartermaster department marching shoe issued in 1908, the best shoe I have ever found for walking under all sorts of conditions. Head nets must always be worn to protect against mosquitoes. They should be black in front, as other colors reflect the light and interfere with the vision. The number, size and persistency of Alaskan mosquitoes exceeds anything I ever saw in a temperate or tropical swamp. They appear as soon as even a small patch of ground becomes bare of snow in the spring and they are found in undiminished numbers from the Coast Range to the Arctic Ocean. Gnats are, if possible, more troublesome in places, but they appear only for a short time in the fall.

A large proportion of the interior is broken and rugged. The valleys and level portions have usually a dense underbrush. Where the ground is frozen, scrubby spruce timber prevails; where not frozen, birch and poplar. Most of Seward Peninsula is treeless and there is little timber outside of the river valleys. The ground, as a rule, is permanently frozen to a great and unknown depth, the surface being protected from thawing in summer by a thick blanket of moss or turf—the "tundra" of Siberia and the North generally. On account of this freezing, water from melting snow and rain is prevented from sinking, and, the evaporation being slight, any level ground is quite sure to be covered with water or shallow swamp in summer. Aside from this, the moss blanket prevents thawing more than a few inches in the brief summer. Cutting up this layer with wheels or horse hoofs lets in warm air and rain and, melting the ice, quickly forms an impassable quagmire. Travel over the tundra is, at best, difficult, and over the level and swampy places described above, almost impossible. For these reasons most travel, except on the rivers and where roads have been built, is in winter. Supplies are carried in summer by boat to the different places along the rivers and thence, as a rule, by sled in winter to their destination. The ice on the rivers is generally used when possible for travel where sled roads have not been built, and, on account of the easy grade, often for freighting where roads have been built. However, the ice is apt to be broken and rough, the open river is often swept by winds, and the frequency of open water makes river travel dangerous. Travelers and mail carriers always, and freighters usually, will use a poor road rather than the river.

Open water, often met with, is very dangerous. To get wet away from camp and without a change of clothing is to freeze to death. For this reason, the one indispensable thing that must be carried is extra socks. Other articles may be necessary for comfort, cleanliness, or common decency, but the extra socks are often the price of a man's life. Just why open water should be found in the very coldest weather is not clear. Of course, warm springs exist in places, but this does not explain why water should appear in some places only in the coldest weather. A possible explanation is that when a stream freezes solid in places clear down to the frozen subsoil, the line of least resistance for any water flowing along the bottom is up through the ice, which cracks under the hydrostatic pressure and lets the water through.

Mules are never used, as their small hoofs cut through the snow in winter and the tundra in summer. Large horses are used, even for packing, as the cost of feeding them is but slightly greater than for small horses, and they can do so much more work. Dogs are used only where horses can not be. They are indispensable away from the beaten trails. The native Malamutes and Huskies and the Siberian Wolf Hounds are, for most purposes, the best, as being natives of the North they know how to take care of themselves. They are, however, part wolf and very treacherous, the wolfish nature asserting itself at times. Snowshoes are, of course, used where the snow is too light to carry one otherwise. The snowshoes used in the interior are much larger than those used on the coast, as the snow is light and powdery. Skis are used to a limited extent where there is a crust or a beaten trail for speed, or comfort, or sport. Below—50 degrees the snow does not soften readily under the friction of the runners of a sled so as to let it slip along, as it does under warmer temperatures. The sled grates and slides along much as though going over sand, making traction much heavier. This and the difficulty of breathing, before referred to, make travel slow at low temperatures.

The scale of wages for common labor was, with a few exceptions, as follows: In the Valdez district, which included the basin of the Copper River and the Valde-Fairbanks Road at first to the summit of the Alaskan Mountains and later to the Tanana River, $3.50 per day, less the cost of the provisions at Valdez—about 50 cents per day. This included the time going to and from Valdez. In the interior and on the Bering Sea coast, except around Nome, $5.00 per day and board. This sometimes included the time going to the place of work and sometimes not, depending on the circumstances. Provisions at the bases of supply along the navigable rivers cost roughly from $1.00 to $1.20 per day more. Around Nome, $4.00 per day and board. Provisions were not much more expensive at Nome than at Valdez. Labor, as may be seen, was expensive, but very efficient, as weaklings do not thrive in the North.

Game, particularly moose and caribou, and fish, particularly salmon, were plentiful, and some working parties had regular hunters and depended entirely on them for meats. The cost of all kinds of supplies and equipment was high on account of the freight rates.

There is little rain, and daylight lasts for twenty-four hours throughout most of the working season. A common practice is to call baseball games at midnight, as such announcements look very attractive and impressive in the home newspapers.

The equipment used was usually such as might be expected for building a road by hand, supplemented in a few places by horse graders. Mosquito tents were used for sleeping. These tents had a canvas floor continuous with the rest of the tent, forming what was really [illegible] mosquito bar for ventilation. Men lived quite comfortably in canvas tents, even in the coldest weather. Under such conditions, they usually sleep in [illegible] similar material lined on the inside with wool or fur, or preferably eiderdown, on account of its warmth and lightness. If caught out in the open at night [illegible] unless a fire can be built. In case of need, it is possible to sleep fairly comfortably by burrowing into the snow and sleeping in sleeping bags.

(To be Continued.)

More than 4,000,000 sq. ft. of asphalt is being laid on streets of the Panama-Pacific International Exposition [illegible] sq. ft. per day.

Repair and Maintenance Costs of Park Roads in Boston, Mass., in 1913.

The itemized cost of repairing and maintaining the roads and drives under the supervision of the Boston Metropolitan Park Commission in 1913, J. R. Rablin, engineer, is stated in the last report of the commission. The work was accomplished for the most part by contract. Table I gives a detailed statement of the cost.

TABLE I.—SUMMARY OF COST OF ROAD REPAIRS AND MAINTENANCE IN BOSTON, MASS., PARKS IN 1913.

Length (ft.)	Width of roadway (ft.)	Square yards	Labor (cts.)	Gravel (cts.)	Broken stone (cts.)	Sand (cts.)	Kind of material	Gallons per sq. yd.	Cost (cts.)	Cost of applying (cts.)	Total cost (cts.)	Total amount	Remarks
1,610	26 and 30	4,962	15.12	.....	3.82	.....	Tarine (American Tar Company)	.650	5.09	.....	24.03	$1,192.55	Patching and surface treatment.
7,700	36	30,800	2.06	.....	.....	.....	Asphaltic oil (the Texas Company)	.219	1.18	.....	3.24	999.54	Surface treatment.
2,550	8	2,267	1.90	.....	1.11	.....	Tarine	.295	2.42	.....	5.43	123.01	Surface treatment.
1,900	26	5,489	1.43	.....	.....	.....	Asphaltic oil (the Texas Company)	.219	1.18	.....	2.61	143.55	Surface treatment.
2,600	36	10,400	.53	.....	.....	.....	Asphaltic oil (the Texas Company)	.173	.93	.....	1.46	152.20	Surface treatment.
1,500	16	2,667	.67	.....	.....	.....	Asphaltic oil (the Texas Company)	.112	.61	.....	1.28	34.20	Surface treatment.
6,500	16	11,556	1.26	.....	.....	.....	Asphaltic oil (the Texas Company)	.208	1.12	.....	2.38	275.60	Surface treatment.
8,500	16	15,111	2.42	.....	.....	.....	Asphaltic oil (the Texas Company)	.278	1.50	.....	3.92	591.80	Surface treatment.
5,700	16	10,133	.72	.....	.....	.....	Asphaltic oil (the Texas Company)	.118	.64	.....	1.36	137.80	Surface treatment.
220	45	1,100	9.77	.....	15.38	.....	Tarvia (Barrett Manufacturing Company)	1.423	11.38	.....	36.53	401.90	Reconstruction.
1,880	45	8,573	1.68	.....	2.10	.22	Tarvia	.337	2.70	.33	7.03	602.93	Surface treatment.
9,200	40 and 60	55,550	.45	.36	.....	.....	Calcium chloride	...	1.57	.....	2.38	1,324.47	Surface treatment.
9,300	13	13,500	1.17	.....	.....	.....	Asphaltic oil No. 4 (Standard Oil Company)	...	1.50	.....	2.67	358.94	Surface treatment.
2,700	40	12,000	2.18	.83	.....	.....	Asphaltic oil No. 4	...	1.50	.....	4.51	541.88	Surface treatment.
7,700	34, 36, 40	31,510	1.10	.....	.....	.....	Asphaltic oil No. 1 (the Texas Company)	.231	1.30	.....	2.40	758.07	Surface treatment.
55,715	32-40	24,053	.64	.....	.....	1.75	Asphaltic oil No. 6 (Standard Oil Company)	.291	1.68	.44	4.51	1,084.07	Surface treatment.
2,975	40	11,972	1.00	.....	.94	.....	Tarvia	.275	2.34	.....	4.28	512.41	Surface treatment.
3,200	30-40	11,667	.....	.....	3.65	.....	Tarine	.264	1.85	.20	5.70	665.50	Surface treatment.
1,500	36	6,000	1.76	.....	2.47	.....	Tarine	.267	1.73	.26	6.22	372.98	Surface treatment and patching.
11,720	24 and 40	37,345	1.27	.90	.....	.....	Tarine	.526	4.50	.....	6.67	2,493.51	Surface treatment.
525	28	1,633	26.64	.....	30.23	.....	Tarine	2.310	17.36	.....	74.23	1,212.12	Reconstruction.
1,650	36	6,800	20.80	.....	27.10	.....	Tarine	2.580	19.40	.....	67.30	4,573.60	Reconstruction.
400	26	1,156	2.08	.....	1.16	.....	Tarine	.674	5.04	.....	8.28	95.73	Surface treatment.
250	36	1,000	69.75	11.30	39.76	1.25	Tarine	2.268	15.50	1.35	138.91	1,389.26	Reconstruction.
4,620	36	18,489	3.38	.....	1.93	.....	Tarine	1.800	1.37	.....	6.68	1,234.26	Patching and surface treatment.
1,540	26	4,450	1.45	.....	...	.45	Tarine	.270	2.16	.....	4.06	180.25	Surface treatment.
1,560	36	6,240	1.69	.....	.36	.....	Tarine	.150	1.14	.....	3.19	199.25	Patching.
2,400	36	9,600	1.13	.....	.52	.....	Tarine	.040	.27	.....	1.92	184.77	Patching.
1,740	36	6,960	.22	.....		...	Tarine	.030	.21	.....	.43	29.00	Patching.
1,500	36	6,000	2.13	.....	1.10	.....	Tarine	.420	3.16	.....	6.69	401.89	Patching and surface treatment.
410		2,740	8.20	.....	10.50	.....	Tarine	1.860	13.98	.....	32.68	897.49	Reconstruction and surface treatm't.
1,450	63	5,800	1.27	.....	.....	1.78	Asphaltic oil No. 6 (Standard Oil Company)	.414	2.38	1.27	6.70	388.85	Surface treatment.
4,615	26	13,332	1.74		1.56	.97	Asphalt binder A (Standard Oil Company)	.379	2.61	1.19	8.07	1,076.60	Surface treatment.
3,900	25	10,445	1.36			1.49	Asphaltic oil No. 6 (Standard Oil Company)	.370	2.13	.63	5.61	586.23	Surface treatment.
3,704	36	14,236	1.38		1.62	..	Tarvia	.253	2.15	.38	5.53	787.20	Surface treatment.
3,100	18	6,200	3.17	.....	.....	1.68	Asphaltic oil No. 6 (Standard Oil Company)	.415	2.39	.68	7.92	491.03	Surface treatment.
2,150	18	4,300	1.03			3.66	Asphaltic oil No. 6 (Standard Oil Company)	.467	2.69	.61	7.99	343.64	Surface treatment
1,700	18	3,000	.63			1.31	Asphaltic oil No. 6 (Standard Oil Company)	.461	2.65	1.92	6.51	195.37	Surface treatment
956	18	1,912	.59			1.29	Asphaltic oil No. 6 (Standard Oil Company)	.434	2.49	1.79	6.16	117.76	Surface treatment.
1,250	20	2,778	35.01	21.23	52.05	.....		.....	.....	.....	108.29	3,008.35	Reconstruction.
1,100	20	2,445	14.10		24.00		Asphalt binder A (Standard Oil Company)	1.600	16.10	.....	54.20	1,323.76	Reconstruction.
2,870	17	5,420	17.20		36.72		Asphalt binder A (Standard Oil Company)	1.700	15.22	.....	69.10	3,745.46	Reconstruction.
1,784	17	[illegible]	17.79		41.1		Asphalt binder A (Standard Oil Company)	1.744	12.92	5.30	103.02	3,471.80	Reconstruction.
		6,000	1.17		1.08		Tarvia	.280	2.34	.....	4.59	276.08	Surface treatment.
3,760	40	16,600	.68	.....	.....	.....	Tarvia B	.477	4.52	.23	5.43	904.22	Surface treatment.
2,750	26	7,944	.92				Asphaltic oil (the Texas Company)	.208	1.12	.....	2.04	162.10	Surface treatment.
1,000	36	4,000	.97		1.11	.....	Tarine	.250	1.90	.....	3.98	158.13	Surface treatment.
6,030	40	25,000	1.44		[illegible]		Tarine	.350	2.36	.....	4.16	1,039.07	Surface treatment.
4,200	40	16,772	.82		.74		Tarine	.300	2.29	.....	3.90	654.57	Surface treatment.
10,975	16	19,511	1.40			.18	Asphaltic oil (the Texas Company)	.185	1.24	.....	2.82	550.65	Surface treatment.
2,100	16	3,734	1.80		...	.32	Asphaltic oil (the Texas Company)	.320	2.36	.....	4.48	166.88	Surface treatment.
1,200	18	[illegible]	.78			.07	Asphaltic oil (the Texas Company)	.210	1.47	.....	2.32	199.28	Surface treatment.
1,100	26 and 36	[illegible]	11.80	1.41	6.60		Tarvia	1.906	16.20	.84	36.25	1,247.00	Resurfacing.

Asphalt-Paper Pipes.—Pipes made of asphalt paper are being introduced in Austria [illegible] uses, and numerous advantages are claimed for them by the manufacturers. Originally the paper pipes were employed for electric-cable conduits, but other uses have now been found for them, and the makers assert they can replace iron, steel, copper, and clay piping for all purposes except the conveyance of hot fluids, concentrated acids, and petroleum products. The pipes are made from a special kind of asphalt paper of German manufacture (the paper made in Austria not being wide enough), are exceedingly light in weight compared with metallic or clay piping, and, [illegible], are flexible to a slight degree. They are not affected by sudden changes of temperature, nor do the stray currents of electric-tramway systems cause electrolysis of buried mains.

Proposed Standard Terms for Bituminous Road Materials.

At the recent meeting of the American Society for Testing Materials the committee on standard tests for road materials submitted a tentative report, a portion of which was referred back to the committee. Part of this committee's report referring to road terminology is given here:

Asphalts.—Solid or semi-solid native bitumens, solid or semi-solid bitumens obtained by refining petroleum, or solid or semi-solid bitumens which are combinations of the bitumens mentioned with petroleums or derivatives thereof which melt upon the application of heat, and which consist of a mixture of hydrocarbons and their derivatives of complex structure, largely cyclic and bridge compounds.

Asphaltenes.—The components of the bitumen in petroleum, petroleum products, malthas, asphalt cements and solid native bitumens, which are soluble in carbon disulphide but insoluble in paraffin naphthas.

Blown Petroleums.—Semi-solid or solid products produced primarily by the action of air upon originally fluid native bitumens which are heated during the blowing process.

Carbenes.—The components of the bitumen in petroleums, petroleum products, malthas, asphalt cements and solid native bitumens which are soluble in carbon disulphide but insoluble in carbon tetrachloride.

Cut-back Products.—Petroleum or tar residuums which have been fluxed with distillates.

Tars.—Bitumens which yield pitches upon fractional distillation and which are produced as distillates by the destructive distillation of bitumens, pyrobitumens or organic materials.

Coal Tar.—The mixture of hydrocarbon distillates, mostly unsaturated ring compounds, produced in the destructive distillation of coal.

Coke-oven Tar.—Coal tar produced in by-product coke ovens in the manufacture of coke from bituminous coal.

Dehydrated Tars.—Tars from which all water has been removed.

Gas-house Coal Tar.—Coal tar produced in gas-house retorts in the manufacture of illuminating gas from bituminous coal.

Oil-gas Tars.—Tars produced by cracking oil vapors at high temperatures in the manufacture of oil gas.

Pitches.—Solid residues produced in the evaporation or distillation of bitumens, the term being usually applied to residues obtained from tars.

Refined Tar.—Tar freed from water by evaporation or distillation which is continued until the residue is of desired consistency; or a product produced by fluxing tar residuum with tar distillate.

Water-gas Tars.—Tars produced by cracking oil vapors at high temperatures in the manufacture of carburetted water gas.

Concrete Base Construction on California State Highways.

A large portion of the roads under construction by the California State Highway Commission are being built on a concrete base, which is surfaced with a thin bituminous carpet of asphaltic oil and screenings. A timely discussion of the methods and cost of constructing these thin bases by J. B. Woodson, division engineer, presented in the California Highway Bulletin, is given here:

TABLE I.—COSTS OF CONCRETE BASES 15 FT. WIDE AT VARIOUS PRICES PER CU. YD.

Depths. Ins.	Cubic yds. per mile.	Cost per mile.	Cost per sq. foot.	Cost per sq. yard.
		$5.00 per cu. yd.		
4	978	$4,890	$0.062	$0.556
4½	1,100	5,500	.069	.625
5	1,222	6,110	.077	.694
5½	1,344	6,720	.085	.764
6	1,467	7,333	.093	.833
		$5.25 per cu. yd.		
4	978	5,135	.065	.584
4½	1,100	5,775	.073	.656
5	1,222	6,416	.081	.729
5½	1,344	7,056	.089	.802
6	1,467	7,700	.097	.875
		$5.50 per cu. yd.		
4	978	5,379	.068	.611
4½	1,100	6,050	.076	.687
5	1,222	6,721	.085	.764
5½	1,344	7,392	.093	.840
6	1,467	8,066	.102	.917
		$5.75 per cu. yd.		
4	978	5,624	.071	.639
4½	1,100	6,325	.080	.719
5	1,222	7,027	.089	.799
5½	1,344	7,728	.098	.878
6	1,467	8,433	.106	.958
		$6.00 per cu. yd.		
4	978	5,868	.074	.667
4½	1,100	6,600	.083	.750
5	1,222	7,332	.093	.833
5½	1,344	8,064	.102	.916
6	1,467	8,800	.111	1.000
		$6.25 per cu. yd.		
4	978	6,113	.077	.695
4½	1,100	6,875	.087	.781
5	1,222	7,637	.096	.868
5½	1,344	8,400	.106	.955
6	1,467	9,166	.115	1.042
		$6.50 per cu. yd.		
4	978	6,357	.080	.722
4½	1,100	7,150	.090	.812
5	1,222	7,943	.100	.903
5½	1,344	8,736	.110	.993
6	1,467	9,533	.120	1.083

Foundations.—The 4-in. base of Portland cement concrete, consisting of a 1:2½:5 mixture, if constructed upon a perfectly prepared subgrade, should afford sufficient strength to carry all traffic and any type of road vehicle now existing.

Except on alkali, quicksand or marsh-land soil, the 4-in. thickness is as good as 1 ft., so far as sustaining qualities are concerned. Upon unstable soil the pavement should be increased in thickness according to the degree of firmness of the subgrade.

Some soils make a firm subgrade, but warrant extra thickness of pavement. For example, adobe makes the hardest kind of subgrade if the clods are pulverized, but "wet streaks" in an adobe fill will, upon evaporation, have a tendency to settle locally; this in effect requires greater spanning power, which virtually means a heavier beam.

Under the heading of adobe should be classified alkali and dry bog. This latter, when thoroughly dry, resembles volcanic ash and the difference in its displacement under dry compression and when it is wet and heavy would produce an appreciable upward pressure under the pavement. All forms of adobe, however, have the cracking qualities in the dry season, and it is doubtful whether the extra thickness of concrete would prevent the cracks breaking through the pavement above.

Pavements constructed on a low grade line, over marsh-land or wet sandy soil, should be reinforced by increased thickness of concrete.

Placing.—Concrete for 4-in. pavement should be mushy, but not "sloppy." It should be dry enough to require considerable tamping before it looks wet on top. When the mixture flows out on the chute as if under pressure resembling a fire hydrant, the work should be stopped until the water feed can be adjusted. Some contractors claim that with a certain type of mixer the concrete will not discharge down the chute unless the mixture is very wet. This is a mistake, and when the water runs down the pavement in little rivulets floating away from the mortar, it will not be long under traffic until the rocks begin to break loose. The concrete should be tamped until individual rocks disappear in the mortar, and the final floating should leave the pavement smooth and true to section. The floating is done with a wooden float, and from 20 to 50 ft. in the rear of the placing of concrete. Some men soon become skilled in making a good finish.

A 10-ton roller is used in preparing the subgrade. Sufficient rolling is done to compact the rough grade firmly. The rough grade is left from ½ to 1 in. high to allow for compression.

When finished, the subgrade is theoretically parallel and 4 ins. lower than the desired elevation of the surface of the pavement. The section drops ⅜ in. per lineal foot each side of the crown, the bottom of header board being 2¾ ins. lower than the crown.

Curing.—When the concrete is sufficiently set, the time required varying from 6 to 12 hours, according to the weather, 2 to 4 ins. of loose dirt is thrown over the pavement and is kept wet for a period of six days, and on later contracts 10 days. During the hot weather, great care should be exercised to see that the concrete is moist under the covering of earth. A more satisfactory method of curing the concrete is by building little dykes about 20 ft. apart and along the edge of the pavement, and flooding the concrete twice a day, or oftener if needed. This latter method always shows whether the concrete is wet, is cheaper for the contractor, and is much more easily cleaned off.

Cleaning off the pavement is usually accomplished with a road grader; the dirt being thrown out on the shoulders, again shaped, and rolled.

An approximate cost, as shown by our records, of covering with dirt is 2.5 cts. per lineal foot, or 13.4 cts. per cubic yard of concrete. The cost of dyking with dirt is 1.25 cts. per lineal foot, or 6.75 cts. per cubic yard of concrete.

When all the dirt is taken off that can be removed with the road grader, horse brooms are run over a pavement. If the dirt still remains in patches, shovels and stable brooms are applied.

Cracks.—The natural transverse cracks appear about 28 ft. apart. This varies considerably, according to the materials used and the care in keeping the concrete wet. With all forms of cracking that have developed so far no serious trouble is noticeable in wearing of concrete or appearances, and the bituminous wearing surface enters the crevices and covers them until they are scarcely perceptible.

TABLE II. — COST OF 15-FT. CONCRETE PAVEMENT AT VARIOUS PRICES PER SQ. YD.

79,200 sq. ft. to mile; 8,800 sq. yds. to mile.

Per sq. ft.	Per sq. yd.	Per mile.
$0.001	$0.009	$ 79.20
.002	.018	158.40
.003	.027	237.60
.004	.036	316.80
.005	.045	396.00
.006	.054	475.20
.007	.063	554.40
.008	.072	633.60
.009	.081	712.80
.010	.090	792.00
.020	.180	1,584.00
.030	.270	2,376.00
.040	.360	3,168.00
.050	.450	3,960.00
.060	.540	4,752.00
.070	.630	5,544.00
.080	.720	6,336.00
.090	.810	7,128.00
.100	.900	7,920.00
.105	.945	8,316.00
.110	.990	8,712.00
.115	1.035	9,108.00
.120	1.080	9,504.00
.125	1.125	9,900.00
.130	1.170	10,296.00
.135	1.215	10,692.00
.140	1.260	11,088.00
.145	1.305	11,484.00
.150	1.350	11,880.00
.155	1.395	12,276.00
.160	1.440	12,672.00
.165	1.485	13,068.00
.170	1.530	13,464.00
.175	1.575	13,860.00
.180	1.620	14,256.00
.185	1.665	14,652.00
.190	1.710	15,048.00
.195	1.755	15,444.00
.200	1.800	15,840.00

Artificial construction joints do not seem to warrant the expense attached to them in the construction of concrete pavements in California. Where they have been used by means of redwood strips placed between strips of tar paper, the joint wears away under traffic and the regular crack appears at its natural interval. The cost of steel expansion joints is practically prohibitive for the benefit derived, and they do not eliminate the cracks. Steel reinforcing is not desirable for pavements, owing to the expense. It would seem more practical to increase the thickness of pavement over sections of doubtful subgrade.

Costs.—Out of five days' run on a contract near Fresno, at an average of 762 lineal feet of 15-ft. pavement, the following force was used:

Item.	Cost.
Foreman, 5 days at $4.00	$ 20.00
Labor, 5 days at $3.50	17.50
Labor, 6 days at $2.50	15.00
Labor, 132½ days at $2.00	265.00
Wagon and driver, 5 days at $4.50	22.50
Concrete mixer, fuel, etc., 5 days at $5.00	25.00
Water pump and engineer, 5 days at $6.00	30.00
Total cost of operation for 5 days	$395.00

Total length of run, 3,808 lin. ft., or 705 cu. yds. of concrete; 56 cts. per cubic yard.

On a cubic yard basis the cement cost $2.17, the stone cost 71 cts., and the sand cost 43 cts.

The average cost of unloading and transportation of material was 68 cts.

The total cost per cubic yard of concrete in place was $4.55.

Division Organization of the New York State Highway Commission.

(STAFF ARTICLE.)

The state of New York is divided for the purpose of road construction and administration into nine divisions, each division consisting of from four to ten counties. These divisions are in charge of division engineers appointed by the State Highway Commission. The division engineer supervises road and bridge work of all kinds within his division and is held responsible for the deportment and efficiency of subordinate employes. An assistant, known as the resident engineer, is pro-

Chart Showing Division Organization of the New York State Highway Commission.

vided. The resident engineer is next in authority to the division engineer and acts for him when necessary. Ordinarily he has supervision over survey work. The drafting work of the division is in charge of an engineer who is subordinate to the division and resident engineer.

Contract construction work, repair gangs and patrolmen in each county are in charge of the county engineer, who is an employe of the state commission. The duties of the county engineer are of a supervisory nature. He visits each contract at least once a week and advises the resident construction engineer directly in charge of work as to questions which arise in connection with the progress of the work, examining their records and accounts to see that they are kept in a neat and workmanlike manner and performing other work of a similar nature. The county engineer reports daily to the division engineer upon the work he has inspected, the unusual conditions found, suggestions regarding inspectors and progress of any particular phase of construction, and the work to be inspected on the following day.

Painted Lines for Street Traffic Control.

An innovation in traffic control in Detroit consists in marking off special spaces across the streets at street crossings for the use of pedestrians. Lines are plainly marked with white paint in the positions indicated in the accompanying diagram. A space the width of the sidewalk is allowed, usually about 18 ft. Street cars and all vehicular traffic must stop at the first white line, the block system of traffic regulation being used. Pedestrians cross within the spaces marked. Signs are used freely for the information and instruction of traffic.

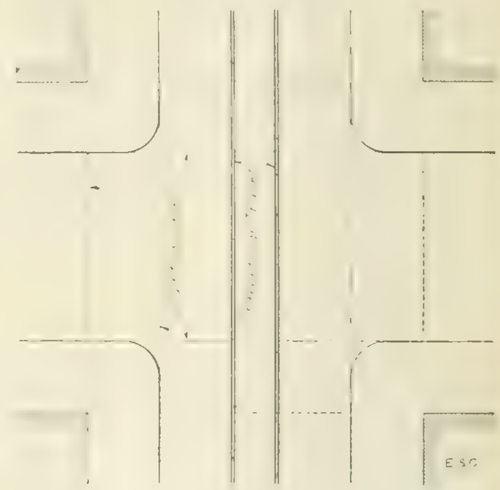


Diagram Showing Position of Painted Lines Used for Street Traffic Control.

To provide parking space for automobiles on certain streets a white line is marked at a distance of 8 ft. from the curb, the area within which is to be used for this purpose. Machines are parked obliquely.

CONSTRUCTION PLANT
MACHINES DEVICES MATERIALS

Cone-Shaped Delivery Applied to a Hand Bitumen Distributor.

A bucket distributor for bitumen having a cone-shaped device for spreading the material evenly is shown in the illustration. The design is quite simple. A heavy bucket with an outlet valve in the bottom, the stem of which [illegible] the rim of the

A Hand Bitumen Distributor Delivering Over a Cone Spreader.

bucket, has attached to it a sloping tray 18 ins. wide at the edge. On this tray is supported a half cone over which the bitumen flows onto the tray and thence to the road surface, distributing the material in an even film 18 ins. wide.

The bucket may either be fed by a hose, or may be filled by means of dipper, as in the case of the ordinary type of hand distributor. In operation, the bucket is filled with bitumen heated to the proper temperature, the gate valve opened and the material flows out, the man holding the bucket walking backward and forward to control the distribution. In using a hose, the end of the hose is screwed to the valve in the bottom of the pail and the bitumen flows directly onto the cone without filling the bucket.

An important feature of this device is the steady flow obtained. Also, the material is delivered close to the road surface, retaining a large proportion of its load.

The device was invented by S. Sutcliffe of Mytholmroyd, and is manufactured by the Patent Snowplough Co., Mytholmroyd, S. O., Yorks, England.

An Underslung Truck for City Refuse Removal.

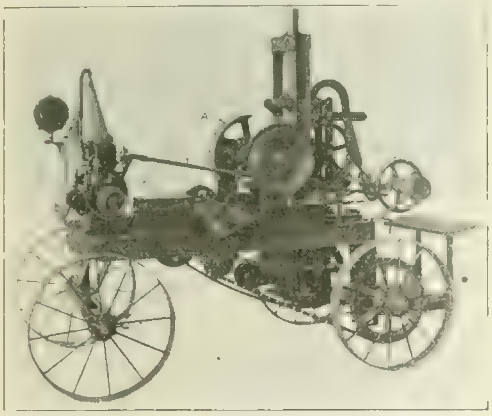
The main advantage in a truck of underslung construction for refuse removal lies in the comparatively low bed obtained. This feature is of importance in any vehicle that must be loaded from the ground. The advantage obtained by this type of construction is increased by making the sides in removable sections, facilitating loading and making it unnecessary for the crew to lift refuse receptacles more than waist-high.

The body of the truck illustrated, which is manufactured by the White Co., Cleveland, O., is 13 ft. 3 ins. long by 7 ft. 5 ins. wide, having a capacity of 252 cu. ft. The top is covered with sectional doors for sanitary reasons and the sides have the above mentioned sectional feature.

Underslung Truck for Refuse Removal.

When dumping the body elevates to an angle of 53°; the hoisting mechanism being placed horizontally across the frame in a position out of the way during operation of the truck.

Ashes, garbage and all kinds of refuse are transported. A truck of this type operated by the New York Street Cleaning Department is said to accomplish the work formerly done by 15 one-horse carts. From 110 to 122 cans of ashes, the equivalent of five one-horse cartloads, is the capacity of the truck. In one district, with an average haul to the dump of 3 miles, one man driving and dumping makes a round trip in 30 minutes.

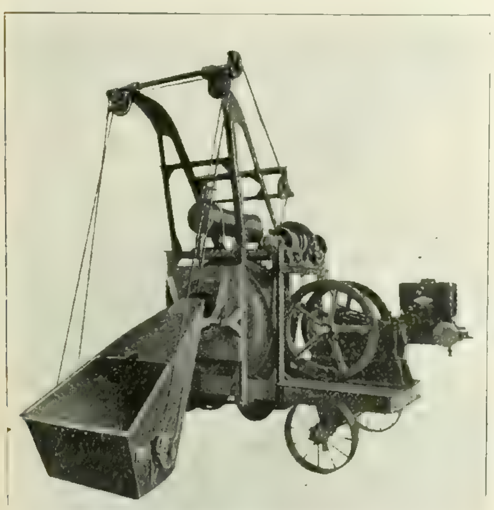


Self-Propelled Power Tamper.

A Self-Propelled Power Tamping Machine.

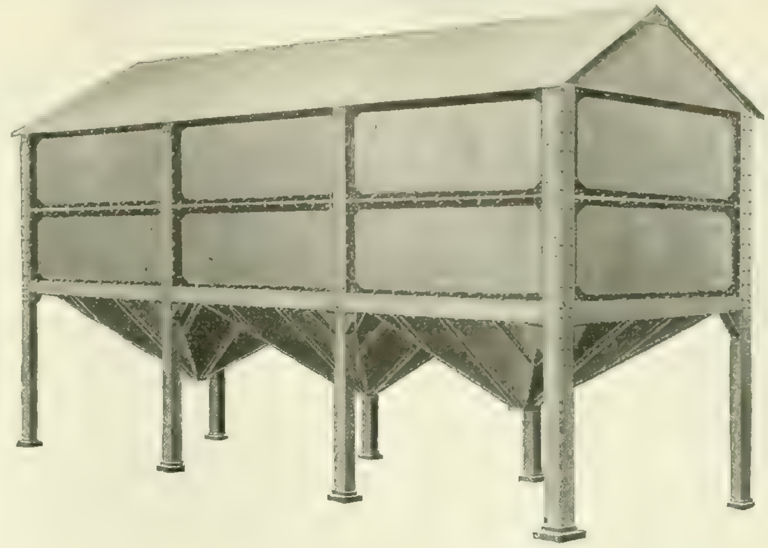
A power tamping machine, which is self-propelled, has recently been placed on the market by the Pawling & Harnischfeger Co., Milwaukee, Wis. The machine is equipped with a tamping ram, which weighs 150 lbs. and strikes 40 to 50 blows per minute with an average stroke of 22 ins. The tamping head, which is 9x12 ins., has a side travel of 20 ins., enabling it to tamp a width of 32 ins. The ram is operated by means of a segmental pulley "A" (see illustration), brought in friction contact with a hardwood ram "B," by spring pressure on a large pulley "C" which is located on the opposite side of the ram.

The machine has a speed of 1⅓ miles per hour for road traveling, although this speed can be varied by changing the engine governor. A feeding speed of from 6 to 15 ft. per minute is provided for to furnish a slow continuous movement, forward or backward, when at work. The machine is mounted on a four-wheel truck having adjustable wheels which permit the tread to be varied from 4 ft. 8 ins. to 6 ft. 8 ins. The wheel base is 5 ft. 9 ins. and the rear wheels are mounted so that they have a vertical adjustment of 10 ins., to compensate for the crown of the road and to enable the machine to work where the ground at one side is higher than at the other. The front axle is swiveled at its center to permit the adjustment of this end of the machine.



Kerosene Engine Operated Concrete Mixer.

Power is furnished by a 4-HP. water-cooled gasoline engine, "D," mounted over the front axle. The transmission is by friction clutches, gears and chains. The main countershaft is driven by a high-speed roller chain "E." A second chain "F," running from the countershaft, operates the splined lifting shaft. When desired, the tamping ram can be thrown out of action by means of a lever "G." The tamping ram and its operating mechanism are mounted in a frame supported at one end by the shaft and at the other by a roller, which runs on a track riveted to the framework of the machine. The tamping frame is moved transversely by means of a wheel "H" and a series of drums and wire ropes. The steering mechanism is controlled by a wheel "I," and all operating wheels and levers are within reach of the operator's seat at the rear of the machine. Although the standard tamper head is 9 x12 ins., other sizes can be furnished.



View of Weller Unit System Storage Bins.

A Kerosene Engine Operated Concrete Mixer with Improved Cast Drum.

The special features of the concrete mixer illustrated are the improved cast drum, the kerosene driving engine and the newly designed clutch for operating the side loader. This mixer is one of a full line of mixers made by the Power & Mining Machinery Co. (Cudahy), Milwaukee, Wis.

The drum construction is shown by the accompanying pen drawing. It consists primarily of two (right and left) semi-steel castings. These castings are joined by bolted outside flanges and the bolts through the flanges also engage the rack ring. Except for the mixing buckets as shown there are no projections inside the drum. Outside the drum is encircled by two steel truck rings shrunk onto and pinned to the castings.

The engine is a horizontal, water cooled, four cycle engine which is started with gasoline and then by means of a three-way cock is switched to kerosene or distillate. The fuel tank is in the base and has two compartments, a small one to hold gasoline for starting and a larger one for the main fuel supply. The cylinder is cast separately from the base but the cylinder and water hopper are cast in one piece. The ignition is make-and-break, the carbureter is suction feed and the governor is of the fly ball type. All engines are completely equipped.

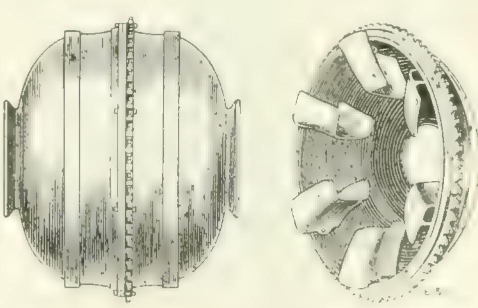
The clutch shown on the driving shaft in the general view is known as a rest clutch and is described as follows: By pulling the lever toward you, the clutch band is engaged; by reversing the lever, the brake band is applied, which does away with the extra lever. There are only two adjustments on the clutch and brake, consisting of a T-bolt and nut (or take-up ends) for adjusting clutch and also the same attachment for adjusting brake band. This clutch requires no oiling with the exception of a pint or quart of ordinary black oil being placed in the winding drum, which will last for several months. The shifting collar however should be kept well lubricated. When mixer is equipped with gasoline or kerosene engine, an extra clutch is required on the main countershaft so that mixer can be disengaged when starting engine. When bucket is loaded, pull the clutch handle toward you and bucket is elevated to the discharging position. Then throw the lever in the opposite direction and brake is applied. To lower the bucket release the lever gradually and the bucket will return to the loading position. It requires but ten seconds for this operation.

A Unit System Storage Bin for Contractors' Use.

The Weller unit system storage bin is illustrated herewith. This storage system consists of a bin or series of bins made up of certain interchangeable units which may be assembled, taken apart, moved, stored or reassembled as many times as may be desired.

The main idea of the bins is to provide suitable storage for concrete aggregates, arranged to feed by gravity to the charging box of the mixer, or with suitable and simple conveyor system to carry material from different bins to the mixer.

The claim is made that with one of these complete outfits the material for 1 cu. yd. of a 1:3:5 mixture in place may be unloaded from cars, put into storage, reclaimed and delivered to mixer charging box for 10 cts., including interest, depreciation, repairs and renewals.



Drum Construction of Kerosene Engine Operated Concrete Mixer.

The bins may be set up in a row, as shown in the illustration, or in an L or T-shape, quadrangle or any other similar position. The sides and bottoms of the bins are made up of sections which are bolted together and to the posts and cross struts. The structure is easily taken down and loaded for transportation. The unit parts are interchangeable. The bottoms are made up of steel plates, being hung down from the inside of the bin frame.

The bins are made in two regular sizes. The large size is for handling material in carloads, the bins having a capacity of about 80 cu. yds. each. This allows the storing of another car of material without waiting for bin to become empty. The general dimensions of these bins are 12 ft. by 12 ft. by 12 ft., not including the hopper bottom. The smaller size is designed especially for road building work and small screening and storage plants. These bins have a capacity of about 20 cu. yds. each, and are so arranged that wagons can be driven directly under the bins and loaded by the opening of the discharge gates.

An interesting feature in the application of these bins is the use of bulk cement which is now meeting with considerable favor by many contractors.

These bins, which are manufactured and marketed by the Weller Manufacturing Co. of 851 North Ave., Chicago, may be used for the following purposes: Concrete mixing plants, stone-crushing plants, sand and gravel plants, contractors' coaling system for coaling locomotives and steam shovels, and road construction plants.

A Compact Trailing Oil Sprayer.

The machine illustrated was designed as a practical substitute for the expensive sprayer tank wagon having self-contained pumping and spraying attachments. The standpipe shown is connected by a suction hose to any kind of tank, or to a barrel carried on a wagon. By means of a force pump geared to the axle the oil is drawn from the tank or barrel and forced through a large strainer box direct to the spraying nozzles. The spraying is done under pressure, and is accomplished evenly. Provision is made to engage or disengage the gears by a movement of a lever, and the pump flow is by-passed through a pressure regulator, so that damage to the pump is prevented in case valves are closed suddenly. An air chamber is provided above the strainer, which insures a steady pressure at the nozzles. The strainer may be opened, cleaned and closed in a few minutes' time. The spraying nozzles are arranged in two batteries of six nozzles each, and are controlled by two convenient levers reached from a seat so placed as to give the operator full view of the work. A connecting link enables the operator to manipulate all the nozzles with one lever, if desired, and regulate the quantity of oil discharged. The sprayer is manufactured by the Universal Road Machinery Co., Kingston, N. Y.

The Reliance Trailing Oil Sprayer.

A Threaded Cast-Iron Pipe.

The manufacturers of cast-iron pipe have long been urged to make pipe with threaded joints. Such pipe is now being made and sold by James B. Clow & Sons of Chicago. A view of a section of threaded pipe is shown in the accompanying view. In the manufacture of this new class of pipe machines similar to those for threading wrought pipe are used.

View of Section of Threaded Cast Iron Pipe.

In assembling a line of this pipe cast-iron couplings are used, with expansion joints at intervals suitable to take up the expansion and contraction in the line. A line of all cast iron is thus secured which is capable of standing pressures up to 250 lbs. per square inch.

For corrosive soil this pipe will answer the demands of many who have had to replace other kinds of pipe at short intervals. For wells driven in corrosive soil, where thin tubing is soon eaten up, cast-iron threaded pipe has been used to great advantage. Gas companies on their high-pressure lines will find this class of pipe to be economical. Steam-heating companies have already used large quantities of it.

A Continuous Bond Curb Bar.

A new curb bar recently placed on the market by the Trussed Concrete Steel Co., Youngstown, Ohio, involves a novel feature. The bars are made of open hearth steel, the anchorage flange being punched and expanded to the shape shown in the illustration. After forming, the whole bar is heavily galvanized. This expansion increases materially the rigidity of the whole bar, making it easier to handle during construction and increasing its ability to withstand shocks and distribute them throughout the mass of concrete. Moreover, it is claimed that the loop provides a positive anchorage independent of the adhesion of the concrete. The open anchorage reduces the danger of the plate separating from the concrete and eliminates any tendency of the concrete to split at the corners. The whole curb bar acts as a rigid frame. No special tools are required for installation.

A Continuous Bond Curb Bar.

Safety Device for Pneumatic Riveting Hammer.

To comply with the provisions of safety appliance laws recently enacted the Ingersoll-Rand Co., 11 Broadway, New York, has developed a retainer for use on its "Little David" pneumatic riveting hammer, which is designed to prevent the accidental ejection of the rivet set from the nozzle of the hammer. The retainer consists of a single piece of heavy spring steel, closely wound into spiral form, one end of which fits over the outside of the nozzle of hammer and hooks over a projection on the nozzle, the other end being wound to a small diameter. The sets for rivets over ⅞ in. in diameter are formed with a coarse thread and are simply screwed into place, while those for smaller sets are formed with a shoulder and are slipped into the retainer when it is detached from the hammer, the shoulder being relied upon to hold the sets in place. These hammers are made either with outside or inside triggers in five sizes. In addition, two sizes of jam riveters, which are especially adapted for work in close quarters, are made.

Rapid Loaders in Filling Position.

A Tilting Wagon Loader for Use in Unloading Railroad Cars.

The Rapid loader, illustrated in the accompanying figures, applies the tilting principle to a car-side wagon-loading device, thereby shortening materially the time required for that operation. The loader is designed so that it tips easily, and returns to the loading position automatically by gravity after dumping. It is constructed entirely of steel and as light as consistent with proper strength and durability. The two separate parts of which it is made facilitate rapid handling while moving from one car to another, two men accomplishing this task with ease.

A brief description of the one-cubic-yard capacity loader is as follows: The pan is 4 ft. wide (parallel with the car body), about 30 ins. deep and 5¾ ft. long; and is made of sheet steel reinforced with steel angles. The pan is supported by a shaft passing through it, in turn supported by steel brackets built of structural angle shaped members; the brackets being so designed as to hang at the car side and are separable if necessary. The pan and brackets each weigh about 130 lbs.; the combined weigh being 260 lbs.

The loader is made in 1 cu. yd., 1¼ cu. yd. and 1½ cu. yd. sizes and is controlled and sold by the Bonney Supply Co., 371 St. Paul S.., Rochester, N. Y.

Metallic Tape Threader.

A patented measuring tape attachment, known as the "threader," has been brought out by the Lufkin Rule Co., Saginaw, Mich., which hereafter will be furnished with its metallic woven tapes without extra charge. This device is a loop and stud attachment by means of which the tape, although securely fastened to the winding drum of the case when in use, can be easily detached from it and a new tape as readily attached, no manipulation of the case, case screw or drum being required to do this. As the case represents approximately half of the cost of the outfit, the new device permits the fullest measure of use of the case as well as of the tape.

Dumped Position of Rapid Loader.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design and to Methods and Cost of Construction

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A Preferential Vote on an Engineering Problem at Wheeling, W. Va.

The present water supply of Wheeling is of poor quality. It is pumped from the Ohio River into the water mains. Under the circumstances it is not surprising that for the past ten years the city has had a typhoid fever death rate of 80 per 100,000. This figure is reduced to 56 per 100,000 when the deaths of non-residents, who have died of typhoid in Wheeling hospitals, are deducted. That this is a disgraceful typhoid rate for a city the size of Wheeling is fully recognized by the citizens of that city. An improved water supply will soon be secured; at any rate a sum of money sufficient to provide the city with an ample supply of wholesome water will undoubtedly be voted by the people of Wheeling before long. A considerable controversy has arisen in that city as to the merits of different methods of purifying the river water to such a degree that it will be fit for industrial and domestic purposes.

That the quality of the present supply is bad is well indicated by the fact that all who can afford to do so use bottled table waters, for domestic use, at a price ranging from 5 to 8 cts. per gallon. The less well-to-do either boil the tap water or depend on water drawn from wells drilled along the city streets. The latter water often is nearly as bad as the raw river water. Of course there must be the usual carelessness about boiling the water and the high typhoid rate indicates that this is the case.

Largely as a result of the poor quality of the supply the consumption of water is enormous, being no less than 380 gals. per capita per day on the average throughout the year. Probably attempts on the part of the householder to draw a clearer water from his taps causes him to leave them running for an undue length of time, thus greatly increasing the waste of water. The grit in the water also leads to much leaky plumbing. In the aggregate this large waste of water greatly increases the cost of pumping over what it would be with a reasonable consumption.

This condition of the water supply caused the Wheeling Board of Control to appoint, late last year, a filtration commission, and to instruct this commission to report upon the relative desirability and cost of securing an improved water supply from driven wells, from the Smith System of Natural Slow Sand Filtration, and from the rapid sand filtration process, respectively. The commission consisted of reputable and experienced engineers. Late in March of this year the commission submitted its report and recommended the rapid sand process as being both the best and cheapest for Wheeling conditions. This process has been applied with conspicuous success in over 450 cities in the United States, including several along the Ohio River, notably Cincinnati and Louisville.

For some reason not clear to the editors and, it appears, equally obscure to many professional men in that city, the Wheeling city council did not use the expert advice for which the city had paid but set aside the report of their advisers. The choice of a purification system was submitted by the city council to a vote of the people, and in July of this year, the people declared for the Smith System. Work on plans and specifications for the site and details of the river bed plant is now going forward and it is hoped that the vote on the bond issue will be taken this fall. If the bond issue carries work on the construction of the plant will probably be started next spring.

In the water works section of this issue there is published an article which describes the Smith system and sets forth its advantages as seen by its advocates. Quotations are also made from the report of the filtration commission, above mentioned, which unqualifiedly condemned this system of filtration. Thus both sides of the story are placed before our readers who can draw their own conclusions as to the merits of the system in question.

It will be noted, from a reading of the article to which we refer, that experts differ widely as to the dependability of the Smith system. How, then, could the Wheeling voters be expected to reach a wise decision in such a purely technical matter?

In discussing this phase of the question, Dr. S. L. Jepson, Secretary of the West Virginia State Board of Health, recently characterized as a "monumental blunder" the action of the city council in asking the people generally to express their preference by a vote as to what system of filtration should be adopted. He considered this a matter for scientific determination, one about which laymen cannot be expected to know, and about which even the best informed know none too much.

We think Dr. Jepson stated the case mildly and well. While it is true that the people will have to pay for the plant it is equally true that they are not in a position to know what is best for the city when it comes to choosing between filtration processes. There is such a thing, surely as carrying to extremes our ideas of popular government. As we view it the action of the Wheeling council in submitting this proposition to a popular vote came dangerously near reducing our democratic institutions to the absurd and ridiculous. It would be but little more absurd to allow the school children of Wheeling to vote on the advisability of opening the city's schools this fall.

Traffic and the Cost of Hauling as Factors in the Rational Design of Country Roads.

In details of construction, in methods used and in the quality of work accomplished it may be stated without danger of refutation that roads constructed in the United States will compare favorably with those built in other countries. Yet it must be admitted that few roads in this country have been planned to accomplish a definite result, or with a knowledge of the effects that would be produced as a consequence of their construction. Good roads are popularly accepted as a good thing. The building of country roads improves general conditions, increases the value of land, reduces the cost of hauling, increases the comfort of the rural population and aids the business facilities of city dwellers. But to what extent these results may be counted on before beginning improvements is, unfortunately, largely a matter of conjecture.

The approximate solution of such questions as a preliminary feature of road improvement projects and their consideration in the laying out of routes and the selection of surfaces constitutes the rational design of country roads. It has been only quite recently that the importance of studies instituted to secure data for this purpose has been felt. Recently several factors have aided in impressing upon the minds of engineers the great need for the consideration of such matters in the development of road projects. Of these, the increased cost of road surfaces suitable for modern traffic and the increased number of extensive, well-knit road systems planned are perhaps the two most important.

Aside from the convenience and comfort to be obtained, the cost of hauling and the radius of effective hauling are distinctly functions of the size and justifiable cost of a proposed road system. Also, the amount and type of present and future road traffic are just as distinctly functious of the type of surface suitable to meet the requirements of this traffic. Both traffic and hauling problems are interwoven with other factors and are, in a measure, dependent on each other. Yet even a partial solution of the problem of the requirements of these factors with regard to a given road problem greatly simplifies and aids in economic design.

A data blank has been recently sent to many engineers by a special committee of the American Society of Civil Engineers requesting information in connection with road traffic and its effect upon bituminous surfaces under their observation. The importance of aiding in the investigation undertaken by that society is worthy of emphasis. It is only from the gradual accumulation of data and the study of many men that principles of value may be established. The insistent need of even fairly well established principles and their considerable value is felt by every engineer that has occasion to approach this subject. We can but urge engineers to lend their assistance.

The foregoing is but a single phase of the subject. Its intricacy is baffling and it is complicated by conditions vastly more complex than those involved in railroad construction. The importance of the subject can be estimated from the fact that during 1913 there were expended on country roads alone in the United States over $200,000,000.

The Procedure and Status of Public Building Construction.

It is interesting to note the procedure followed by the supervising architect for the Government in preparing plans for and in constructing public buildings. The insistent requests for the immediate preparation of plans for public structures which have been authorized by existing legislation, irrespective of the date of such authorization, have called forth an official statement from the supervising architect, in which an explanation is made of existing conditions. The attempt of politicians to appease their constituents by securing the "authorization" of buildings, even though such buildings may not be built for many years, has resulted in several hundred buildings being authorized, even though no funds have yet been appropriated for their construction.

It is stated that there are at this date approximately $38,500,000 worth of authorizations for new buildings and extensions, exclusive of the value of the land, on the books in the office of the supervising architect, waiting to be placed under contract. There is little wonder that inquiries are received by the supervising architect when his present office staff is placing buildings and extensions under contract at the rate of about $7,000,000 annually, which means that at the present rate of progress the buildings authorized in the act of 1913 will *probably* be placed under contract in 1920. Legislators who have recently been able to secure for their districts the authorizations for public buildings will be lucky if they are still members of Congress when such buildings are dedicated. The current program of the supervising architect, made up of the remaining projects which were authorized prior to the 1913 act, includes about one year's work from date, which means that work on plans for buildings authorized in the act of 1913 will not commence until about July 1, 1915. The program of work contained in this act has not yet been laid out, and it is impossible to state, even approximately, the date when any building authorized in this act will be advertised for bids or placed under contract. Some idea of the magnitude of the supervising architect's task—and incidentally that of the taxpayer—may be gained from the fact that the 1913 act contained the authorizations for 304 new buildings and 23 extensions to existing buildings. It is estimated that the cost of these buildings and their sites is approximately $42,000,000. It should be borne in mind that these 327 new buildings and extensions were authorized, but that not one cent was appropriated for their construction.

By "authorization" is meant merely authority to acquire a site, erect a building, etc., within the stated limit of cost, and it is not an actual "appropriation" of money out of the Treasury. Appropriations are made by Congress on a basis of estimates submitted by the Secretary of the Treasury. These appropriations are carried in one of the annual supply acts and are intended only to be sufficient to carry forward the building program for the fiscal year to which the acts apply. All appropriations for sites, buildings and extensions are available until expended, unless repealed by act of Congress, and are not automatically returned to the Treasury if not expended before the expiration of the fiscal year. If a subsequent act increases the limit of cost, or otherwise modifies prior legislation for a building or extension, before that project is reached in turn, the position of the project in the program of work is not affected thereby. Moreover, if a project is set aside when its turn is reached in order to await action upon a proposed increase in the limit of cost, or for any other reason, it is taken up whenever "released" in as near its original position as the conditions of the work may render it possible. In determining the precise order in which the plans for the various buildings shall receive attention, those authorized in any certain act are first divided into groups according to the priority of the acts in which their sites are authorized, and all buildings in each such act-group are then arranged in the order of the dates of the acquisition of their sites. Contracts for the construction of buildings are let in this order. Although extension projects are included with the new buildings authorized in any particular act, discretion is exercised in determining the order in which extensions shall be placed under contract for construction, depending upon the urgency of the need for the improvement.

The above order of procedure which is followed in constructing public buildings explains, in a measure, why certain public buildings possess inadequate facilities even when first constructed. It does not, however, explain why some relatively unimportant cities possess unduly large and expensive public buildings.

Learning from Mistakes.

In previous issues we have recorded in these columns the conviction that the engineering profession has much to gain from a frank discussion of the mistakes of engineers. It has been gratifying to us to learn that many of our readers agree with the views we expressed, for in this agreement there lies the promise of substantial contributions to engineering knowledge. Thus, in the present issue, we are publishing an article in which a successful consulting engineer enumerates and discusses some mistakes he made, in his less experienced days as a water supply consultant, and draws a lesson from each mistake.

The lessons learned from mistakes are the ones longest remembered. The ordinary mistakes of the engineer, when frankly acknowledged and carefully studied, eventually become assets rather than liabilities. Our contributor mentions the fact that his mistake in connection with putting down the dug well to which he refers, while expensive at the time to his client, led directly to improved methods which ultimately saved, in the digging of other wells, many times the loss caused by the faulty method first employed. The editors happen to know that in that city a water supply has long been drawn from this system of dug wells at a lower overhead cost for construction charges and at a lower operating cost per 1,000,000 gals. than is the case in any city in the country depending on ground water supplies. The low total construction charges were a direct result of the intensive study of the initial difficulty and its contributing causes.

While the various matters discussed are of greatest interest to water works men, the author's concluding remarks are of interest to all. As pointed out, the young engineer can learn much from the honest and experienced contractor about construction methods and difficulties. By co-operating with this type of contractor the young engineer can often catch his mistakes before they get beyond a paper existence, thus saving his client money and saving himself worry and embarrassment.

On Suggesting Subjects for Undergraduate Engineering Theses.

Upon the return of students to the engineering colleges the senior class members will find themselves confronted with the necessity of making an early selection of subjects for theses. This is always a bothersome matter to the student since, in his opinion, so many of the good subjects have been well covered by former classes. Consequently he is usually willing to entertain suggestions as to a suitable subject. Practicing engineers, having young friends in college, know that such suggestions often are solicited. It is the object of this brief note to urge engineers to extend their co-operation in this matter.

The graduate engineer knows the limitations of the student investigator and of the facilities for conducting engineering investigations at his alma mater. These limitations are not sufficient to debar the student from conducting some tests and investigations of real value to the profession.

It must be a common experience among graduate engineers to note the lack of data on certain specific points, in engineering literature, which it is within the capabilities of the undergraduate investigator to supply. When such matters come to the attention of the engineer he should suggest them as worthy of consideration as theses subjects. Such suggestions may be made to the student personally, where acquaintance exists, and in other cases to the professor in charge. The engineer may be sure of the appreciative co-operation in this matter of both student and teacher.

BRIDGES

Design Features of the Cantilever, Simple-Truss and Girder Spans of the Bloomfield Bridge, Pittsburgh, Pa.

(Staff Article.)

I.

The Bloomfield Bridge, which is under construction in Pittsburgh, Pa., spans a ravine and connects Grant Blvd. at Ridgeway St. with Liberty Ave. near Main St. This highway structure, which possesses some unusual features, has a total length of about 1,740 ft., and consists of the following spans: Ten 60-ft. girder spans; three 120-ft. truss spans; two 120-ft. anchor arms; two 140-ft. cantilever arms; one 120-ft. suspended span; and seven 20-ft. tower spans supported on seven towers and six bents. Figure 1 shows an elevation of the bridge and a profile of the site. This drawing indicates the type of construction adopted to conform to the topographical conditions, the bridge being on a 3.907 per cent grade. The roadway has a clear width of 34 ft., and each cantilevered sidewalk has a width of 8 ft. The asphalt roadway paving is supported on a steel buckle-plate floor, which is in turn supported on steel stringers and floorbeams. The sidewalk floor consists of a reinforced concrete slab, supported by the curb girder and brackets, by the fascia girder and by the stringers. The roadway provides for two car tracks.

LOADS AND ALLOWABLE STRESSES.

Dead Load.—In estimating the dead load the asphalt paving was assumed to weigh 30 lbs. per square foot; the concrete, 150 lbs. per cubic foot; track rails and fastenings, 160 lbs. per linear foot of bridge; sidewalk railings, 50 lbs. per linear foot each; and two water pipe lines, 150 lbs. per linear foot each.

Live Loads on Floor System.—The specified live load for the floor system consisted of a continuous line of 35-ton, double-truck cars 42 ft. long, having wheels of trucks spaced 5 ft. 7½ ins. center to center, trucks of car spaced 26 ft. 6 ins. center to center, and trucks of adjacent cars spaced 20 ft. 7½ ins. center to center, the two tracks occupying 18 ft. width of roadway. In addition to the load on the car tracks a 15-ton truck, occupying an area 8x20 ft., and having one-third of its weight on the front axle and two-thirds on the rear axle, with a gage of 5 ft. and with axles spaced 12 ft. center to center, was placed on each side of the street cars. In addition to these concentrated loads a uniform load of 125 lbs. per square foot was used on all of the roadway area not occupied by the street cars and trucks. The live load for the sidewalks consisted of a uniform load of 140 lbs. per square foot and a concentrated load of 1,000 lbs. at any point.

Live Load on Trusses.—In computing the live load stresses in the various truss and girder spans the loads per linear foot of span varied as given in Table I.

TABLE I. LIVE LOADS ON TRUSSES OF VARIOUS SPAN LENGTHS.

Length of span, in ft., up to and including—	Lbs. per lin. ft. of each street car track.	Lbs. per sq. ft. of remaining roadway floor surface.	Lbs. per sq. ft. of sidewalk floor surface.	Total load, in lbs., per lin. ft. of span.
100	1,670	125	100	6,940
110	1,650	123	98	6,836
120	1,630	121	96	6,732
130	1,610	119	94	6,628
140	1,590	117	92	6,524
150	1,570	115	90	6,420
160	1,550	113	88	6,316
170	1,530	111	86	6,212
180	1,510	109	84	6,108
190	1,490	107	82	6,004
200	1,470	105	80	5,900
210	1,450	103	78	5,796
220	1,430	101	76	5,692
230	1,410	99	74	5,588
240	1,390	97	72	5,484
250	1,370	95	70	5,380
260	1,350	93	68	5,276
270	1,330	91	66	5,172
280	1,310	89	64	5,068
290	1,290	87	62	4,964
300	1,270	85	60	4,860

Impact.—The impact stresses were computed using the formula $I = S \frac{100}{L + 300}$; where I = impact stress; S = maximum live load stress;

and L = loaded length, in feet, to produce the maximum live load stress. No impact was added to the lateral or wind forces or to the floor loads on the sidewalks.

Wind Loads.—In computing the wind load stresses the following loads were used (a) 30 lbs. per square foot on exposed surfaces of all trusses, towers and floor system as seen in elevation, and in addition 150 lbs. per linear foot on the loaded chord, treated as live load; or (b) 50 lbs. per square foot on exposed surface of trusses, towers, and floor system, with no live load.

Allowable Stresses.—The allowable unit tensile stress was 16,000 lbs., and the unit compressive stress $16{,}000 - 70\frac{l}{r}$ (with a maximum of 14,000 lbs.), where l is the length of the member in inches and r is the least radius of gyration.

SUPERSTRUCTURE FEATURES.

The types of bridge adopted for various parts of the structure are indicated in Fig. 1. Starting at the south end of the structure there are two 60-ft. deck girder spans and a 20-ft. tower span, followed by a 120-ft. deck truss span, the north end of which rests on an anchor bent (No. 4, Fig. 1). Between bents Nos. 4 and 9 there is a cantilever system, the outside anchor arms of which are anchored to these bents. This cantilever system, which has a clear span of 400 ft., consists of two 120-ft. anchor arms, two 140-ft. cantilever arms, and a 120-ft. suspended span. From bent No. 9 (the north end of the cantilever system) northward the bridge consists of a 120-ft. deck truss span and a 20-ft. tower span, another 120-ft. deck truss span followed by a 20-ft. tower span, two 60-ft. deck girder spans and a 20-ft. tower span, and three 60-ft. deck girder spans.

As regards duplication of spans it will be noted that the design is an economical one. The cantilever system is symmetrical about its center line, there are ten duplicate 60-ft. girder spans, four similar 120-ft. truss spans (one of which is the suspended span of the cantilever system), and seven duplicate 20-ft. tower spans.

The lengths of the various spans of the cantilever system and of the adjacent simple trusses are such as always to produce an upward reaction at the anchor bents Nos. 4 and 9.

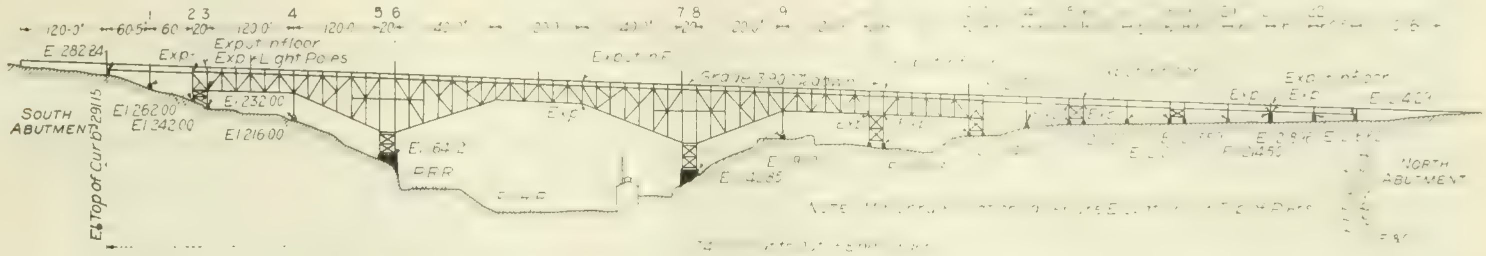


Fig. 1. General Elevation and Profile at Site of Bloomfield Bridge, Pittsburgh, Pa., Showing Type of Structure and General Dimensions.

ANCHOR AND CANTILEVER ARMS.

The south anchor and cantilever arms are supported on a tower about 30 ft. high. The anchor arm has a length of 120 ft. and the cantilever arm a length of 140 ft., the trusses being spaced 32 ft. apart, center to center. The panel length in both arms is 20 ft. The height of these arms, center to center of pins, at the tower, is 75 ft. and at the ends 24 ft. 5⅛ ins. Figure 2 is an outline drawing of the south anchor and cantilever arms and of the lower lateral bracing and floor system. This drawing gives the principal dimensions, the maximum stresses (sum of dead, live and impact stresses), and the composition of each truss and lower lateral member. There is no lateral bracing in the plane of the upper chord, as the bridge has a buckle-plate floor. It will be noted that the tower bents have heights of 30 ft. 1 3/16 ins. and 30 ft. 10 9/16 ins., to conform to the grade of the bridge. The upper chord member which connects the cantilever arm and the suspended truss span consists of two 6 x 1⅛-in. bars with turnbuckles. As the suspended span has riveted joints, the connection is made by extending plates beyond the upper chord panel point and boring pin holes through them. To provide for a considerable movement at this connection the pin holes are slotted. By referring to Fig. 2 it will be noted that the intermediate posts of the anchor and cantilever arms are extended above the upper chord pins a considerable distance. This extension is sufficient to permit the connection of the floorbeams above the pins. A rigid system of sway bracing connects the truss and girder spans.

The north anchor and cantilever arms are of similar construction to those shown in Fig. 2, the dimensions and sections being the same, although the stresses in some of the members differ slightly. These arms are supported on a tower, the bents of which are 34 ft. 11½ ins. and 35 ft. 8⅞ ins. high.

SIMPLE TRUSS SPANS.

The simple deck truss spans have a length of 120 ft., center to center of bearings, and a height of 25 ft., center to center of chords. The panel length is 20 ft. and the trusses have riveted connections, except at the supports, which are pin-connected to the tower bents. The trusses are spaced 32 ft. apart, center to center. The floorbeams rest on top of the upper chords, the cantilever sidewalk brackets being attached to them. The trusses have rigid lower lateral systems, but the buckle plate floor takes the place of the upper lateral systems.

PLATE GIRDER SPANS.

The plate girder spans have an extreme length (measured horizontally) of 60 ft. 10 ins., and a length, center to center of bearings, of 59 ft. 2 ins. The depth, back to back of flange angles, is 7 ft. 0½ in. The flanges of the girders are parallel to the grade of the bridge, while the stiffeners are vertical; therefore the stiffeners are not at right angles to the flanges. The ends of the girders rest on cast bearings, which are in turn supported either on the steel bents or directly on the concrete piers. The sidewalk brackets are connected to the stiffeners, the top of the bracket being in line with top flange angles of the girders.

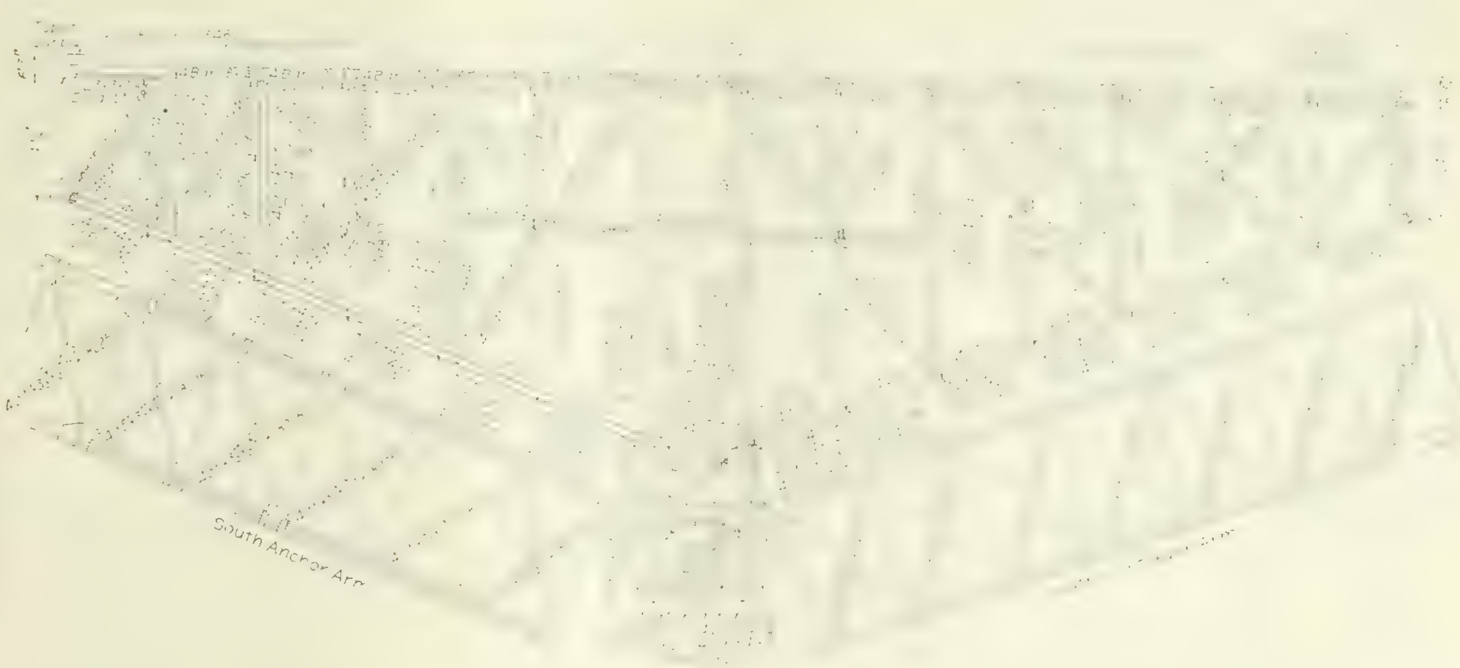


Fig. 2. Stress Sheet for South Anchor and Cantilever Arms of Bloomfield Bridge, Pittsburgh, Pa.

FLOOR SYSTEMS.

The flooring for the roadway consists of a concrete slab on buckle-plates, surfaced with asphalt. The flooring for the sidewalks is a reinforced concrete slab.

Figure 3 shows a half cross section of the floor construction used for the cantilever and anchor arms. This drawing shows the manner of connecting the floorbeams and sidewalk brackets to the extensions of the intermediate posts. The crown of the roadway is obtained by varying the thickness of the concrete slab.

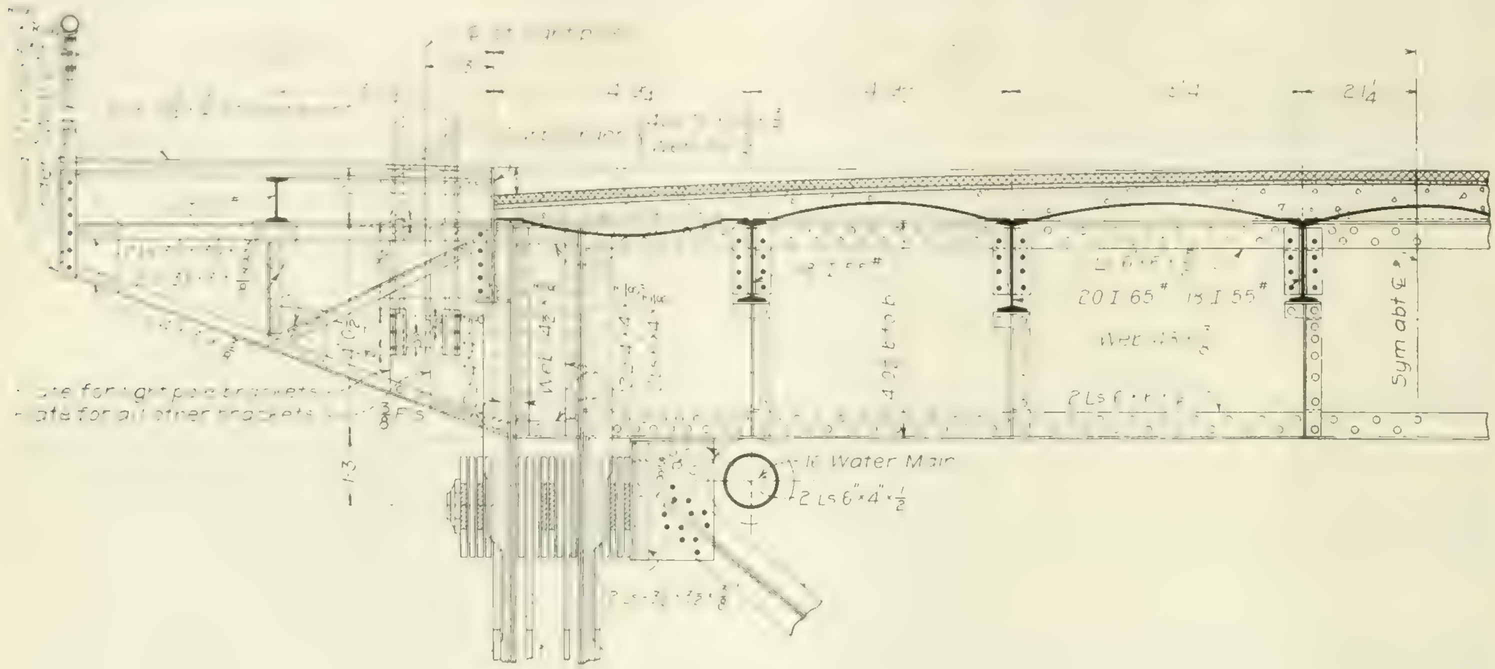


Fig. 3. Half Cross Section of Floor System for Cantilever Spans of Bloomfield Bridge, Showing Type of Construction, Sections and Dimensions.

It was necessary, however, to invert the buckle-plates between the curb girder and the outside line of stringers to obtain sufficient crown (see Fig. 3). The principal dimensions and the composition of each part of the floor system are given in Fig. 3.

The construction for the floor system of the 120-ft. deck truss spans and the 60-ft. deck girder spans is similar to that for the cantilever spans shown in Fig. 3, except for the connections at the ends of the floorbeams. Figure 4 (a) shows a detail at the end of the floorbeams for the 120-ft. truss spans. It will be noted that the floorbeams rest directly on the top chords of the trusses and that the sidewalk construction is bracketed to the end of the floorbeams. Figure 4 (b) shows a detail of the connection between the floorbeams

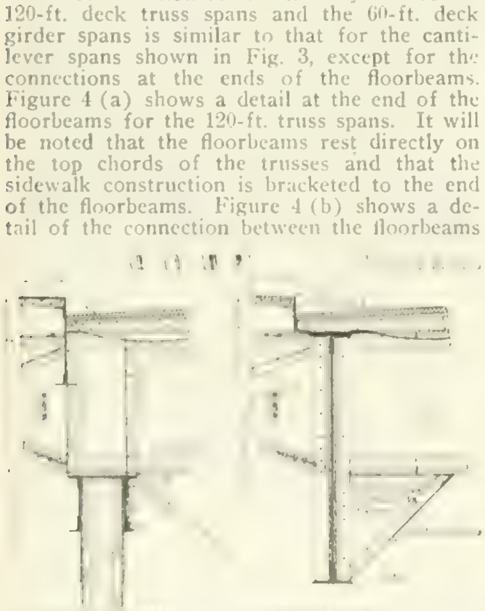


Fig 4. Details Showing Connections of Floorbeams and Sidewalk Brackets to 120-Ft. Truss and 60-Ft. Girder Spans of Bloomfield Bridge.

and the 60-ft. girder spans. It will be noted that floorbeams and sidewalk brackets are riveted to the stiffener angles of the girder.

RAILINGS AND LIGHT POLES.

The bridge has structural steel railings, the posts of which are spaced 20 ft. apart. Figure 5 shows a portion of the railing and gives details of its construction.

Ornamental lamp posts having a total height of 23 ft. 11 ins. are spaced as shown in Fig. 1. These posts have a concrete base, a shaft consisting of four 2 x 2 x ¼-in. angles latticed together, and an ornamental cast-iron top. Each lamp post carries two lights.

BENTS AND TOWERS.

By referring to Fig. 1 it will be noted that, to avoid the use of piers of great height, steel bents and towers were used, supported on low concrete piers. The bents vary greatly in height and in construction.

At the ends of the anchor arms of the cantilever system (at piers Nos. 4 and 9) anchor bents are required which are capable of resisting a heavy upward reaction. These bents are pin-connected to shoes which rest on masonry piers. The shoes are provided with wedges, which permit the adjustment of the bents. The pin which connects the lower end of each of these bents to its shoe is anchored by means of four 6 x 1⅞-in. bars buried in the pier. Figure 6 shows a half elevation of this bent and a side elevation of a tower leg. These drawings show clearly the shoe details and the provisions for anchorage.

At bent No. 3 (see Fig. 1) the 120-ft. truss span is pin-connected to a shoe. This shoe rests on a nest of six 9 x 4½-in. x 1-ft. 6-in. segmental rollers, which are in turn supported on top of the heavy bent.

The towers of the cantilever system, which are composed of bents Nos. 5 and 6 and Nos. 7 and 8, are of similar construction. The anchor and cantilever arms are pin-connected to the top of the bents of these towers, 10½-in. diameter pins being used. The tower legs have very heavy sections, each leg having three webs, a cover plate and heavy lacing bars. Anchorage of each leg is secured by four 2-in. diameter anchor bolts. The tower on piers Nos. 5 and 6 is 20 ft. x 37 ft. 8 7/16 ins. at the base, 20 ft. x 32 ft. at the top and 28 ft. 6 3/16 ins. high; while the tower on piers Nos. 7 and 8 is 20 ft. x 38 ft. 8⅛ ins. at the base, 20 ft. x 32 ft. at the top and 33 ft. 4½ ins. high.

Bents Nos. 10 and 11, which support the ends of two adjacent 120-ft. truss spans, carry expansion shoes, the roller nests being similar to those used at bent No. 3. These bents are 20 ft. x 39 ft. 1⅛ ins. at the base, 20 ft. x 32 ft. at the top and 35 ft. 5 7/16 ins. high. Each post is anchored by four 1½-in. diameter bolts.

Bent. No. 12, which is 17 ft. 4 11/16 ins. high, carries a fixed shoe, to which one end of a 120-ft. truss span is connected. Bent No. 13 is 40 ft. 7⅞ ins. high, and directly supports one end of a 60-ft. girder span. These two bents are rigidly connected with heavy bracing. Bents Nos. 15 to 20, inclusive, are of varying heights and support the 60-ft. girder spans.

PAINTING OF STEELWORK.

The specifications for the steelwork require one shop coat and two field coats of paint, surfaces inaccessible after erection being painted three coats before erection. It is required that all abrasions of the shop coat be retouched before applying the field coats, and that surfaces inaccessible with brushes or swabs be painted with a spraying machine.

The shop paint is mixed in the proportions of 80 per cent pigment and 20 per cent vehicle, the pigment consisting of 85 per cent red lead, 5 per cent neutral zinc chromate and 10 per cent French yellow ochre. The first field coat is mixed in the proportions of 60 per cent pigment and 40 per cent vehicle, the pigment consisting of 100 per cent blue lead. The second field coat consists of 60 per cent pigment and 40 per cent vehicle, the pigment being blue lead with sufficient carbon black to

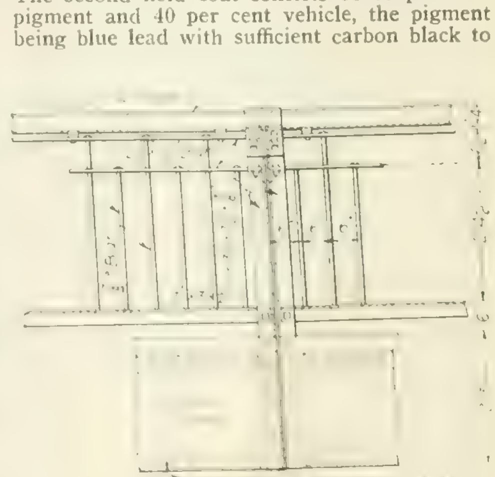


Fig. 5. Portion of Railing for Bloomfield Bridge, Showing Details of Construction.

give the color desired by the Director. The use of driers in the shop and first field coats are not permitted, while in the second field coat not more than ½ pint of drier per gallon of paint is allowed, the drier to contain at least 80 per cent pure turpentine and not more than 20 per cent pure refined benzine. It is specified that all drier must be added to the paint at the site.

QUANTITIES OF MATERIALS AND COST.

The estimated quantities of materials in the superstructure of this bridge, exclusive of paving, are as follows:

Structural steel, tons	3,497
Hand railing, linear feet	3,925
Structural lamp posts, number	18

The contract price for the superstructure, exclusive of paving, was $248,929, and the price for extra steelwork was 3½ cts. per pound.

PERSONNEL.

The Bloomfield Bridge was designed by the Department of Public Works of Pittsburgh, Mr. Charles A. Finley, acting director; Mr. N. S. Sprague, superintendent; Mr. T. J. Wilkerson, division engineer, and Mr. Emil Swensson, consulting engineer.

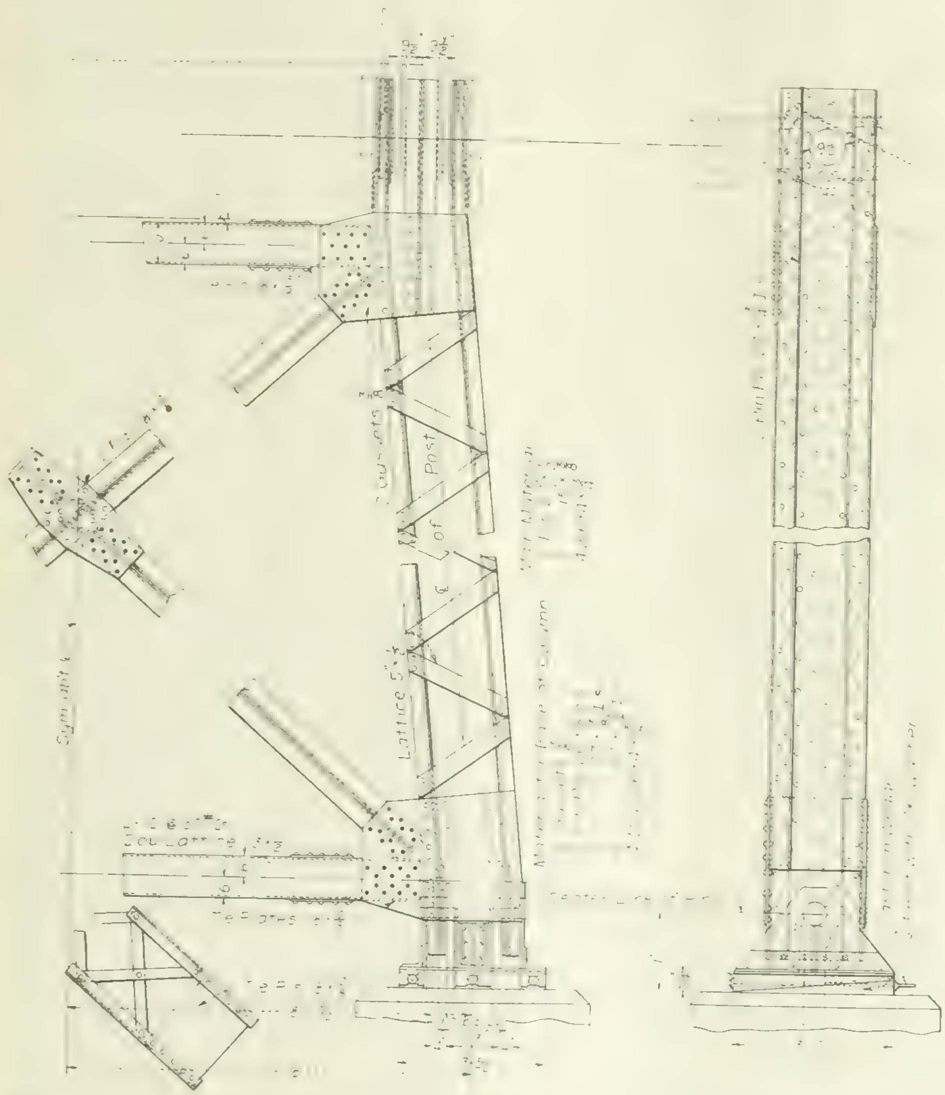


Fig. 6. Detail Drawing of Anchorage (Rocker) Bents Nos. 4 and 9 of Bloomfield Bridge —Note Anchorage and Manner of Adjustment.

Comments on "The Possibilities in Bridge Construction by the Use of High-Alloy Steels."

It is often difficult to secure pertinent discussion of broad engineering subjects. Too often engineers of ability and broad experience cannot be prevailed upon to discuss the fundamentals of engineering design and construction. Thorough discussion will often suggest ideas to those giving intensive study to projects which will aid greatly in their solution. In our issue of June 17, 1914, we gave some data on the use of high-alloy steel in the construction of large bridges, the data being abstracted from a paper by J. A. L. Waddell. In this issue we shall give abstracts from the discussions of this paper by some prominent engineers. The data were abstracted from the discussions of Mr. Waddell's paper, "The Possibilities of Bridge Construction by the Use of High-Alloy Steels," in Proceedings, American Society of Civil Engineers, Vol. XL, p. 1997.

DISCUSSION BY E. L. CORTHELL.

There is no doubt that the spans of bridges are increasing and that there are many locations where the kinds of steel described and treated in this paper would be used to bridge over wide expanses of water not now possible, if they could be manufactured at a reasonable price.

The author states that "The live load stresses on the main truss members will never be quite as great as they are computed, because, first, the trains on the two tracks never advance together so as to produce maximum web stresses; second, such trains are not likely ever to cover entirely the bridge or any individual part of it, except, perhaps, the central span; and, third, it is improbable that any load of cars, unless they be ore or coal cars, will ever be uniformly full and loaded to the assumed limit."

All of which is true and might well be taken into account in the actual design of long-span railroad bridges.

It is customary at present to allow for a train load of 5,000 lbs. per linear foot. Not many cars weigh (loaded) 5,000 lbs. per linear foot, and these are not often loaded to their full capacity. A car having a capacity of 100,000 lbs. will weigh about 50,000 lbs. and is 40 ft. long. This brings the actual load to 3,750 lbs. per linear foot, which is not much more than that allowed 20 years ago. Are designing engineers expecting still larger cars than at present? It would seem almost impossible to increase the loads per linear foot to more than those which now obtain.

Certainly safety should be the desideratum, but if the dead load of a bridge can be decreased materially by using judgment in the application of live loads in designing, a great deal of money would be saved, so much, in fact, that cases might arise where the difference in cost would make practicable a project which otherwise might be impracticable.

This, and substituting for carbon steel a steel of a "high, cheap alloy, suitable in every particular for bridges," which, the author says, "is now before metallurgists and the builders of large metallic structures," may reduce weights and consequent costs to such an extent as to be of immense advantage in the projects of railway bridges of long spans.

DISCUSSION BY DAVID A. MOLITOR.

Aside from a few innovations, chiefly with the aid of alloys, such as nickel, aluminum, chromium and manganese, to raise the elastic limit and ultimate strength, and titanium as a purifying agent, little has been accomplished to improve the quality of steel for bridge and structural uses since the introduction of the open-hearth process. However, the two great defects in all manufactured steel are segregation and piping, for which no remedy is known to mill operators except the costly and wasteful method of discarding from 15 to 40 per cent of the ingot, and, further, scrapping all material showing defects in the finished product.

Both segregation and piping are local defects; the former is always hidden in the finished material, and the latter is usually invisible except when it appears as a noticeable surface defect. Hence it follows that no specification can be written so as to insure a high-grade, homogeneous material, in spite of the most painstaking mill inspection.

Engineers prescribe both the chemical purity and physical requirements for bridge and structural steel, and require that a test shall be made from a specimen taken from each ingot, and on the results of these tests the metal is either accepted or rejected, reserving the right to reject such of the accepted material as may develop defects in course of manufacture. The presumption is that all material not rejected in the mill or at the shops is uniformly good and up to the specifications.

However, nothing could be more erroneous, and, even if every single piece of rolled material were tested in like manner, this could not be accepted as absolute proof of quality. For there might be a hidden flaw or pipe, or spot of segregated metalloids, anywhere in the finished piece, which would never be revealed by any test, but might ultimately come to the light through failure.

Although segregation and piping defects are likely to occur in any or all material, they are less likely to produce disastrous results in bridges than in rails, owing to the more strenuous service of the latter. It is for this reason that ingots for rail steel are cropped about 30 per cent on the average, and bridge steel is cropped about 20 per cent, on the average, with a minimum of about 12 per cent.

It should be noted that Bessemer steel is less subject to piping than is open-hearth steel, and that the latter suffers less segregation than the former, owing to its lesser sulphur and phosphorus contents and lesser quantity of metalloids.

Although these conditions exist in bridge steel of the present day their deleterious effects are largely overcome by using moderately low working stresses. It is well, however, to reflect on the conditions which would inevitably follow when the same basic material is raised to higher strength by using an alloy or higher carbon contents. Unless segregation and piping can be effectually eradicated, it would seem quite out of the question to consider high-alloy steels safe for large bridges, as defects of this class would become far more serious in slender sections with high unit stresses. In other words, a condition which might be tolerated in medium steel, might become prohibitive in the high-alloy steel.

The crucial point of this discussion rests on the willingness and on the ability of the steel manufacturer, first, to improve the quality of the present-day product by the prevention of segregation and internal pipes, and second, to raise the elastic limit and ultimate strength of homogeneous steel by the use of alloys, taking care to secure proper ductility in the high steel.

It may add to this discussion to enter somewhat into details as to the phenomena of piping and segregation, as comparatively little is known with certainty and hitherto all attempts to overcome these defects in steel have proved only partly successful.

Although chemical reactions of a most complex nature are known to take place between the various impurities, sulphur, phosphorus, silicon and oxygen, with iron, manganese and carbon, forming compounds which are known as metalloids, it is practically impossible to show the exact nature and composition of these metalloids by chemical analyses. It is quite probable that the metalloids in the molten state are very different from those in the frozen condition. There is also a considerable gas formation in the interior of the molten metal, chiefly caused by the oxidation of the carbon and some of the impurities.

It is also known that the metalloids have a lower specific gravity than iron, and that their melting points are lower than that of iron, which being the case, the metalloids will cool more slowly than iron, and will naturally free themselves, to a certain extent, and on account of the difference in specific gravity will be forced laterally and upward into the lake of molten metal and continue in this action, so that the last quantity of metal that is cooled will contain a concentration of these segregated elements. The gases are similarly displaced, and, being extremely light, will hasten the collection of the metalloids into nodules which will travel upward along lines of least resistance as the metal cools in the mould. It is this effect, due to the difference in melting points, and the gravity displacement of the metalloids and gases, which gives rise to segregation and may, therefore, be classed as a physical phenomenon.

Piping is also a physical phenomenon, being caused by the shrinkage of the metal in the mould as it cools, and is the result of the following conditions: The metal is poured into the cold mould, and, as it strikes it, it chills or freezes to it at the bottom and sides, the chill being continued to the top level of the poured metal. The cooling or freezing takes place successively toward the top and center of the mould, and as the metal cools it naturally contracts and draws its supply for contraction from the molten lake in the center which sinks as the demand is made on it by the contraction of the surrounding metal, the result being the formation of the funnel-shaped cavity which frequently reaches below the center of the mould. This cavity may contain, in some instances, hanging walls or bridges, which may seal across the lower part of the pipe, resulting in a hidden or concealed pipe. This may not be detected, either in the bloom or in the finished product.

Segregation and piping are thus seen to be physical manifestations which cannot be prevented by any chemical process or treatment.

Although a chemically pure steel would necessarily be free from segregation, yet no steel, or, in fact, no metal, can be poured into an ingot mould without showing a pipe. The great necessity of freeing steel from internal pipes is thus made apparent.

The use of titanium unquestionably improves steel, and especially when added to Bessemer steel, which is high in sulphur and phosphorus, and also in gases. For this reason, Bessemer steel boils more in the mould. The addition of titanium is especially useful to quiet the metal by preventing the gas production through combinations with nitrogen and oxygen, forming light substances which rapidly rise to the surface of the ladle, purifying the steel at the expense of the titanium, very little of which remains in the steel. Hence, titanium somewhat reduces segregation, but does not affect piping.

More than a year ago the writer's attention was drawn to a process of casting ingots, by which the gases are all expelled, the metalloids are prevented from rising, and the steel cools gradually from the bottom upward instead of inward from the sides of the mould, thus preventing unequal contraction along different elements vertically in the mould. This is all accomplished by one operation, without loss of time in casting ingots, and at little or no expense outside of the apparatus on which the mould is placed while making the ingot casting. The process is one of forging the liquid steel by imparting vertical vibration to the mould and its contents, and is called by the inventors, Messrs. Maxwell and Lash, "The Liquid Forged Steel Process."

From the foregoing description, and the phenomena attending ingot casting, it is quite clear that segregation and piping can be completely obviated, thus making it possible to produce a perfectly uniform and homogeneous steel in which the ultimate strength and elastic limit are raised about 20 per cent by the degasifying effect of the process.

The loss due to necessary discard of the ingots for bridge steel is reduced from about 20 per cent to 3 or 4 per cent, and for rail steel the reduction is from an average of 30 per cent also to 3 or 4 per cent. This would result in an actual saving of from $1.50 to $2.00 per ton for steel production; the cost is merely an interest charge on the value of the machinery used and a small royalty on the patents, to say nothing of the far greater value of the finished product and the savings brought about by lesser rejections of the rolled steel.

That these statements are correct cannot be disputed when the tests which were made last year by the patentees are examined. Hence, the introduction of this process into the large steel mills would be somewhat of a revolution in steel manufacture as well as in the vastly improved quality of the finished product.

DISCUSSION BY JOHN C. FERGUSON.

The use of nickel steel cannot be adopted, generally, as the best material for all classes of bridgework; its particular advantages adapt it more for use in bridges of very long span than in short ones. Its special advantages—elastic limit and tensile strength, hence lightness—over the best classes of open-hearth carbon steels, are only obtained in nickel steels of the very highest grades; and their cost would be prohibitive in ordinary structures. Carbon steels, for a tensile strength of 80,000 lbs. per square inch, have advantages over nickel steels carrying low percentages of nickel. The open-hearth carbon steel is a more reliable product, it is stronger in compression, has greater resistance to impact, its manufacture and testing are extremely simple, and the main results are constant; whereas, the testing of nickel steel during the process of manufacture is very complicated, the material is subject to great variations; and, further, the combinations of carbon, manganese, titanium, and tungsten, even in the smallest fractional percentages, produce most unlike results.

It has been found that the addition of nickel to iron causes the nickel to act as a flux on the carbon, manganese, and other minerals in combination with the iron; and the anomalous results produced vary much more owing to slight changes in the proportion of carbon, manganese, or other metals in the chemical composition of the ingot, than in the mere change of proportion in the parts of the nickel to iron.

The idea of adopting a standard specification for nickel steel, to be used in bridgework, seems to be wrong, because the conditions required for various spans and loads are so variable. Would it not be wiser for a civil engineer to state his requirements and see that they are carried out, leaving the actual manufacture of the metal to the metallurgists and ironmasters?

To use a nickel steel of very high tensile strength, chiefly to save weight in the bridge material—unless where the conditions are imperative—would seem to be extravagant, and even unwise. There must be risk in trusting to the highest limit of tensile strength of a manufactured steel which is known to be very variable. The mere saving of weight in bridges of moderate span would hardly warrant the risk, coupled with the extra cost of material of higher grade. Standard specifications for nickel steel would be worthless unless they met the highest conditions and requirements for the longest bridges mentioned by the author. It is for bridges of this class, of exceptional length, that he hopes to find a sufficiently strong and suitable material in nickel steel.

The author, in his previous paper, states that the best percentage of nickel was found to be 3.5. The allowance of impurities in the nickel steel was put as phosphorus, 0.03 per cent; sulphur, 0.04 per cent; silicon, 0.04 per cent; and manganese, 0.75 to 0.85 per cent; which produced an elastic limit of 61,300 lbs. and an ultimate strength of 99,300 lbs. per square inch. The impact was less than that of carbon steel, the relative values being 87 and 73 to 100, for low and high nickel steel.

The author also states that the nickel present in larger quantities makes the material unworkable in the shops.

If the author is still of these opinions, and if we may regard his present able paper as supplementary to his former one, the foregoing statements deserve consideration, because they do not agree with the general idea as to the chemical composition of a very high-class nickel steel that would best fill the requirements of the extraordinary long bridges he has mentioned. The proportion of 3.5 per cent of nickel, which he gives, appears to be too low. As he does not mention carbon in his analysis it may be presumed that he has eliminated it; surely this is a difficult operation in commercial manufacture. This is a most important point, for, with a low precentage of nickel, the reduction of carbon below 0.20 becomes essential.

Provided the carbon is kept low there is a constant rise in the tensile strength of nickel steel, in proportion to the increase of the nickel present, up to 12 or 15 per cent. The increase of nickel requires a decrease of carbon, together with an increase of manganese, otherwise, although the breaking load may be the same, the value of the elastic limit will fall rapidly.

The writer has pleasure in acknowledging that some of the earliest and most successful workers of nickel steel are Americans. The Bethlehem Iron and Steel Co., the Homstead Works of Pittsburgh, and others, are well known.

There are a few points about which there appears to be considerable difficulty to agree. The author states that steel containing more than 3.5 per cent of nickel was unworkable, yet in Sheffield they found no difficulty in working material containing nickel, 7.65 per cent; manganese, 0.68 per cent, and carbon, 0.17 per cent, although it was hard.

In America there is a preference for adding nickel in iron ores, whereas in England it is preferred to add the nickel otherwise. In Sheffield it is found difficult to produce forgeable bars in the absence of manganese, and it appears to be impossible to get satisfactory results.

Results of Some Experiments Made to Determine the Effect of Varying the Percentage of Water in Concrete.

The paucity of published data on the effect of varying the percentage of water used in concrete mixtures is evidence that this feature has not been given sufficient attention by engineers. The development of rapid mixers and chuting devices for placing concrete has resulted in the use of exceedingly wet mixtures, without much regard to the effect of large percentages of water on the strength and other properties of concrete. Samples of concrete taken from the floor of a building in Chicago, in which the chuting system was used, were recently tested. The results of these tests (as reported by Mr. McCullum) showed that the compressive strength of these samples averaged only 1,491 lbs. per square, the strongest of six samples failing at 1,650 lbs. per square inch, and the weakest at 1,230 lbs. per square inch. The following data on the results of experiments made to determine the effect of varying the percentage of water in concrete were taken from a paper by R. K. Skelton, in the Proceeding of the Connecticut Society of Engineers. The tests were made in the masonry materials laboratory of the Sheffield Scientific School by Mr. Robinson, under the direction of Professor Barney and with the assistance of Messrs. Lewis and Sherman.

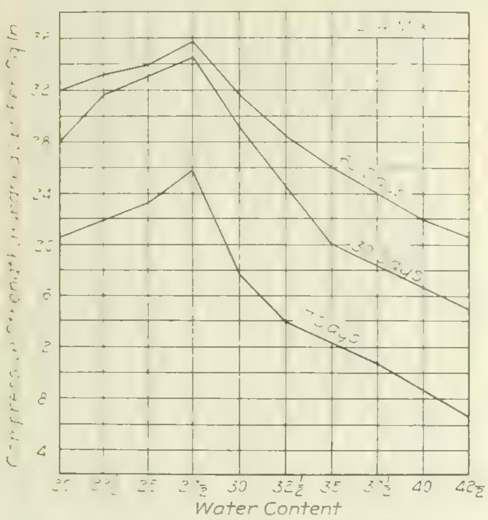


Fig. 1. Compressive Strengths of 6-in. Concrete Cubes Containing Various Percentages of Water.

DETAILS OF TESTS.

A 1:2:4 mixture was used in all of the tests. All materials were mixed dry, and the amount of water, exclusive of that required to moisten the sand and stone, was varied from 20 to 42.5 per cent of the weight of the cement. Four 6-in. cubes and four beams, each 2 ins. x 3 ins. x 3 ft. long, were made, the beams being reinforced with two ⅜-in. round rods and designed to carry a load of 274 lbs. placed at the center of the span. The cubes were broken at the end of 7, 30 and 60 days, and the beams at the end of 30 and 60 days. Mortar briquettes, containing 20 ozs. of sand and 12 ozs. of cement, were made and broken at the end of 7, 30 and 60 days.

The amount of water required to moisten the sand was determined by drying thoroughly a given quantity, weighing it, flushing it with water, draining off the excess water, and again weighing the sand. The difference between the first and last weights was considered to be the amount of water absorbed. Several trials were made, and it was found that the sand took up water to the extent of about 10 per cent of its weight. In a similar manner it was determined that the stone absorbed water to the amount of 2.1 per cent of its weight. In performing the above tests it was found that the sand weighed 80.5 lbs. per cubic foot and the stone 85.9 lbs. per cubic

TABLE I.—WEIGHTS OF MATERIALS USED IN CONCRETE SPECIMENS.

Mixture.	Stone, lbs.	Sand, lbs.	Cement, lbs.	Water, lbs.
20	120.7	56.5	35.1	15.2
22.5	120.7	56.5	35.1	16.0
25	126.2	59.1	36.7	17.7
27.5	131.6	61.7	38.3	19.4
30	131.6	61.7	38.3	20.4
32.5	131.6	61.7	38.3	21.4
35	131.6	61.7	38.3	22.3
37.5	131.6	61.7	38.3	23.3
42.5	131.6	61.7	38.3	25.3

foot. The proportions of 1:2:4 by volume were then changed to 1:1.61:3.44 by weight. The actual weights used in the various tests are given in Table 1.

The water in each case represented 2.1 per cent of the weight of the stone plus 10 per cent of the weight of the sand plus the percentage of the cement by weight indicated. The sand and cement were mixed until the mixture was of uniform color. The stone was then added and the mass thoroughly mixed, after which the water was added and the mixing continued. The concrete was tamped moderately in placing. The forms were removed from the cubes at the end of 24 hours, and from the beams at the end of 48 hours. All specimens were kept wet for 10 days, and then stored in air. At the same time a corresponding series of briquettes was made using the proportions shown in Table II.

The water required to moisten the sand is included in the gaging water. This was figured in a manner similar to that used in the case of the concrete for the beams and cubes. The briquettes were stored 1 day in the damp closet and the remainder of the time in water.

TABLE II.—WEIGHTS OF MATERIALS USED IN MORTAR BRIQUETTES.

Mixture.	Sand, ozs.	Cement, ozs.	Water, ozs.
20	20	12	4.4
22.5	20	12	4.7
25	20	12	5.0
27.5	20	12	5.3
30	20	12	5.6
32.5	20	12	5.9
35	20	12	6.2
37.5	20	12	6.5
42.5	20	12	Too wet to make.

RESULTS OF TESTS.

The beams and cubes were tested in an "Olsen" compression machine of 200,000 lbs. capacity. As the specimens were made in wooden forms they were carefully measured before crushing. Table III shows the results of the compression tests of the cubes.

The wide range in the strength of the various mixtures is very noticeable in Table III and in Fig. 1. This must be due entirely to the variation of the gaging water, for particular care was taken to keep all other factors the same. Considering only the question

TABLE III.—RESULTS OF COMPRESSION TESTS OF CONCRETE CUBES.

	Seven days.		Thirty days.		Sixty days.	
Mixture, per cent water.	Lbs. per sq in.	Relative strength.	Lbs. per sq. in.	Relative strength.	Lbs. per sq. in.	Relative strength.
20	2,052	0.82	2,777	0.80	3,194	0.89
22.5	2,173	0.87	3,132	0.90	3,305	0.92
25	2,277	0.91	3,277	0.94	3,388	0.94
27.5	2,500	1.00	3,486	1.00	3,597	1.00
30	1,722	0.69	2,875	0.82	3,166	0.88
32.5	[illegible]	0.56	2,416	0.69	2,666	0.74
35	1,222	0.49	1,977	0.57	2,388	0.66
37.5	1,097	0.44	1,819	0.52	2,173	0.60
42.5	652	0.26	1,500	0.43	1,888	0.52

Seven-day tests—one specimen.

Thirty-day tests—average of two specimens.

Sixty-day tests—one specimen.

of strength, the 27.5 per cent mixture appeared to be the most desirable. In breaking the cubes, a drier mixture than this was noticeably weaker, and a slight increase in the amount of water resulted in a decided decrease in strength. The 20 per cent cube at the end of 7 days was 0.8 as strong as the 27.5 per cent cube, and at the end of 60 days it was 0.9 as strong. On the other hand, the tests of the extremely wet mixtures did not give nearly as good results. At the end of 7 days the cube made of the 42.5 per cent mixture, which is of about the same consistency as is in use in practice (if anything a little drier), was approximately 25 per cent as strong as that made of the 27.5 per cent mixture. At the end of 30 days the difference was not so marked and at the 60-day tests the 42.5 per cent mixture had increased 190 per cent in strength, while the 27.5 per cent mixture had increased but 44 per cent, thus making the latter about twice as strong as the former, instead of four times as was the case at the end of 7 days. However, the 42.5 per cent mixture at this age was not quite so strong as the 20 per cent mixture at 7 days. Figure 1 shows graphically the relative strengths of the different mixtures. The strongest mixture (in each case the 27.5 per cent mixture) was taken as unity, and the strengths of the other specimens were expressed as percentages of the strongest. The effect of aging is shown very clearly in the flattening of the curve. This can be partially explained by the fact that the age renders more constant such factors as the personal equation and slight differences in manipulation or in storing, although undoubtedly the greater part of the gain in strength of the wet mixtures was due to the drying out of the excess water. Although the writer has no data upon which to base definite conclusions it seems reasonable to expect a further flattening of the curve, but it hardly seems possible that it would ever become a straight line—which would mean that the mixtures had all attained the same strength. Inasmuch as the forms were removed within a comparatively short time after pouring, and the concrete was subjected to heavy loads while green, the curves showing the relative strength at 7 and 30 days are of more interest to the engineer than those representing the strength developed in a period covering several months.

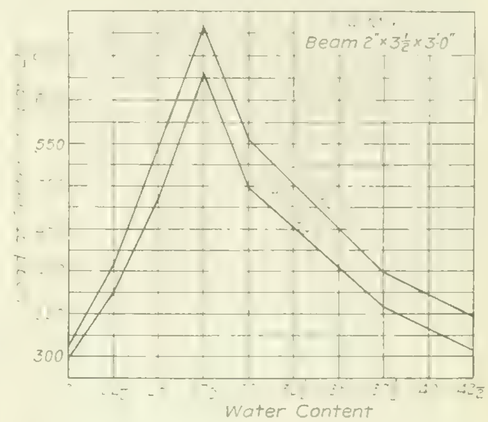


Fig. 2. Breaking Strengths of Reinforced Concrete Beams Containing Various Percentages of Water.

The results of the tests of the beams are given in Table IV, and are shown graphically in Fig. 2.

All specimens carried the load for which they were designed, but the extremely wet and especially the extremely dry mixtures had a very small factor of safety. As in the cube

TABLE IV RESULTS OF TRANSVERSE TESTS OF BEAMS.

Mixture, per cent water.	Thirty days Lbs. per sq.in.	Thirty days Relative strength.	Sixty days Lbs. per sq. in.	Sixty days Relative strength
20	[illegible]*	0.46	300	0.44
22.5	368†	0.58	410	0.60
25	[illegible]	0.74	550	0.81
27.5	637	1.00	[illegible]	1.00
30	507	0.80	560	0.82
32.5	457	0.72	500	0.74
35	417	0.66	450	0.66
37.5	360	0.56	400	0.59
42.5	313	0.49	340	0.50

Thirty day tests—average of three specimens.

Sixty-day tests—one specimen.

*One beam broken by premature removal from mould.

†Two beams broken by premature removal from moulds.

tests the 27.5 per cent mixture proved the strongest, both at the end of 30 and 60 days. The relative strengths of the beams which were made of concrete containing 27.5 to 42.5 per cent of water were practically the same as for the cubes of the same consistency. The drier mixtures did not give as satisfactory results when cast into beams. For example, the 20 per cent mixture when subjected to compressive stresses was 0.8 as strong as the 27.5 per cent mixture at the end of 30 days and 0.9 as strong at sixty days, but when tested as a beam the mixture was somewhat less than half as strong in both cases. In the cubes, the wet mixtures were extremely weak, but in the beams the dry mixtures were a trifle weaker than the wet ones. This may be due to the fact that a beam exposes more surface

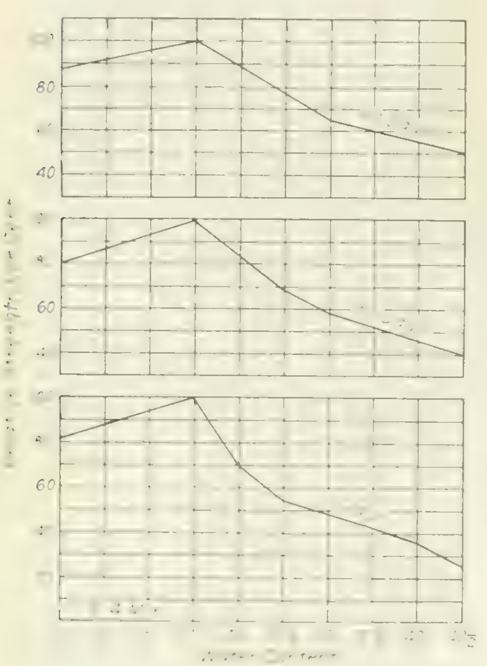


Fig. 3. Relative Strengths of 6-in. Concrete Cubes Containing Various Percentages of Water.

to the atmosphere per unit of volume than the cube. In this case, the former exposed about 24 sq. ft. per cubic foot; and the latter about 10. Undoubtedly this had some bearing upon the results.

The tensile strengths of the various mixtures are given in Table V.

These results verify those obtained in the other tests. The mixture which gave the maximum strength was a trifle drier, containing 25 per cent of water against 27.5 per cent for the tests of the beams and cubes, but the general relation of consistency to strength was the same. The wet mixtures seemingly developed their tensile strength faster and therefore showed up better, especially at 7 days. The increase in strength of the different mixtures was more uniform and for this

TABLE V.—TENSILE STRENGTHS OF BRIQUETTES.

Mixture, per cent water.	Seven days. Lbs. per sq. in.	Relative strength.	Thirty days. Lbs. per sq. in.	Relative strength.	Sixty days. Lbs. per sq. in.	Relative strength.
20	212	0.67	353	0.84	450	0.80
22.5	293	0.98	412	0.98	505	0.90
25	300	1.00	418	1.00	563	1.00
27.5	280	0.93	397	0.95	537	0.95
30	230	0.77	356	0.85	425	0.76
32.5	170	0.57	227	0.54	305	0.54
35	141	0.47	193	0.46	275	0.49
37.5	137	0.46	173	0.41	220	0.39

42.5 Too wet to make.
Seven-day tests—one specimen.
Thirty-day tests—average of three specimens.
Sixty-day tests—average of two specimens.

reason the relative strength was practically constant, regardless of age.

CONCLUSIONS.

These tests have demonstrated that the percentage of water has a direct bearing upon the strength. Nevertheless, it is the exception rather than the rule that the engineer concerns himself with the question of mixing beyond seeing that the proper proportions of sand, stone and cement are used. Careful tests are made of the cement and steel, both standard articles put out by firms which have reputations to maintain; on rare occasions, the water is analyzed; but no attempt is made to control the mixing. On some pieces of work the contractor is furnished the cement so that there will be no temptation to skimp on materials. He is then given the liberty to mix these materials as he sees fit. He may aim to get the maximum strength out of them, but it is probable that he will strive rather to keep the cost of mixing and placing at a minimum.

Of course, it must be admitted that the strongest mixture is not always the most desirable or economical. In preparing the specimens for these tests it was found necessary when making mixtures containing more than 27.5 per cent of water to increase the proportions 10 per cent in order to fill the moulds. This indicates an increase in density, which was obtained at a loss of strength. However, in some cases this is highly desirable. Often bulk and impermeability are the two requisite features. In reinforced concrete structures a moderately wet mixture is far more practical than a dry one; but it must not be forgotten that the use of wet mixtures increases the cost of materials. An increase in density means a corresponding increase in raw materials. The additional water weakens the concrete, and therefore a richer mixture or lower unit stresses must be used. The former is preferable, for the beams and columns in concrete building are always heavier than the corresponding members of steel or wooden structures.

If a wet mixture is used the engineer should be cautious in permitting the removal of the forms at an early date. It will be recalled that in some of the tests the wet mixtures were very slow to develop their strength. The use of such consistencies in practice would

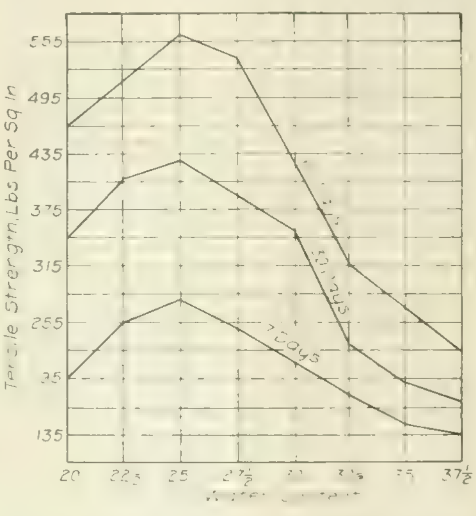


Fig. 4. Tensile Strengths of Mortar Briquettes Containing Various Percentages of Water.

necessitate extreme care. It might be a month or more before the forms could be struck with any feeling of security.

The question of consistency is not altogether a laboratory question. The increased cost of materials due to the use of wet mixtures may be more than balanced by the saving in placing; but the engineer should bear in mind that consistency has a direct relation to strength, and if he permits a wet mixture he should provide for the same in his design. In any event he should control the details of the mixing.

WATER WORKS

Some Water Works Engineering Mistakes.

Contributed by Dabney H. Maury, Consulting Engineer, Chicago, Ill.

Agreeing entirely with the views of the editors of ENGINEERING AND CONTRACTING, expressed in an editorial published some months ago, that a frank discussion of engineering mistakes by those who have made them ought to be useful and instructive to others, the writer has decided to describe a few of his own. A number of others might have been included; but the ones which follow, while none of them is in any way startling or sensational, are those which seemed most likely to convey useful lessons, for the reason that they have doubtless been repeated many times before and since the writer made them.

In 1895 it became necessary for the Peoria Water Works Co., of which the writer was then Engineer and Superintendent, to increase its water supply, and to do so as rapidly as possible. A detailed description of the way in which this was accomplished was published in Engineering News of Jan. 13, 1898, under the title "Supplemental Pumping Plant of the Peoria Water Works Co."

As will be seen from that description, this plant included, among other items, an open well, 11 ft. inside diameter.

In the construction of this well the earth was first excavated as far as the ground-water would permit, and a cast iron well shoe, built of circular segments, was bolted together, and on top of this well shoe were bolted segments of oak designed to widen the shoe and to serve as a foundation for the brick masonry wall of the well. Excavation was started inside of the well shoe, the masonry wall being built up on top of these wooden segments as the well sunk. It was the intention to sink this well to bed rock, seal the bottom with concrete so as to make a watertight pit, and then drive out push wells through apertures in the wall.

The writer, however, made the mistake of pumping from the inside of this well, part of the material being removed by the pumps and part by buckets which were filled by men standing on the bottom. The well sunk rapidly from 2 to 3 ft. a day until within 12 ft. of bed rock, when its progress was suddenly diminished and all efforts failed to get more than 4 ft. further. It was found that at about 8 ft. from bed rock the sand ran in under the shoe as fast as it could be removed.

An effort was then made to sink a steel ring 10 ft. in diameter by 5 ft. high by ½ in. thick on the inside of the well in the hope that this could be sent down to bed rock; but after much time had been lost in trying to keep ahead of the incoming sand, the attempt was abandoned. Oak wedges were then driven so as to close solidly the space between the ring and the wall of the well, and four 6-in. driven wells were sunk to bed rock. Four 6-in. push wells were also driven out through holes in the wall of the well, and all eight of these 6-in. wells were connected up to the centrifugal pumps and the plant was finally put in operation. It has been operated successfully, whenever required, during the past 19 years.

The reason why it was impossible to sink this well any further was that as the well approached the bed rock, the area of the belt of water bearing sand through which the water had to pass in order to reach the pump, was constantly growing less. At the same time the rate of pumping was constantly increasing. Consequently, when the point was reached at which the velocity of the inrushing water became so great that it dragged in the sand, this sand simply continued to run in as fast as it was removed, and the well naturally could sink no further.

On looking back at the work, this explanation of the trouble seems so simple and natural that the writer has wondered many times why he did not foresee the difficulty; but he has since seen the same mistake made so often by others that he feels encouraged to hope that this description of his own experience may prevent someone else from going wrong in the same direction. In his own practice, the lesson has been of much service, for it has led to the construction of a number of wells, in the design of which special attention was paid to the matter of preventing the inflow of sand, not only during the process of sinking the well, but also after the well has been finished and put in operation.

The particular well first described could have been sunk to bed rock at much less cost, either by the pneumatic process, or by excavating from the inside with orange peel bucket without pumping, or by sinking other wells outside of the large well and pumping from these to lower the water in the large well. As the sand was removed, the well would have sunk, provided the weight of the well were sufficient to overcome the skin friction. No more sand would have run into the well because of the fact that the water would have had no velocity to cause the sand to flow.

This mistake of the writer was at the time an expensive one for his client; but he has the satisfaction of knowing that the extra cost of this well was saved many times thereafter in other wells sunk in better fashion for the same client.

The writer made another mistake in connection with this same piece of work. The water furnished by the supplemental well was brought down by gravity to the main well, some 3,300 ft. away. For conveying this water, a 24-in. vitrified tile pipe was built. It was hoped that this tile pipe would cost much less than a cast iron pipe, and it was believed that the tile pipe could be made practically watertight. Tests of the watertightness of this pipe were described in a paper read before the Illinois Society of Engineers and Surveyors, in January, 1896.

The attempt to make the pipe practically tight was successful, but the expense was so great that the writer can now see that he should have made this pipe of cast iron instead of tile, for much of the pipe had to be laid in quicksand, and where a great deal of ground-water had to be taken care of, making it very difficult and expensive to make full cement joints in the tile, and to hold the pipe and the joint in place until the cement had set. Under these conditions, cast iron pipe would have actually cost less, laid in place, than the tile pipe.

Tile pipe has since been so often used for watertight work instead of cast iron, and so often with disappointing results, that it is hoped that a recital of the foregoing mistake may prove of service to others.

A third mistake entered into the design of a small water works, built about 20 years ago, and the writer's first complete water works installation.

This plant included an elevated tower and steel tank of about 60,000 gals. capacity, and for every part of that structure detailed working drawings were prepared. With the desire for originality too often displayed by young men, inventors, and others lacking the wisdom of practical experience, considerable pains were taken to design connections which were novel. It is safe to say that the like of some of these connections has never been seen in any other structure, before or since.

The working out of all of these details not only multiplied the time required for the design of the structure by three or four, but the novelty of the connections also added several hundred dollars to the contract price, for the bidders could not use their standard shop connections, but were compelled to figure on the writer's more expensive and less practical designs.

The tank still stands, and a recent inspection showed it to be in first-class condition. The total compensation received by the writer for his services in connection with this early job was so small that his conscience troubles him somewhat less than it otherwise would; but whenever he passes that way he views that tank from the train window with mingled emotions.

The lesson to be learned from this last mistake is that no engineer, if he has his client's interests at heart, can afford to overlook the contractor's viewpoint when he prepares his specifications.

Contractors must be expert in the practical details of construction, and especially in cheapening construction costs, or they cannot continue in business. If the young engineer wishes to learn more about these matters than he has ever learned before in the same length of time, let him, after he has prepared a rough draft of his plans and specifications, discuss these with a few of the more trustworthy and experienced contractors who are likely to bid on the work, and he will be astonished to find how many changes he can make to his client's advantage, without in any way impairing the design of the work or the validity of the contract.

If the young man will then not let his exalted ideas of his dignity as an engineer prevent him from accepting promptly and gratefully the practical suggestions of the contractors, he will show that he is worthy to receive much of the success which is almost sure to crown his future efforts.

The Smith System of Natural Slow Sand Filtration as Applied at Parkersburg and Proposed for Wheeling, W. Va.

The present article describes the so-called Smith System of Natural Slow Sand Filtration, with special reference to the existing plant of this type at Parkersburg, W. Va., and to the proposed plant at Wheeling, W. Va. The description of the system in general and of the Parkersburg plant in particular, which follows, is taken from a paper by Mr. L. E. Smith of Charleston, W. Va., the patentee, before the recent annual meeting of the Central States Water Works Association.

THE PARKERSBURG PLANT.

The Parkersburg plant was installed under the supervision of Mr. Samuel M. Gray, C. E. of Providence, R. I., and has been in successful operation nearly three years. Through low and high water, with but three-fifths of the plant completed, it supplied the city for 13 months, often being operated up to and above the guaranteed capacity of the entire plant. I am able to state that the result has been beyond my highest anticipations. It went through the great flood of March, 1913, unscathed and when fire threatened to destroy the city, with both pumps operating to their full capacity, supplying double the guaranteed amount, there was no perceptible deterioration of the product, nor was there any apparent injurious effect upon the plant.

Location and Installation.—A plan of the piping is shown in Fig. 1. The beds comprise five units, each unit consisting of two parts, the parts and units being separated 25 ft. by unexcavated portions of the bar, are located about 450 ft. from the West Virginia shore, in a sand bar in the Ohio River. An 18-in. gravity line leads from each of the units to a valve chamber on the shore, the effluent entering a manifold, to which is attached the 24-in. pipe leading to the pumps.

The guaranteed capacity of the plant is 4,000,000 gals. daily. There are 160 strainers, 5 ins. in diameter and 15 ft. long, each containing 9,400 slots. The maximum capacity of each strainer is 300,000 gals. in 24 hours. On that basis, the strainer capacity of the plant is 48,000,000 gals. daily. The strainers are placed in an excavation 5½ ft. deep; covered with coarse screened gravel to a depth of 15 ins. Over this bed is a layer of fine gravel, of about 3 ins. thickness, and above this is sand to the level of the stream bed. All sand and gravel is thoroughly washed before used.

Strainers are attached to 12-in. cast iron collecting pipes by means of a flexible joint, the point of contact being reduced to 3 ins. The collection pipes are connected with 20-in. reservoir pipes which, in turn, are attached to the gravity lines. There is thus provided a considerable body of water, upon which the pumps operate, positively precluding any possibility of suction. The bed of coarse gravel contains a vast amount of filtered water, upon which the strainers depend, which is another safeguard against suction. The water passes down through the sand at the rate of about 5 ins. per hour.

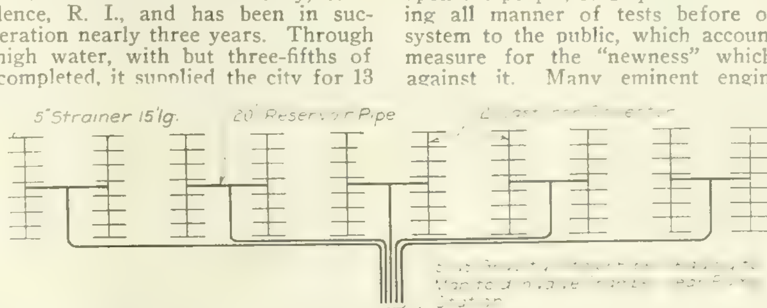
Backflushing.—Provision has been made for purifying the beds, should they become polluted. A system of backflushing is provided, by which any unit may be detached and backflushed, without interference with the pumps or the other units. If by any chance pollution should occur, chloride of lime may be applied by backflushing and the bed purified. After treatment with chloride of lime the bed is thoroughly backflushed and the chloride disappears. This has never been necessary at Parkersburg.

It may be advisable to backflush the beds if the waves and current do not keep the surface of the beds clean. There again the bed of coarse gravel comes into play, distributing the pressure from the backflushing to all parts of the beds, thus gently lifting the sediment, which is carried away by the current.

In this particular, the Smith System is different from the old slow sand system. The accumulation of sediment on beds of that system is scraped off when it becomes impervious. As it is not possible to backflush the old slow sand beds.

Actual Results.—There having been many claims of alleged purification systems foisted upon the people, so I spent several years making all manner of tests before offering the system to the public, which accounts in large measure for the "newness" which is urged against it. Many eminent engineers were consulted and practially all who gave the system fair consideration found no fault with it. If changes or further tests were suggested, they received full consideration.

There were the usual investigations by engineers at Parkersburg. Some recommended wells, others declared the chemical system was the only solution. All refused to give me a hearing until Mr. Gray was employed with the understanding that his recommendation would be final. In order to get to the bot-



Plan of Straining and Collecting Piping of the Smith System of Natural Slow Sand Filtration, Installed in the Ohio River at Parkersburg, W. Va.

tom of the matter, the Water Works Commission submitted to Mr. Gray the following questions relative to the Smith System: "Is the Smith System feasible and practicable for Parkersburg's water supply? Is it equal to or better than the filter system (mechanical) as recommended by Messrs. Fuertes and Fuller? (a) As to quality of water? (b) As to cost of construction? (c) As to cost of operation? Is it equal to or better than the well system as recommended by Mr. Knowles? (a) As to quality of water? (b) As to cost of construction? (c) As to cost of operation?"

The result of Mr. Gray's investigation was an unqualified recommendation of the Smith System. I here quote briefly from the report:

After a thorough study of the subject, based on personal examination of the ground, a careful study of the reports and other data furnished me, and after a careful consideration of the points and features contained in the various propositions. I have to say: That, in my opinion, the so-called Smith System of filtration is both feasible and practicable for furnishing an ample supply of water of a suitable quality for the City of Parkersburg for present needs.

The annual cost of operating a Smith System similar to the one proposed by Mr. Smith is comparatively small, as no chemicals will be required for filtration and no attention beyond what would be required for the pumping station.

As to the quality of water from the two systems: There is no doubt in my mind but that the Smith System of filtration, if properly built and operated, will furnish a better quality of water than that derived from a mechanical filter plant, for the reason that in operating a mechanical filter plant, even under the best of care, some of the chemicals will find their way through the filter and into the water supply, causing more or less trouble with steam plants, as has been the experience with this kind of works.

From a careful analysis of all the reports on the subject of well water . . . and from former experience, it is my opinion that water derived from this source will not be equal in quality to the Ohio River water derived from the Smith System. . . .

In all my experience I have found that ground water from driven wells obtained under conditions that exist along the Ohio River grows harder from year to year.

After careful study of the problem for supplying the City of Parkersburg with water, I recommend that you adopt the Smith System.

Cost of Operation of the Smith System.—No other system costs so little for operation. The pure water absolutely free from sediment of every character appears to act as a lubricant upon the pumps. There is practically no wear on packing. The fuel bill at Parkersburg is cut over half by the new system.

My estimated annual cost of operation of the Parkersburg plant was $10,180 or $7 per 1,000,000 gals. Messrs. Fuertes and Fuller estimated the cost of operation of a mechanical plant at $21,000 per annum or $16.44 per 1,000,000 gals. The actual cost of operation of the Smith System plant for one year was $7,326.88 which on a basis of 4,000,000 gals. daily is but $5.02 per 1,000,000 gals.

Quality of the Water.—Frequent tests as to purity were made as the works of installation progressed. After completion I made daily tests for one year. The city did likewise. Dr. L. O. Rose, City Chemist and Bacteriologist, at the expiration of nine months submitted a report to the city. The daily average number of bacteria for the nine months was 19. The highest average for any month was 34 and the lowest 9 bacteria. Dr. Rose said:

From the foregoing analysis it will be found, [illegible] amounted to nineteen per c.c. This is, without doubt, most extraordinary, when it is known that at times the raw Ohio River water contained as many as 25,000 bacteria per c.c. I desire to add that at no time have I found the "Colon Bacteria," commonly known as the "sewage germ," in the filtered water.

The foregoing paragraph concludes the quotation from Mr. Smith's paper.

RECENT ANALYSES OF PARKERSBURG SUPPLY.

The mayor of Wheeling requested information of the United States Public Health Service as to the results of examinations of the Parkersburg water supply. Dr. W. H. Frost, Passed Assistant Surgeon, in charge of the sanitary survey of the Ohio River, reported to Mayor Kirk the results of the bacteriological examination of samples of the Parkersburg water supply here shown in Table I.

THE PURIFICATION PROBLEM AT WHEELING.

In a report dated March 23, 1914, an improved water supply for the city of Wheeling was considered by the Filtration Commission composed of George A. Johnson, J. Richard Kommer and C. B. Cooke. The commission considered the practicability and cost of obtaining the new supply from driven wells, from the Smith System, and from rapid sand filters. The comparative costs per 1,000,000 gals. of the three systems, to yield 20,000,000 gals. per day, were estimated as follows by the commission:

System.	Total first cost.	Fixed charges on investment.*	Operation and maintenance.*	Total annual cost.*
Driven wells	343,600	2.82	6.81	9.63
Smith sand filters.......	662,280	5.45	3.56	9.01
Rapid sand filters.......	398,000	3,28	4.97	8.25

*Per 1,000,000 gals. of water.

Conclusions of the Commission.—The commission pointed out the fact that so far as cost is concerned the advantage rests with the rapid sand filtration project which, on a 20,000,000-gal. daily basis, is cheaper than the Smith System by about $5,500 per year, and cheaper than the driven well project by about $10,100 per year. These differences represent, at 5 per cent, capitalized values of $110,000 and $202,000.

We quote the final conclusion of the commission as follows:

It is our final conclusion that the best interests of the city will be served by the adoption of the rapid sand filtration process for the purification of the Ohio River water as herein recommended, for by its adoption there will be insured to the city the requisite amount of water at all times, which cannot be promised in the case of the other two projects. Furthermore, the filtered Ohio River water will be materially softer than the water from either one of the other two projects, the hardness of the filtered water being that which the consumers have long since been accustomed to. The filtered water will always be clear and free from suspended matter and color, which cannot be promised in the case of the Smith System; and there is absolute assurance that it will always be hygienically pure, which cannot be promised in the case of either the driven well supply or the supply from the Smith System.

Our final recommendation is that the city adopt the rapid sand filtration process for the improved water supply of the city of Wheeling.

TABLE I.—RESULTS OF BACTERIOLOGICAL EXAMINATION OF SAMPLES OF MUNICIPAL WATER SUPPLY OF PARKERSBURG, W. VA.

Sample No.	Date [illegible] 1914.	Bacteria per cubic cent. On gelatine at 20 deg. C.	On agar at 37 deg. C.	Bacillus coli present in—
691	July 7	..	15	10 cc.
762	July 14	9	8	10 cc.
[illegible]	Aug. 4	21	31	10 cc-1 cc.
1,097	Aug. 11	12	11	None in 10 cc.

Note.—All samples collected from tap in City of Parkersburg, promptly iced, shipped on ice to Wheeling, received at laboratory and examined within six hours of time of collection. Tests for bacillus coli always made in amounts of 10 cc., 1 cc. and 0.1 cc.

COMMENTS OF THE COMMISSION ON THE SMITH SYSTEM.

The commission, above mentioned, commented upon the efficiency of the Smith System in the report to which we have made reference. We have set forth Mr. Smith's claims, including his quotations from Mr. Gray, and in order to lay the opposite side of the case before our readers are here giving certain excerpts from the report of the filtration commission composed of Messrs. Johnson, Kommer and Cooke, which give the commission's estimate of the value of the Smith process of natural slow sand filtration:

It is the design in this system to allow a rate of filtration, downward through the sand and gravel overlying the system of perforated pipes of some 3,000,000 gals. daily per acre of filtered surface. In other words, it is expected that the downward velocity of flow of river water through the bed of sand and gravel will be at the rate of about 5 vertical inches per hour, under which conditions each square foot of filter surface will pass about 70 gals. of water daily.

The inventor evidently expects that from time to time the filtering material over the system of perforated pipes will become clogged with mud, silt and clay, and at such times fail to pass the requisite amount of water. In freeing the beds of this clogging material, the inventor forces water backwards through the system of perforated pipes, and upwards through the overlying layers of gravel and sand, this flushing water being applied at a rate of about 1 gal. per square foot per minute, according to our understanding. The application of a reverse current of water in this manner is expected to separate the clogging clay and mud from the sand grains and force it upwards and out of the filter bed, to be carried away by the water in the river flowing over the surface thereof. All these things are conjectural, however, as the filter beds are submerged in the river, and their action during such operation cannot be observed.

Being familiar with the manner in which rapid sand filters are cleaned by the application of a reverse current of water, we are of the opinion that such flushing facilities as above described are clearly inadequate to perform satisfactorily the desired function, the upward velocity of flow of the flushing water being only from one-eighth to one-twentieth that used with success in cleaning rapid sand filters.

For a 20,000,000 gal. daily capacity installation of the Smith System there would be required some seven acres of such filter beds, and owing to very uncertain river conditions, in addition to a first high cost, their construction would be a matter of no little difficulty. Even if successfully built, should they in the course of their operation become clogged and require scraping and resanding, the difficulties and cost surrounding such operations would be great. In the first place, such operations would have to be carried on during a low river stage, and the beds might become clogged and require renewal at a time when it manifestly would be impossible to get at them, due to unfavorable river conditions. The fact that the system is not capable of ready access, and that the normal workings of the beds are not open to the sight and consequently amenable to constant observation and control, is, aside from the high first cost of construction, one of the most serious objections to this system. In brief, after spending the money for the construction of a plant of this type, the city would be forced to trust to good fortune for its ultimate satisfactory performances. The fact that the only plant thus far constructed by Mr. Smith (Parkersburg) has been in service about two years and is apparently giving good satisfaction is not sufficiently convincing that it will not prove otherwise after it has been longer in service. With the facts before us we do not feel kindly disposed toward the idea of expending $650,000 on the installation of the Smith System at Wheeling.

COMPARISON OF SMITH SYSTEM AND REGULATION SLOW SAND SYSTEMS OF FILTRATION.

The following excerpts present the substance of the commission's estimate of the relative worth of the regulation slow sand filtration process and the Smith natural slow sand filtration process:

In the Smith System the filter beds proper are submerged in the bed of the river, are subject to disturbances from the scouring action of the river when in flood, and the clogging action of clay, mud and silt at other times, and are inaccessible and beyond accurate control. No part of their operation can be viewed and studied. On the other hand, in the regulation slow sand process, every part of the plant is open to the sight, easy of access, and constantly under observation and control. When the beds become clogged and require cleaning the clogged material is scrapped off. In the Smith System such clogging is relieved, so far as possible, by back flushing, but such back flushing may result disastrously in bringing about impaired efficiency of the filtering process immediately following such operations.

We have made a rough estimate of the cost of building a modern slow sand filtration plant for the city of Wheeling, the same to have a capacity of 20,000,000 gals. per day. We find that to build such works following the same general lines as those observed in connection with the Smith System, the rapid sand filtration system and the driven well system, would, in the case of a modern slow sand filtration process cost about $670,000, which sum does not include the cost of land for the filters, which would, in all probability, could a suitable site be obtained, cost upwards of $75,000 more. In brief, the total cost of filtered water from a plant of this type, including charges on the cost of construction, operation and maintenance, would amount to about $10 per 1,000,000 gals. of water filtered. It is seen, therefore, that such a project would be the most expensive of all the projects under consideration, but the high charge would be necessary on account of special features of construction and operation in a plant of this type which could be counted on to efficiently purify the Ohio River water.

The Smith System does not in any measure even approach, in points of refinement, in construction and operation, the slow sand filtration system we are discussing. We cannot see, therefore, where the Smith System is applicable to the local problem on any ground whatsoever. Its only hope for success lies in the possibility that the bulk of the water delivered from it might be drawn from underground sources, and not by filtration of the Ohio River water downward through the sand and gravel overlying the system of perforated pipes. In other words, the only grounds upon which the Smith System could be taken seriously in connection with the local problem would be to consider it in the light of a well supply; and in the face of the facts and figures we have presented, referring to the cost of developing a driven well supply, wherein the water from such wells would be at least the equal, and probably the superior, hygienically, of the water obtained from the Smith System if it all came from underground sources, although somewhat harder, it would appear that the preference between the driven well system and the Smith System distinctly lies with the former.

In brief, we find that the whole problem narrows itself down to the relative desirability, on practical grounds of the driven well project and the rapid sand filter project.

PREFERENTIAL VOTE ON FILTRATION SYSTEM AT WHEELING.

The final recommendation of the commission was that the rapid sand filtration process be adopted as stated elsewhere in this article. There is now something of a controversy on in Wheeling between the advocates of the Smith and rapid sand systems. A preferential vote was taken in July and the people declared for the Smith system. It is planned to vote on the bond issue this fall. Editorial comment on the unwisdom of submitting this question to popular vote is made on the first page of this issue.

Resonable Requirements in Water Filter Performance.

Cities utilizing for their public water supply a raw water polluted by sewage invariably have very high typhoid rates. The installation of filter plants to purify such polluted supplies almost without exception effects a prompt and remarkable reduction in the typhoid rate. This reduction is usually so great that municipal officials are satisfied that their water supply is perfect when in reality there is still something to be desired. When a city with typhoid fever rates consistently above 100 deaths per 100,000 population has a reduction coincident with the installation of a filtration plant to a rate between 20 and 30, there is good ground for general rejoicing because of the undeniable saving of human lives. Nevertheless, the raw water may be of such a character that an unreasonable burden is imposed on the filtration plant, and under such circumstances, in spite of fair efficiency, the plant delivers an effluent which is unsafe at times.

The present article, which is based upon a report by Dr. Allan J. McLaughlin, Chief Sanitary Engineer and Director of Field Work of the International Joint Commission, at a recent conference of the commission, relates to the characteristics of a safe drinking water and the requirements which reasonably may be exacted in filter plant performance.

With the general sanitary conditions which pertain in American cities and a safe public water supply, there is no valid excuse for typhoid rates above 20 deaths per 100,000 population. There is excellent evidence to show that if all the water-borne typhoid were eliminated in northern cities the rate for typhoid fever would be less than 10. As a matter of fact, there is a group of American cities which is fast approaching European cities in the matter of low typhoid fever rates. These are the cities which have gone farthest in making their water supply safe, and while their yearly typhoid fever rates are not always expressed in a single figure, their rates are usually below 12. In these cities with safe water supplies the general sanitary conditions, exclusive of water supply, are not conspicuously better and in some instances are very much worse than those found in cities with polluted water supplies and high typhoid fever rates.

There is a large group of cities in which, following the substitution of a filtered for a polluted public water supply, the rates have been greatly reduced, but still remain too high. These cities should not be satisfied with typhoid fever rates of from 15 to 30, but it behooves them to make a searching investigation to determine whether the raw water imposes an unreasonable burden on their filtration plant, or if their plant is efficiently operated and delivering a safe water at all times.

This brings us to the question of what is safe drinking water. In order to say that a drinking water is hygienically safe one must be assured that it contains no pathogenic bacteria. The efficiency of water purification plants varies from day to day and from hour to hour, and an opinion upon the absolute safety of a given water supply can not be rendered unless many bacteriologic analyses, made at short intervals during each 24 hours, show an absence of B. coli. While an absolute dictum is thus most difficult to secure, it is not difficult by daily bacteriologic analysis to determine that a water does or does not give a reasonable index of safety. Instead of attempting to find the germs of typhoid fever, Asiatic cholera, and dysentery in water, it is the custom to accept the presence of B. coli as an index of pollution with sewage, for the reason that the chances of finding B. coli are very much better than the chances of finding the specific germs in the small quantity of water examined.

When we consider the grossly polluted water supplies used by many of our large cities until recent years, we must admit that even if the present effluents from filter plants do not show constant absence of B. coli, still they must be classed as reasonably safe, or relatively safe water.

In order to secure statistics from some of our largest filtration and purification plants a circular letter was sent out to about 40 cities. About 15 responded, and in most instances the statistics covered at least one year. The list included mechanical filtration, slow sand filtration, precipitation and disinfection, and disinfection alone. Table I shows the average number of B. coli per 100 c.c. in both raw water and filtered or treated water, as indicated by the replies received.

Following a sanitary survey of the cities and towns in the basin of the Great Lakes, Dr. McLaughlin recommended that a standard for filtered or treated water be established which should be a minimum requirement for the prevention of the spread of water-borne disease in interstate traffic. The Surgeon General of the Public Health Service appointed a commission in January, 1913, to fix such a standard. The report of this commission will be published soon. The majority of the members favored a standard of four negatives out of five 10 c.c. tests for B. coli. Dr. McLaughlin expressed this standard in a different way, recommending a standard of not more than 2 B. coli per 100 c.c. of water, taking the average of many samples by Phelps' method.

Allowing a sufficient margin of safety, filter plants with a decent raw water should produce effluents of less than 2 B. coli per 100 c.c. and it is the opinion of Dr. McLaughlin that a modern water purification plant which delivers an effluent which has more than 2 B. coli per 100 c.c. is either inefficiently operated or is dealing with a raw water which imposes an unreasonable burden upon the plant. Accepting tentatively the standard of less than 2 B. coli per 100 c.c. as a good drinking water, although perhaps not an ideal or a safe drinking water at all times, the results indicate that the standard is attained by both rapid and slow sand plants, even with a very bad raw water. Cincinnati and Washington, D. C., are good examples of each type. Close examination of the daily records at these cities shows that while an excellent average is attained for the year, there are periods when the capacity for purification seems to be overtaxed by the very bad raw water. At Cincinnati the use of hypochlorite seems to compensate for the deficiency in purification by the standard process, but in Washington the excellent general average of 1.4 is only attained by the almost perfect purification effected during periods when the raw water is fairly good.

TABLE I.—AVERAGE NUMBER OF B. COLI PER 100 C.C. IN RAW AND PURIFIED WATER IN 12 AMERICAN CITIES.

City.	Number of samples.	Type of filtration.	B. coli per 100 c.c. of— Raw water.	Filtered or treated water.
Toledo, Ohio	342	Mechanical filtration.	804	0.02
Minneapolis, Minn.	418	"	75	.1
Grand Rapids, Mich.	365	"	92	.3
Birmingham, Ala.	205	"	196	1.0
	174	"	400	.2
Cincinnati, Ohio	240	"	1,175	1.4
Binghamton, N. Y.	420	"	59	1.2
Columbus, Ohio	365	"	606	1.3
Washington, D. C.	348	Slow sand	2,501	1.4
Providence, R. I.	600	"	732	4.3
Reading, Pa.	138	"	68	5.8
Baltimore, Md.	306	Alum and hypochlorite	1,349	2.5
Richmond, Va.	237	"	460	8.0

There is a strong tendency in America to accept any raw water, however bad, as a source for a municipal filtration plant. This often imposes an unreasonable burden and responsibility upon the water purification plant. Now filter plants are not infallible. They are mechanisms which must be properly constructed and efficiently operated under careful bacteriologic control in order to secure a safe effluent. They are operated by human agency and subject to the results of human error. It is true that properly constructed and efficiently operated filter plants can produce safe water from a very bad raw water, especially by the use of hypochlorite or liquid chlorine as an adjuvant. The responsibility of effecting such purification every hour of every day in the year is unreasonable and unfair. Many plants are now struggling with a raw water of such a character that a safe effluent is only obtained at the price of eternal vigilance, perfect operation every day in the year, and the free use of auxiliary chemicals. The raw water demanding such extraordinary treatment is, like a sword of Damocles, constantly threatening disaster. There is no margin of safety under such conditions.

Dr. McLaughlin believes that a sufficient margin of safety should be given to all filter plants by reducing the pollution of the raw water to a point where it would not impose an unreasonable burden or responsibility upon the plant. He believes that in reckoning the bacterial purifying capacities of filtration plants hypochlorite or liquid chlorine should not be considered, but that a raw water should be furnished of such a character that the plant could turn out constantly a safe effluent without the aid of chlorine. This would reserve the chlorination as an additional margin of safety for use in extraordinary fluctuations of the raw water or during accidents to the plant or interruptions in its ordinary efficiency.

Object and Limitations of Bacteriological Examination of Water.

The general aims of the bacteriological examination of water are well understood by the well informed water works superintendent. His knowledge of the subject rarely extends, however, beyond the bare generalities. This is due to the fact that particularized discussions are usually too technical to be of help to the superintendent. Such discussions usually are encumbered with references to special and alternative tests, kinds of culture media and types of testing apparatus, in such a manner as to make the ultimate effect of the discussion exceedingly confusing to all but bacteriologists. For the above reasons we are presenting herewith an unusually lucid and brief exposition of the aims and limitations of the bacteriological examination of water. The following matter is from a report by Dr. W. H. Frost, Passed Assistant Surgeon, United States Public Health Service, in charge of the sanitary investigation of the Ohio River, to Mr. H. L. Kirk, mayor of Wheeling, W. Va., with reference to the examination of certain public water supplies drawn from the Ohio River. The appended notes call attention to the object and aim of bacteriological examinations of water supplies in general, and to their limitations, especially in affording conclusions from a small number of examinations.

The greatest danger to be feared from drinking water is pollution with disease-producing germs. Bacteriological examinations therefore afford the most direct and significant evidence available concerning the sanitary quality, or in other words, the safety of a water supply. Even this evidence is, however, rather indirect. All surface waters, all ground waters except those from very deep and well-protected sources, and even freshly-collected rain waters, show the presence of more or less numerous bacteria. The vast majority of these bacteria are altogether harmless to the human body, and even when disease producing germs are present they are usually in such exceedingly small proportion as compared to the harmless varieties that it is rarely possible to isolate the actual disease-producing germs and so prove their presence by direct evidence, this being the more difficult because the disease-producing bateria are generally more delicate and more difficult to cultivate under laboratory conditions. In view of these difficulties, practical water examinations aim not to isolate disease-producing bacteria, but to determine the numbers of bacteria belonging to three general groups:

1. Bacteria developing on standard gelatine plates at the temperature of 20° Centrigrade (about 65° Fahrenheit) within 48 hours. This group of bacteria includes a large proportion of harmless varieties which may be found in water which is quite free from dangerous pollution. In general water supplies taken from rivers and efficiently purified by filtration should not show more than 100 bacteria per cc., as determined by this method.

2. Bacteria developing on standard agar-agar at 37° Centigrade (the temperature of the human body). This group of bacteria includes a larger proportion of the varieties which normally live in the animal body and whose presence in water indicates probable pollution with discharges from the animal body. Really good, pure water supplies from deep wells or from efficient filter plants will ordinarily show not more than 10 to 50 bacteria per cc., as determined by this method—frequently a smaller number.

3. Bacteria belonging to the so-called "Bacillus Coli" group. The bacteria of this variety, so far as known, develop naturally only in the intestines of man and other warm-blooded animals; they do not develop, but tend to die out in water or in the soil. The presence of such bacteria in water is proof that the water has been polluted with the intestinal discharges of some of the lower animals or even human intestinal discharges. This is not only the most disgusting but by far the most dangerous kind of pollution to which drinking water is subject, since typhoid fever and other dangerous and common water-borne diseases are caused by bacteria closely related to the bacillus coli group and discharged from the intestines of infected people. All unpurified surface water (from rivers, shallow wells, etc.) is contaminated with animal discharges to a greater or less extent. Water from well-protected deep driven wells should never show the presence of such bacteria, and efficiently purified river-water should show, at the most, only occasional traces of very slight pollution. To determine the *degree* of pollution with intestinal discharges the test for bacillus coli (which is somewhat complicated) is made in varying amounts of the water examined, for example, in 10 c. c. (about two teaspoonfuls), 1 cc., 0.1 cc., 0.001 cc., etc. Evidently the smaller the amount of water which shows the presence of such bacteria, the more numerous they are. The largest amount commonly tested from a single sample is 10 cc. The test in this amount should usually be negative in well-purified river water; not more than 10 per cent to 30 per cent of the samples showing presence of bacillus coli. The test in 1 cc. and in smaller amounts should be practically always negative. This standard is based on experience with many municipal supplies obtained by filtration, and is well maintained by the best of such supplies.

To form an accurate estimate of the quality of a water-supply it is generally necessary to continue examinations over a considerable period of time. Rivers, for example, vary greatly from time to time; and purification plants of all kinds are apt to show the same kind of variations, requiring the most constant attention to maintain a uniformly good quality of effluent; also wells which are subject to pollution with surface water are very often contaminated only occasionally, giving fairly pure water in the intervals. The fact that a sample of water is polluted is always of definite significance, for even if this is merely a rare, accidental condition, it proves the possibility and danger of pollution. On the other hand the fact that a water supply, examined [illegible] shown no pollution. does not prove that the supply is at all times safe.

In general, conclusions as to the safety of a water supply should be drawn not solely from the results of examinations of samples, but with due regard to knowledge of the source of the supply and the sources of pollution to which it is exposed.

The Use of Filter Wells and Filter Cribs Along the Ohio River.

There are in use several methods of filtration by the various consumers of Ohio River water, namely, the well system, the crib system, slow sand, rapid sand and the Smith system. These systems were described by Mr. C. E. Cooke, city engineer of Wheeling, W. Va., in a paper before the Central States Water Works Association. The well and crib systems are here described from information taken from that paper. The Smith system is described in a separate article in this issue.

THE WELL SYSTEM.

The well system either takes its water from wells drilled in the sand and gravel bars forming the bed of the river or from sand and gravel banks along the river, and in either case, the wells secure a large percentage of ground water. The wells are usually drilled wells with the casing driven down to or into the bed rock a short distance. The bottom of the casing is drilled with holes varying in diameter from ¼ in. to ½ in. Some of the well systems have a solid piece of casing at the bottom of the wells and then a perforated pipe above this solid piece. The water is either allowed to flow by gravity from the wells to the pumping station well and is then lifted by steam pumps to the consumer, or is lifted by compressed air out of these wells into a sub-basin and is then pumped to the consumer by steam pumps. The local conditions govern the method of furnishing the water used. The casing in the wells varies in diameter, as does the size and number of the perforations used. The well system generally supplies a water that is hygienically pure, but the water is nearly always harder than the river water, and, in some cases, wells have been known to show contamination after some years of use, as well as to show a considerable decrease in the quantity derived from the wells. Yet in some localities along the river, especially where the consumption is small, the well system seems to give satisfaction.

THE CRIB SYSTEM.

The crib system is usually built of timber, brick or concrete near the bank of the river with the water flowing in and through the sand and gravel with which the crib has been filled. The river water filtering through the said sand and gravel to the intake pipe of a pumping station and from there is pumped to the consumer. The danger of contamination is great in the crib system and the mud and silt are drawn through and into the sand and gravel, causing the crib to become clogged and the supply greatly, if not wholly, cut off. The crib is usually entirely submerged so that it is not readily accessible and so cannot be cleaned; thus the danger of contamination is great, and after contamination is found to exist, the only remedy is to abandon the crib or to cofferdam it and remove the sand and gravel, which is necessarily an expensive process and causes the consumer to use the raw river water during the execution of the work, thus contaminating the mains and services and opening the way for typhoid fever or other water-borne disease.

Contract System of Road Maintenance.—Several parishes in Louisiana maintain their roads by the contract system. Specifications are prepared and contracts let each year for the complete maintenance of the roads within the district specified. For the most part the work consists of repairing wooden bridges and dragging and repairing earth roads with a blade road machine. A similar system of maintenance is also in vogue in several counties in Mississippi.

GENERAL

Methods and Plant Employed in Sinking a Vertical Shaft at the Palms Mine, Bessemer, Mich.

Before the sinking of the shaft at the Palms mine of the Newport Mining Co., at Bessemer, Mich., was begun, the management made a detailed comparison of the advantages and disadvantages of inclined and vertical shafts. An inclined shaft would have the disadvantages of rails, back runners, skip wheels, axles and boxes, and the expense and trouble of axle lubrication, and of frequently replacing supports for ropes; longer ropes would be required and the skips would have to travel a greater distance and at limited speed. There would be a constant and considerable expense for the upkeep of the shaft and its equipment. A vertical shaft in the foot-wall would have only the disadvantages of longer crosscuts from the ore body to the shaft, and of the greater distance of transportation; but with transportation by electricity, distance is a small consideration. Accordingly a vertical shaft was decided upon, to be lined with steel and concrete. The construction plants and methods employed in sinking this shaft were described by Mr. Frank Blackwell of Ironwood, Mich., Chief Engineer, in a paper before the recent annual meeting of the Lake Superior Mining Institute which we here reprint.

DESIGN OF SHAFT.

The shaft, shown in section in Fig. 1 and plan in Fig. 2, is divided into five compartments; a cage compartment 6 ft. 2 ins. by 10 ft.; two skip compartments 4 ft. 10 ins. by 6 ft. each; a ladder compartment 3 ft. 8 ins. by 4 ft. 10 ins. and a pipe and counterweight compartment 3 ft. 8 ins. by 4 ft. 10 ins. It is 10 ft. 10 ins. by 17 ft. 6 ins. in outside dimensions. The wall plates, 17 ft. 6 ins. long, and

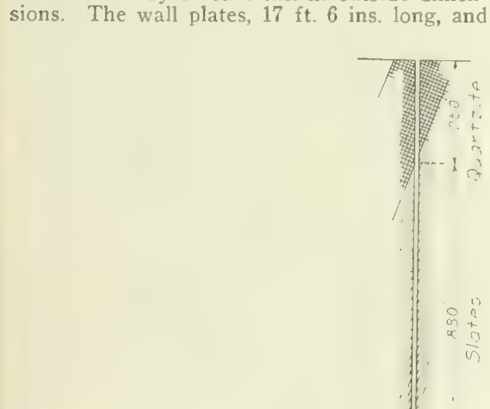


Fig. 1. Cross Section Through Shaft at Palms Mine of the Newport Mining Co., at Bessemer, Mich.

the end pieces and the two dividers, each 10 ft. long, are 5-in. 18.7-lb. H sections. The other two dividers, 4 ft. 10 ins. long, are 4-in. 13.6-lb. H sections. The eight studdles are 3-in. by 3-in. by ¼-in. angle iron. Most of the sets are placed 8 ft. apart center to center. Because of the heavy ground encountered several sets are placed 6 ft. apart, and a few of them 4 ft. The wooden guides are 5¾ ins. by 7¾ ins.; two of them are strengthened by 7-in. channel iron.

EQUIPMENT FOR SHAFT SINKING.

The temporary headframe used was high enough so that the bucket of rock could be

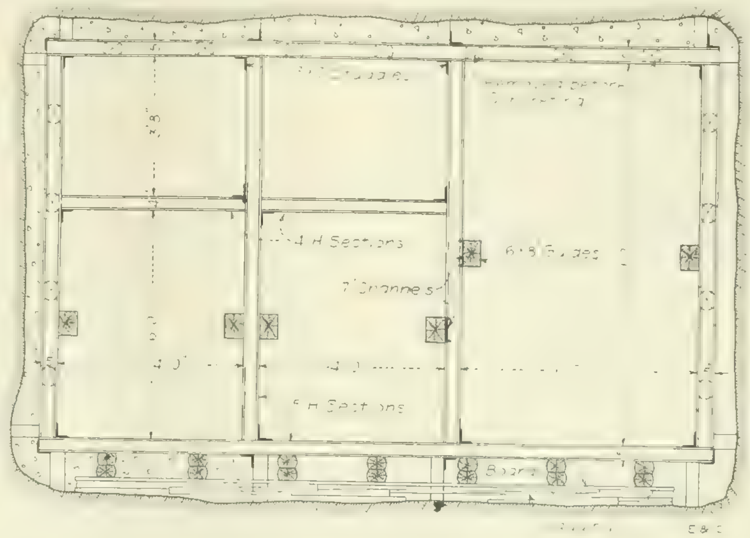


Fig. 2. Plan of Vertical Shaft at the Palms Mine.

dumped into a car 18 ft. above surface. After a bucket, see Fig. 3, was hoisted, a counterweight door was let down and the car run upon it. A chain was hooked to the bottom of the bucket so that when the bucket was lowered it dumped its contents into the car. The quartzite in the shaft was dumped from the trestle and reserved for concreting, see Fig. 4. For crushing the rock for concreting a Gates gyratory crusher No. 2, driven by a 20-HP. motor, was used.

Near the shaft was the shop, see Fig. 4, where the drills were sharpened and the drilling machine repaired. For sharpening the drills, an Ingersoll-Rand No. 5 Leyner drill sharpener was used. The die accompanying this machine very easily shanked the drills for use in the jack-hamers. The bits ranged in size from 1¼ to 1¾ ins. At one end of the shop was located a small dry and, conveniently near, a powder house.

In the temporary engine house was a double-drum double-gear-reduction electric hoist. The drums were 40 ins. in diameter, and had 30-in. faces, and were designed for a total load of 6,000 lbs. with an average rope speed of 600 ft. per mintue. The motor was 70-HP. with a speed of 550 r.p.m. This operated the two 26-cu. ft. rock buckets in the two end compartments of the shaft with a ¾-in. rope usually in balance. Here was also located a geared single-drum 50-HP. electric hoist, with a drum 2 ft. 6 ins. both in diameter and in face, which operated with a ¾-in. wire rope a light cage for timbermen in the middle compartment of the shaft. The same engineer fired a small boiler which heated the entire surface equipment for shaft sinking.

For ventilation, a 12-in. pipe, which still remains in the shaft for the cage counterweight, was connected to a 7½-HP. electric fan. The pipe extended down to within 15 or 25 ft. of the bottom of the shaft. Immediately after the blasting, compressed air was blown into the shaft through a valve on the surface, and the fan was started. The smoke and gases were drawn through the fan in this way for about half an hour.

To the bottom end of the air line was connected a flanged fitting with eleven ¾-in. valved outlets for hose connections. Before blasting, this fitting was replaced by a flanged reducer having a ¾-in. valve opening. The timbermen also used this for a hose connection for running the auger machine.

SINKING THE SHAFT.

Forty holes were drilled per cut; 10 Ingersoll-Rand jack-hamers and four spares with ⅞-in. hollow hexagonal steel were used. In the soft slates 8-ft. holes were drilled. During the drilling the holes were cleaned out with a blow-pipe. This was found to be indispensable for rapid drilling. When the drilling was nearly completed, only about four of the machines were running; the other men were preparing the explosives and removing air hose.

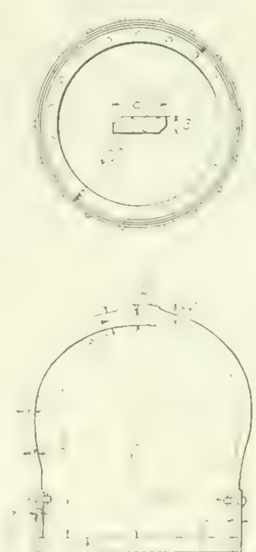


Fig. 3. Bucket Used for Shaft Sinking at Palms Mine.

The blasting box used was a paraffined pasteboard box 9 ins. by 3½ ins., and 1½ ins. deep. With an iron punch, holes just large enough for a fuse to fit tightly were made in the sides of this box near the bottom. In the first boxes used, a positive wire was led through one end and a negative through the other. The ends of these were connected with a one-ampere fuse. Two of these boxes were used at the same time to blast a whole cut. Two positive wires, one for each box, of copper, No. 14 gage, were strung from the surface and the two negative wires were connected to the air pipe.

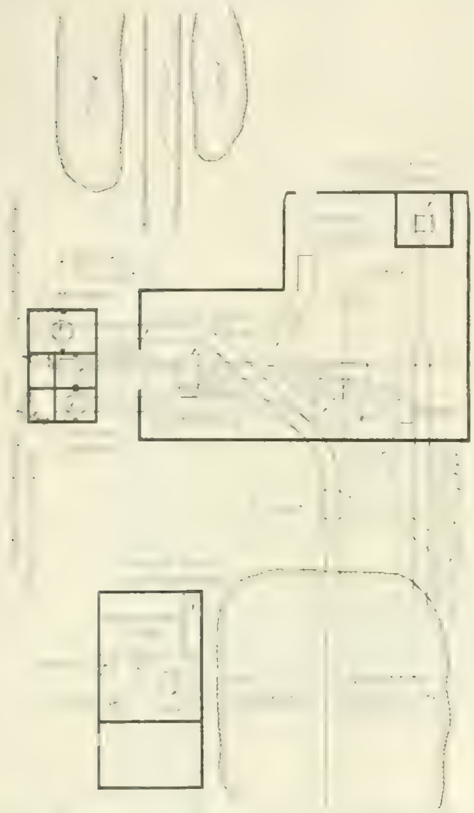


Fig. 4. General Layout of Construction Plant and Materials at Surface of Palms Mine Shaft.

After fuses of proper length were inserted through the holes in the box, a small amount of a mixture of FF rifle and ordinary black blasting powder was strewn over the one-ampere fuse, and the box was covered with a wooden lid. When the men reached the surface they could determine positively by means of a galvanometer whether they had made the wire connections properly, and whether the circuit was closed. Then the 250-volt current was thrown on, and the one-ampere fuse

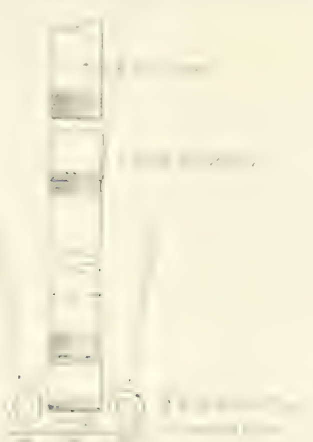


Fig. 5. Detail Showing Connection of Air Line Used in Sinking Palms Mine Shaft.

burned and ignited the powder, and this, in turn, ignited the fuses. If only a few of the fuses spit fire at first, these in turn ignited others, and almost instantaneously all the fuses threw fire across the inside of the box, so that it was almost impossible for any one to miss fire. The fuses were cut to such lengths that only one hole went off at a time.

However, too much labor was required to prepare these one-ampere fuse boxes, so that later an electric blasting squib was used to ignite the powder in the box. A squib was placed through a hole at each end of the box, two being used to insure the igniting of the black powder. The two boxes were connected in series, with but one No. 14 positive

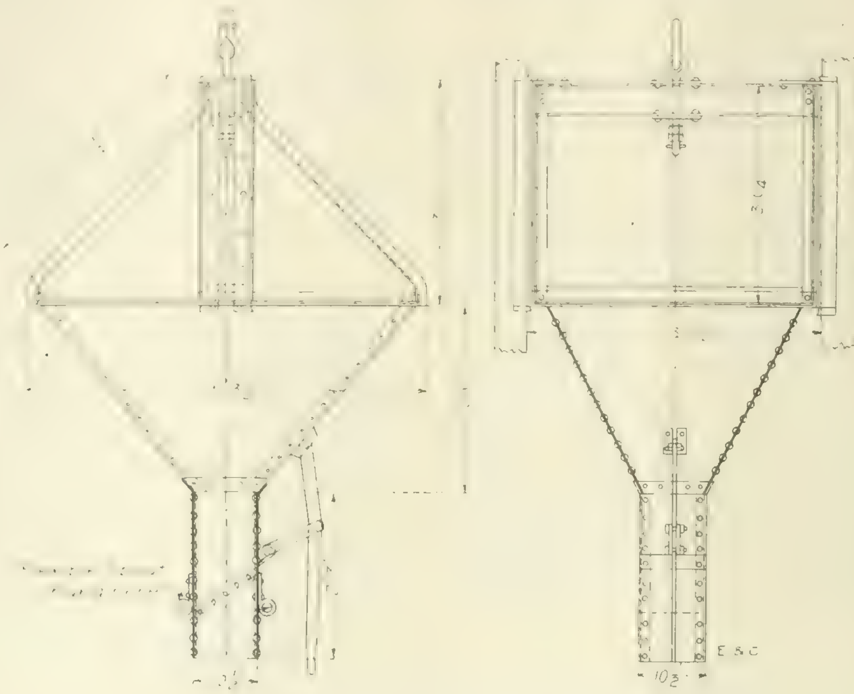


Fig. 8. Detail of Hopper for Lowering Concrete in Palms Mine Shaft.

copper wire from surface, and with the negative wire connected to the air pipe. Finally, a Du Pont delay electric fuse-igniter was used in place of the squib.

Du Pont 80 per cent gelatin 1 in. by 8 ins. was used for blasting. For a 7-ft. cut from 250 to 300 sticks were used; for a 5-ft. cut, from 200 to 250 sticks were used.

After the blasting, when the smoke had been blown out, the miners cleaned down the sets, trimmed the sides, and began mucking. Toward the end of the mucking some of the men used one bucket to lower the hose, machines, tools, etc., for the next cut, while the other men picked the bottom thoroughly and finished mucking with single hoisting.

PLACING THE SETS.

Some of the steel sets were riveted together on the surface. Where the rock was sufficiently hard so that a distance of 14 ft. underneath the last set was available, the set was lowered entire and swung into place. Shoes on the two lower corners guided it

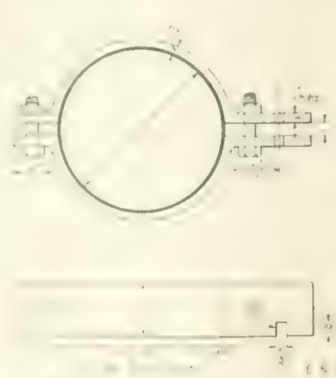


Fig. 6. Detail Showing Method of Supporting Pipe in Palms Mine Shaft.

through the shaft. Four one-ton duplex chain blocks were used for swinging it into place. To each corner of the set was fastened a ½-in. sling chain about 3 ft. long, with a 5-in. ring on one end and a 3-in. ring on the other, and to these the hooks of the chain blocks were attached. If the distance under the last set in the shaft was less than 14 ft.

the sets were lowered in parts and bolted together in the shaft. For blocking the sets a supply of wood sprags of different lengths was always ready on surface for immediate use.

When the solid rock was more than 8 or 9 ins. from the steel sets, 4-in. tie timbers were placed vertically 4 ins. outside the sets and about 2 ft. apart. Between the steel sets and these timbers 4-in. wood blocks 12 ins. long were placed. One-inch rough boards were placed horizontally outside the verticals to act as outside forms for pouring concrete. Lagging was filled in between the boards and the solid rock. When the rock was less than 8 or 9 ins. from the sets, 4-in. flat timbers were placed between the flanges of the H-section sets, and lagging was placed behind to the rock. This lagging was left until concreting time, when it was removed and hoisted to the surface. In the two ends of the shaft the rock was from 2 to 8 ins. from the steel sets for nearly the whole distance. In the other two sides the rock was 9 ins. or less from the wall plates for about one-half the whole distance.

A length of pipe was connected to the bottom end of the air line as follows, see Fig. 5: To the top end of the section that was to be lowered, a coupling was fastened very loosely

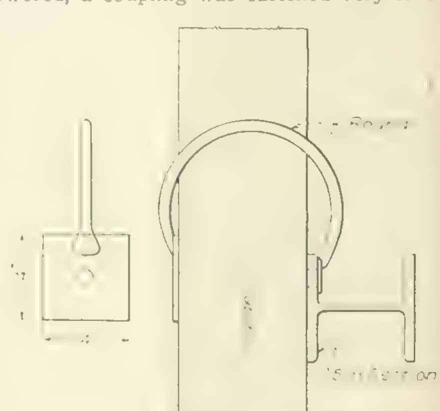


Fig. 7. Device for Lining-up Guides in Shaft.

by a very few threads, and to the bottom end was attached a temporary coupling with the socket of a ball-and-socket joint. The pipe was lowered underneath the bucket with a ½-in. chain. A clamp kept it from slipping. The plate of the ball-and-socket joint was supported underneath the air line by two chain blocks. The lower end of the section of pipe was swung over upon the plate by hand. The chain blocks raised the section of pipe up to the end of the air line. Then with a few turns of the loose couplings by hand the connection was quickly made. The couplings were tightened with chain tongs. In the couplings every 40 to 50 ft., ½-in. air connections were made.

To lower a section of the 12-in. flanged pipe the bucket was removed from the hoisting rope; eye-bolts were inserted in three of the holes of the flange, and rods connected these eye-bolts to a ring in the clevice at the end of the hoisting rope. At the end of the 12-in. line two chain blocks were hung and by means of two ¾-in. wire-rope slings the section of pipe was taken from the hoisting rope and placed in the proper position for connection to the 12-in. line. Figure 6 shows the methods of supporting this pipe in the shaft. The slot engaged the flange of the H-section sets. Every alternate one of these rested upon the wall plates and the others upon the dividers.

The guides were lowered either in or underneath the bucket. When the proper depth was reached a sling chain was fastened around the guide and a chain block hung to the set swung it into place. In lining the guide a 2x2-in. wooden gage was placed between the wall plate and guide. This gage was supported by two hooks hung over the flange of the H-section divider or end piece. The bolt holes were bored after the guide was lined up. In order to start the hole directly opposite the hole already drilled in the steel set, the device shown in Fig. 7 was used. The hole for the bolt head was counter-bored by hand with an extension bit, and the bolt hole was bored with an air auger machine with a twist drill.

CONCRETING.

During the si king, every 75 to 100 ft., two or three adjacent sets were filled in to the solid rock with concrete; this made it unnecessary to cut hitches and place steel bearers. This concrete also serves as a permanent support to the shaft. It was mixed on the surface and lowered in a hopper, see Fig. 8, at the bottom of which was a flexible spout, as shown in Fig. 9.

When the shaft was sunk to a depth of 1,207 ft., it was thought necessary to complete the concreting because of the approach of cold weather. Concreting was started at a depth of 1,170 ft. The concrete was mixed in the proportions of 1:3:5 in a ½-cu. yd. electric driven mixer, and conducted through a launder to a 4-in. flanged pipe laid from surface. The lower end of the 4-in. pipe telescoped into a 5-in. branch. This 5-in. branch took the blow of the concrete. To the bottom of the branch was connected a reverse bend with its lower end vertical. A flexible spout 18 ft. long which fitted over this conducted the concrete to the forms. While the concreting force was filling one set, other men were removing the blocking from the set above as explained, hanging the strands of old wire rope vertically 1 ft. apart and horizontally about 3 ft. apart for reinforcement, and placing the outside forms. For an 8-ft. span, 2-in. hardwood plank was used, as shown in Fig. 9, and for 4-ft. and 6-ft. spans 1⅝-in. hardwood plank. The plank was cut on a bevel on the upper end, so that the concrete came underneath the steel sets for a support. The bottom end came tight against the outside flange of the H-section. Two-inch strips of wood about 12 ins. long were laid 1 in. apart between the bottom end of the plank and the inside flange of steel. When these strips were taken out the planks were easily removed from the concrete.

In all cases the corners were left open for a distance of at least 12 ins. from the corners of the sets. This left a solid column of concrete in each corner for the entire depth of the shaft. Also where the lagging and timber was left between the concrete and rock, openings for concrete were left directly back of the wall plates and end pieces to the solid rock. Thus in all cases the concrete extended from the steel set to the rock. A 6x8-in. block 12 ins. long was laid in the concrete midway between the 8-ft. sets to serve as a support to the two end guides.

WATER.

Most of the water entered the shaft at 15 ft. from surface, and a concrete dam built about 100 ft. down collected most of it. When the dam was full, the water was run into a bucket through an opened valve and hoisted to surface. The water in the bottom of the shaft was also handled in buckets. On the trestle landing a wheeled water tank was pushed underneath the bucket.

LABOR.

The day was divided into three 8-hour shifts. Nine miners and a foreman per shift did the drilling, blasting and mucking, and assisted the timbermen in placing the sets, concrete bearers, and 12-in. pipe.

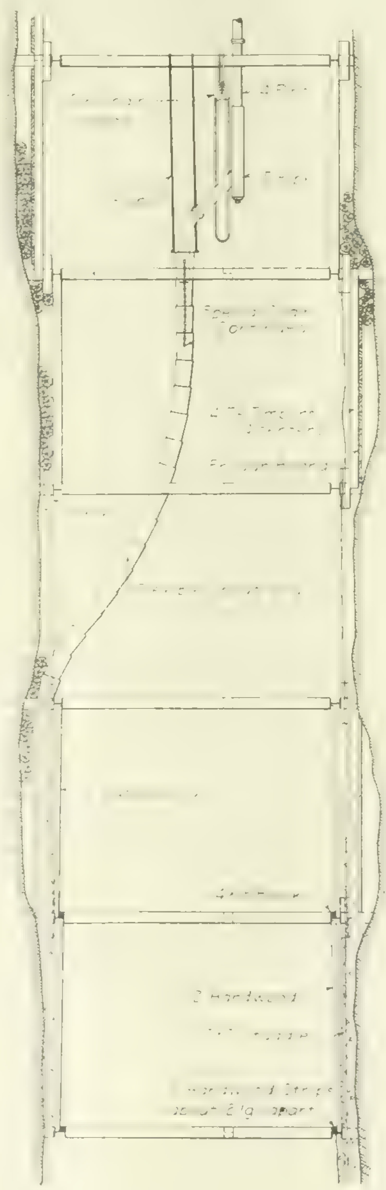


Fig. 9. Detail Showing Form of and Method of Using Flexible Spout for Distributing Concrete from Hoppers, Palms Mine Shaft Construction.

Three timbermen per shift for three shifts with two foremen for the 24 hours lagged the sets, put in the guides, extended the air line, placed the ladders, and substituted for absent miners, etc. During 24 hours two engineers operated the double-drum hoist, and two the single-drum. There were two top-landers per 12 hours and two men to handle the rock, tram, move track and level the stockpile ground. Two blacksmiths were engaged during 24 hours with a helper for one shift. After each cut was drilled all of the machines were taken apart for inspection and repairs and oiled. This required a mechanic for a few hours each day.

The concreting required the 10 miners for removing lagging, placing reinforcement and placing plank forms. The four timbermen attended to the distribution of the concrete to the forms. On surface three men wheeled rock to the mixer, two men the sand and cement, one poured water and attended to the securing of the proper mixture, one discharged the mixer, one looked after the launder from the mixer to the 4-in. pipe and two men conducted the concrete down the 4-in. pipe. All the men worked 8-hour shifts on the concreting.

The approximate time required for a 7-ft. cut was as follows:

	Hours.
Drilling	4
Hoisting tools and blasting	1
Blowing smoke	½
Lowering men and cleaning off sets	1
Trimming the sides	1
Mucking and picking bottom	14½
Placing a set	2
Lunch	½
Changing shifts	¼

For extending the shaft equipment the approximate time required was as follows:

	Hours.
To lower and place one length of air pipe	¾
To lower and place one length of 12-in. pipe	1
To lower and place one length of guide	¾
To lower and place one length of 7-in. channel	¾

Concreting during sinking required two shifts to make a plank bottom and fill between three adjacent sets or 16 ft. The speed of sinking the shaft, including the placing of steel sets and lagging, occasional concreting, etc., averaged from 4 to 4.56 ft. per day during several months. For the last three weeks in August, 1913, it averaged 5 ft. per day.

The speed of final concreting was from 35 to 48 ft. per day. For the total distance concreted 78 gondolas of sand and 15,695 sacks or 21 carloads of cement were required.

During the entire shaft sinking not a single serious accident resulted. Great credit is due the men for the versatility of their suggestions, their willing application to the work, and the interest they manifested in the speed and general progress of the shaft sinking.

Some Principles Governing the Organization of City Engineering Departments.

The duties devolving upon the city engineer's department vary rather widely as between various American cities. This variation is due in part to differences in population but it is also due to arbitrary organization of public works departments and sub-departments. Thus in some cities the city engineer is in charge of the municipal water works while in the majority of cities, it is believed, he has little or nothing to do with the water works plant. There are, however, certain principles which may well be permitted to govern the organization of the engineering department, and this article gives a British view of these principles as stated by Mr. Edward Willis, of Chiswick, England, in a recent paper before the Institute of Municipal and County Engineers. Much depends on a good organization of the forces available; much is to be gained by limiting or eliminating the overlapping of duties and responsibilities.

In approaching the subject the author is met with the initial difficulty of the large variation in the work and responsibilities that the municipal engineer has to undertake, and he therefore proposes to deal briefly with only those duties which appertain to practically all appointments, while he will touch but very lightly upon those branches of the department which in large towns and cities are often placed under a separate head, but which in the smaller towns are usually added to the multifarious responsibilities placed upon and undertaken by the modern municipal engineer.

Assuming that it is possible to create a new and complete department, the author would suggest the following main headings as the basis of its organization: Highways; sewerage and sewage disposal works; open spaces, parks and pleasure grounds; collection, removal and disposal of house refuse; stores and depots, and establishment and staff.

The real essence of good organization is the efficient and harmonious working of each section of all departments, with no friction or wasted time where two sections overlap or become interwoven. It is often in this overlapping between the separate sections of a department that most difficulties arise, loss of time is incurred, and individual character, ability, and tact are required to avoid friction.

In the creation of a department for a new small urban district it will probably be found that a very small staff will be allowed for a considerable amount of work, and this must result in several sections of the work being thrown upon one individual. If, however, the scheme is properly thought out and organized, its development will be automatic with the increased prosperity of the district, and no reorganization, with its consequent possible upheavals and heartburnings, should ever become necessary.

It may be desirable before dealing with the above sub-headings to say a few words about the personnel of a staff.

In considering the smallest type of urban district, it may be assumed the staff of the municipal engineer's department consists of one man, described in Great Britain, in that most comprehensive word, "The Surveyor." If his council are particularly generous he may have the advantage of a pupil and office boy. In this case but little organization of staff will become necessary, but the organization of work is a *sine qua non* if the administration of the district is to be efficient, and no harm can result from the scheme of such work being considered on broad, but definite, lines; in fact, the man that can grasp and deal with his work on broad lines is individually the most useful and best organizer, provided he can likewise appreciate the importance of detail.

In the case, however, of an average municipal engineer's office in a town of, say, from 10,000 to 50,000 inhabitants, the circumstances are undoubtedly very different from the foregoing. The borough or district is probably already in a flourishing condition. It may be a quiet town, or it may be rapidly developing, the latter being probably the case in a town or suburb immediately adjoining a great city, or in one which is being developed owing to the creation of a thriving industry within its boundaries.

Arrangements for expansion or possible decentralization may therefore be anticipated and provided for; the individual abilities of each member of the staff should be appreciated to a nicety; the work in each section of the department should be defined and systematic; supervision should be accorded to all; absolute punctuality should be enforced, and integrity of thought as well as of deed should be fostered.

It might be here mentioned that some officials, while being absolutely above reproach in financial matters, still have not grasped the thought that absolute integrity includes principles as well as actions, and the head of a department who can feel that not only integrity, but absolute loyalty to himself permeates every member of his department is indeed fortunate, and such a knowledge is very helpful when the anxieties and worries of dealing with the numerous members of a large corporation weigh unduly heavily upon him.

It cannot be too much emphasized that a true *espirit de corps* should exist in every official, from the highest to the lowest, to ensure that perfect harmony in work which gives the most beneficial results to the council and taxpayers at large, and therefore any scheme which an engineer can advance to further this spirit will not only redound to his credit, but will probably add to his popularity, and make every individual member of the staff willingly throw his whole energies into his daily work, and not complain when the exigencies of the public service necessitate hours of overtime, often without any thanks from his council.

Highways.—As the construction and management of highways created the necessity for the surveyor and engineer, so it still ranks as one of the most important duties attaching to his department, and in the majority of districts it is organized as a separate section or sub-department.

The number of men engaged in maintenance and supervision must of necessity be governed by the area and the traffic throughout the district, but in practice the author advises that existing highways should be dealt with separately from the construction of new highways, since the latter largely consist of private street works and street improvements, and as such are subject to more detailed drawing and office work than the maintenance of existing roads.

The author is of the opinion that even in a small district it is wise to make one man responsible for the administration of this sub-department, so that the entire maintenance of the existing roads and walks of a district are in his hands. He would then have charge of sprinkling, street cleaning and repairs, including all team labor, manual labor and supplies used upon public highways.

The question of improvements to existing highways and lighting in a small district is also left in this official's hands; but where the administration of a large town or city is to be dealt with, he is of the opinion that the supervision of both improvements and lighting can be relegated to two other officials, who can personally report to him and specialize in their particular work.

Some engineers require personal reporting daily from the head of each sub-department, and this system has many advantages. At the same time the author has found in practice that there are equally great disadvantages, since in his own experience he has wasted hours in obtaining access to his chief to report sometimes comparatively trivial matters, and the procedure he has adopted in his own office is to obtain all reports in writing from the foregoing officials, who attend for interviews only when required.

Before concluding these observations on existing highways the author would like to emphasize the desirability of not only recording the materials and labor expended on repairs through the outgoing supplies notes, but also the advisability of keeping abstracts, charts, and maps of the district showing exactly, from time to time, with the date attached, the amount of recoating of roadways or repaving walks carried out each year, and, as far as possible, the prime cost should be kept of all such work and reduced to a definite standard.

While advocating this the author must admit he has been unable, with a limited staff, to keep entirely to this standard of perfection; but he still advises the continuance of such a system as being ultimately conducive to both economy and efficiency.

With reference to sprinkling he has made a practice of attaching to each water cart a small skeleton map of the route, with positions of all road-watering posts shown thereon, as this tends to prevent the continual waste of time which will constantly occur where the number of road-watering posts is limited, and enables a better check to be maintained over the employes.

The question of street cleaning with gangs or one-man routes is a vexed question, and in most districts too little money is put on this important work. The quality of the labor also is often far from satisfactory, any man being considered fit to handle a broom. This is not the author's experience, as he has found that one man can often do twice the amount of work of another, and that such work is often more efficiently performed.

The author considers new highways, especially private street works, can often in a large district be left in the hands of one capable assistant, with one or two draughtsmen to assist him, since uniformity in practice with private street work and apportionments is conducive to lessening of friction with speculating builders and small property owners.

It is desirable that provisional and final apportionment books be kept in lieu of loose sheets, and that such books should give full particulars of the ownership, frontage, estimate, final account and final apportionment, so that the past history of a private street can easily be traced without the necessity of wading through numerous loose papers.

In some districts it is customary to have an absolutely standard specification and drawing for private street works; but this has not been the author's practice, since what may be desirable for one highway may be totally unsuitable for another, and in recent years under this heading the author has used from 9 ins. to 21 ins. of total formation and the following finished surfaces: 9 ins. to 6 ins. of flints; flints tar-painted and finished with granite clippings; tarred granite; tarred clinker finished with tarred limestone; Guernsey granite with Tarvia and granite chippings.

One other point before leaving this subject may be usefully mentioned. Where trees in streets are generally adopted, they may often be included in the private street works specification and estimated with advantage as well as grass margins, if the cost of maintenance can reasonably be afforded.

Careful records of renaming and renumbering must also be kept, as certificates are continually being required when property changes hands.

Sewerage and Sewage Disposal Works.—In this sub-department the author suggests the separation of new and existing sewers from sewage disposal works; in fact, in all large towns and cities such a separation is a *sine qua non,* in many cases the sewage disposal being undertaken by a joint board, and dealt with either by a sea outfall or bacterial treatment works.

The variations in the management of sewage disposal works are, of course, entirely governed by the size and character of the works and the quantity of sewage treated; but it cannot be emphasized too strongly that where the outfall discharges into a non-tidal stream absolute adherence to instructions is essential, especially in the case of contact beds, where three shifts in 24 hours is adopted. In this case there is no excuse for lack of attention at night-time, but when men are working 12 or 13 hours through the night, under little or no supervision, the author has found neglect sometimes ensues.

The author has made a practice of supervising or inspecting personally at all hours of the day and night, without notice, and he has found this has eliminated to a large extent the risk of serious neglect by his men.

Under the heading of "Sewerage" he suggests separating the management of new and existing sewers so far as the individual staff is concerned. This, of course, would again have reference to a large district, or one rapidly developing, and in practice he has found that the outdoor building department can well supervise the laying of new sewers without undue overlapping, since the majority of these are laid in new streets by private estate owners.

Maintenance, flushing and connections to sewers should be as far as possible under the control of one man, who should be solely responsible for seeing to the actual work of the sub-department, but in practice it is often found desirable to arrange for house connections to be supervised by the building department, which, as an inspector should be upon the works daily, saves time and eliminates overlapping of inspection.

It is needless to detail the many sub-headings that would apply in this section, but the author would emphasize the importance of any spare time being spent on keeping the surveys and levels absolutely up-to-date, and shown on the standard ordnance maps of the district. The surface and invert levels should be entered on such maps at every manhole or change of direction or gradient.

Open Spaces.—In dealing with this important section of a municipal engineer's work, it may again be reasonably subdivided under the following headings: (1) Maintenance of Existing Open Spaces; and, (2) Provision of New Open Spaces.

Briefly referring to the latter heading, the first consideration is defining the most suitable position and negotiating for it. This, of course, is exclusively a matter for the head of the department, probably in conjunction with the clerk of the council.

The design and lay-out construction of walks, banks, flower beds, shrubberies, gymnasia, drainage, water supply, chalets, conveniences, etc., would be best prepared by the indoor staff and the supervision undertaken by the outdoor staff.

A matter which is now continually before municipal engineers is the desirability, or otherwise of executing certain new works by means of direct labor. The author has had experience of both the contact and day labor systems to some large extent, and he is of opinion that there are many works in which direct labor can be economically adopted; but if they are to be successful and economy is to result, the engineer should have an absolutely free hand in regard to the employment of labor, and should be exonerated from the risk of criticism before the work starts, should he find it necessary to modify the labor conditions.

The question of direct labor is one which, at some time or other, is bound to be considered by every municipal engineer, and no one should neglect to take advantage of the experience of his confreres in this matter. Personally, the author has consistently adopted direct labor on new sewerage works and open spaces with both satisfactory and economical results, while he has found it also beneficial in the construction of open-air baths and various road improvements; but where buildings are to be erected, or where there is any question of payment by other persons, such as in private street works, he prefers the contract system.

The systematic upkeep of iron and wood work is often neglected through carelessness, bad organization, or false economy, and therefore one cannot be too careful in clearly defining the word "maintenance."

Collection, Removal and Disposal of House Refuse.—While this is one of the most important duties usually devolving upon the municipal engineer, it is equally one of the most unpleasant, and is probably the cause of more complaints than any other sub-department.

He must consider first the method of collection and disposal. Collection may be by contract or direct labor, and in each case by either horse haulage or motor traction.

Briefly, motor traction is beneficial where the shoot, deposit, or destructor is at a distance, or it can be economically adopted where the refuse cans are placed on the sidewalks at night and are emptied daily. Direct labor can usually be more efficiently supervised and leads to fewer complaints than contract labor in this class of work.

Having decided on the system, definite routes should be assigned to certain wagons and collectors, and the collection from each route should be completed each day. If this work is carried out properly no complaints should arise, but the author has found some difficulty in entirely avoiding these, owing partly to the substitution of other men during sickness or accident and the interference with normal organization during holiday seasons.

Some difficulties are also experienced in regard to empty houses or access to occupied houses when no one is at home, but the author's practice on such occasions is to deliver a card stating when the next collection will be made, and the men report on tabulated sheets the reason for non-collection. By this means it is possible to check whether non-collection is through the carelessness of the men or the temporary absence of the occupier.

In the disposal of house refuse, a proper record should be kept of the quantity received, preferably by means of a weighbridge at the destructor. This record should be checked with corresponding periods in previous years, and also with the average daily quantity, which should show a gradual increase with the growth of the district.

Special care is also required in testing the tare weight of collecting vehicles, since although it is wise to record the tare on every vehicle, refuse wagons are especially liable to be incorrect, owing to the inclusion of glasses, cans, shovels, baskets, etc., and unless special care is exercised in the clearing out of the vehicle, a considerable excess weight over the actual amount collected may be shown. This, in the case of piecework, may not always be accidental, and to avoid it the only proper method is to tare these vehicles out after every load.

In connection with the destructor, proper records should be kept, preferably in tabulated form, of the amount of steam raised, water and coal (if any) consumed, power used for pumping or other purposes, quantity of clinker made, amount of clinker broken in crusher and used for special purposes, such as mortar, brick and flag making, tarred clinker for roads, etc., and a careful analysis of the cost of other materials and labor throughout, so that comparison may be made with corresponding figures at different periods and those of other local authorities, which, with proper allowances for varying conditions, form a good guide as to the economy and efficiency with which this section of the department is being administered.

In the case of extensive repairs to chimney shafts, main flues, etc., provision will have to be made for the removal of the refuse, by road, rail or water, to another destructor or shoot, and sufficient foresight must be exercised to avoid any interference with the normal daily collection and disposal of refuse.

Stores and Depots.—A complete or even partial scheme of reorganization in connection with central or branch depots and storeyards requires most careful consideration, since after the cost of buildings, stabling, and other structural works for the reception of horses, wagons, motors, rollers, etc., has once been incurred, the capital charge in the event of subsequent alteration will be entirely or very largely wasted. The greatest care and ability must therefore be exercised by the engineer before making any recommendation, and he must not only consider the present, but also the ultimate possibilities and needs of the area of which he is the controlling officer.

It is difficult to lay down any hard or fast line as to centralization, but it may be taken as an accepted fact that all horse vehicles should be housed within a reasonable distance of the area upon which they are to be used. At the same time the multiplicity of storeyards or depots permanently increases the establishment charges, for which, as a general rule, a varying percentage is required to be added to the cost of works.

Establishment and Staff.—The author cannot conclude without making a few observations under this heading. In the earlier part of the paper it was pointed out that perfect loyalty and absolute integrity of thought and deed would tend to result in the development of an excellent staff.

In addition, every assistant should continue his studies by means of professional literature, lectures, etc., and always strive to learn and know more than is necessary for the work required of him, since by that means alone he is able to fit himself for a more responsible position.

Omitting many other points, one small one must be emphasized—viz., the initialing and dating of all plans, drawings, or documents by the assistants or clerks engaged thereon, and the checking and initialing of all calculations, quantities, and certificates, since it is so easy in the midst of the multifarious duties which devolve upon this department for a slip to be made, but so difficult to restore that confidence which all desire and endeavor to merit.

ROADS AND STREETS

Economic Factors Involved in Road Construction in Strictly Rural Sections.

The economic factors involved in the planning and construction of a system of improved roads are intricate. A consideration of these factors must include so many variable conditions and be based, frequently, on such incomplete data that an accurate solution of the problem of economic apportionment of funds for construction and details desirable to use is difficult. In a paper presented at a recent meeting of the Canadian Society of Civil Engineers, Gabriel Henry discussed the problem as applied to the rural roads of Quebec, Canada, and his paper is given here in part.

PRIMARY CONSIDERATIONS IN THE APPORTIONMENT OF FUNDS.

It is fundamental that any outlay of money must be made in such a manner that it will result in a paying investment to the community. In order to obtain this not only the first cost of the improvement must be considered but the funds so invested must be protected from loss by waste and decay. The conclusions to be drawn from this are: (1) That no road should be improved without some provision for the maintenance of such road. (2) That all unprofitable work and works "de luxe" should be avoided within certain limits. (3) That all improvements not actually and positively needed should be omitted or postponed. As examples of such improvements the author may cite the adoption of costly surfacing for portions of roads where the traffic provided for will come at a later period, and all other unwise selections of surfacing either with a view to economize on the first cost or to display some luxury. (4). That the choice of some kind of pavement, the durability and ease of maintenance of which has not been ascertained in practice, be avoided as much as possible. (5) That unnecessary experiments be avoided. It is better to profit by experience obtained elsewhere and to make only those experiments which are absolutely necessary for adapting some good recognized product, process or method to local conditions.

Again, as the improvement of roads, like industrial or commercial investments, has to be a paying one, the sum invested should, in the author's opinion, be divided into three parts; one to be set aside for work which does not decay and can be easily and cheaply maintained without loss of value; one for work which decays and cannot be maintained without loss of value and for which a depreciation fund is to be provided; and a maintenance fund for annual repairs.

In the case of a macadamized road, for example, the cost of the earthwork, of the drainage and underdrainage and of the foundation should be considered as belonging to the first class; the cost of the surfacing to the second class, and to the third class the money required annually for maintenance.

Again, there should be no hesitation in spending money judiciously and when needed for earthworks, under-drainage, ditches, substantial, well designed and well constructed concrete bridges and culverts, and good and appropriate foundations.

All money so expended adds value to the country, prepares roads for the surfacing, even when the latter is to be laid later, and is remunerative. Such kind of work well planned and well made lasts a very long time and requires little money annually for maintenance. On the other hand it is a waste of money to use costly wearing coats on a substructure not well drained or a water-bound macadam on hills with a 10 per cent slope.

Further, the author's impression is that the improvement of the roads should be begun by preparing the substructure, lowering grades or rounding up hills, drainage surface waters and under-draining ground waters, constructing bridges, straightening unnecessarily tortuous roads, maintaining these improved substructures as earth or gravel roads as the case may be pending the next step of the improvement, and the laying of surfacing. All this can be done economically. Surfacing should be used only when and where necessary. This mode of procedure would have a great advantage which is worth being insisted upon. Laying wearing coats on new fills not sufficiently settled, or on a new soil not sufficiently trampled by traffic nor well seasoned, accounts for many failures of good surfacings and for expenses of maintenance greatly exceeding the provisions. Quick work in road making is not always to be recommended.

When the substructure of a road has been prepared and the foundation laid in advance, inconvenience is to be feared. Another occasional advantage is to divide the work. Contractors specializing in earthworks sometimes are not well acquainted with the laying of wearing coats, or have not at their disposal the skilled workmen required for such work, and the reverse is the case for surfacing contractors.

WIDTH AND CROSS SECTION.

The author does not advocate the spending at present of all available money in preparing substructures of rural roads. Provincial roads are badly needed and for these the work must be pushed quickly; but stress must be laid on the advisability in many cases of preparing the substructure of roads in advance, and on the inconvenience of laying surfacing on improper and unseasoned substructures. There is no standard cross section for substructures of roads, but details should be standardized in accordance with the results furnished by experience. The width of 24 ft. between ditches is not too great, but just what is needed for two wide automobiles or two hay wagons to pass each other, or a hay wagon to pass a wide automobile; for such cases plenty of room should be provided. It would be better to widen than to narrow the roads. When a horse vehicle meets an automobile, if the horses are nervous and deep ditches run alongside the road, a traveler often feels that 24 ft. is not too wide. This width should be reduced only in roads of secondary importance or when called for by special conditions. Even for roads much less important than provincial roads this width should be adopted if an increase of traffic is foreseen in the near future. In trunk roads the width between ditches may be increased if necessary to 32 ft. or more and should never be less than 24 ft.

On embankments the platform should re- [illegible] a total width of at least 27 ft. from edge to [illegible] should be set at least 1 ft. from the edge of the embankment.

In cuts, generally, no ditches are needed and gutters prevail; consequently, the total width of the platform at the bottom of the cut can usually be reduced to between 26 and 28 ft. for a 24-ft. wide road.

The edges of the embankments should be beveled or rounded when the engineer has to deal with a clay soil, in which case it is useless to try to get sharp edges.

The slopes along the embankments and cut should generally be 1½ horizontal to 1 vertical, but engineers may avail themselves of the nature of the soil if well acquainted with the natural slope of one particular soil.

In this country it is necessary to avoid cuts on account of the snow, which, in winter, fills them and renders the road impracticable.

If the road is to receive its surfacing later, it should [illegible] least 1 in. in the foot in order to shed the rain-water into the ditches and to allow maintenance with split-log drags. If the road is to immediately receive its surfacing, shoulders at least 4 ft. wide should be provided. The slope of these shoulders towards the ditches should be at least 1½ ins. to the foot.

Such are the principal features of a cross section of the substructure.

EARTHWORK.

For the execution of the earthwork the same rules are to be followed as in the case of railways. For shrinkage the rates of fills to cuts comprise between 1.15 and 1.30—1.15 being for heavy work and 1.30 for light skimming work.

In the author's opinion the rolling grade system with vertical curves should be adopted, because it is more economical and facilitates the drainage of the substructure. It is useless to try to get at great cost long stretches of horizontal road. The maximum ruling grade should not be more than 5 per cent whenever possible, 6 and 7 per cent grades being resorted to only in short patches when absolutely imposed by necessity. A waterbound macadam is maintained only at great cost in this country on grades of 7 per cent and more, and on account of the snow filling the ditches in the spring water from melting snow, in many places, injures the roads. The same holds for earth or gravel roads. It is better to try to get 5 per cent by turning around hills when improving a road than by cutting them too much. Minimum grades should not be less than 0.6 per cent to ensure the longitudinal drainage of the road.

Curves having a radius less than 300 ft. should be avoided in steep grades and at the foot of such grades, and a radius less than 200 ft. should always be avoided even in flat stretch. When it is necessary to adopt a radius less than 200 ft., the road should be widened in the curve and be given a transversal slope towards the center of about 1 in. to the foot.

The end of the beginning of a slope should not be found at a distance less than 50 ft. from a level crossing of a railway or from the end of a bridge. Sight distance in horizontal or vertical curves should be at least 300 ft., depending in each case on the conditions. Near a railroad crossing the sight distance on each side in the direction of the track should be at least 3,000 ft.

In every case, the economy of grading should not be the controlling factor. The most important considerations should always be kept in mind in determining the profile and the cross sections. The standards can be used as general guides, but good common sense and acquaintance with local conditions should have much to do with it.

The earthworks are computed as for railways, but for roads greater exactitude is needed because in this case they consist chiefly of skimming work, ditching and shouldering, which cost more per cubic yard and are more difficult to estimate exactly.

The draining of the soil is the most important thing in road construction, because the bearing power of the soil depends upon the moisture it contains. In this country this question is far more important than in the southern countries, owing to the nature of the climate.

The soil is soaked by rain-water and by ground-water. The rain-water should be disposed of as quickly as possible by a good, substantial system of ditches, gutters and culverts. Deep ditches should be avoided as much as possible and gutters substituted. Deep ditches are only necessary along flat stretches of road in order to get the necessary slope to cause a quick and complete drainage of water. The minimum slope to be given to ditches should be 5 ins. in 100 ft. Stagnant water should never be found in a ditch.

The greater slope depends upon the resisting power of the soil to abrasion by water and should never exceed such power.

The slopes of the sides of the ditches should be regular and as nearly as possible 1½ ft. horizontal to 1 ft. vertical. It is very difficult to attain this in a flat country where ditches have sometimes to be very deep and wide and would exceed the portion of the right of way that it is possible to set apart for them. In such cases rip-raps have to be resorted to. Gutters along hills should be paved with cobblestones whenever the soil is not hard enough to resist abrasion by running water.

Tile underdrainage with good outlets is generally to be preferred to deep ditches for disposing of the ground water. Road underdrainage should be made in the same way as in the case of field drainage. Tiles should be laid under the sides of the road rather than under the center, and should be disposed so as to intercept all water coming from the neighboring fields before they enter the right of way. It is important that good and frequent outlets should be provided. Underdrainage, in order to be a permanent and useful work, should be done with the greatest care. It is very difficult to drain a road effectively and completely, and in many cases it is impossible to get the ground water out. Generally, although the soil is dry, its bearing power is not great enough to stand concentrated wheel loads and it is impracticable to sufficiently consolidate it; sometimes a sufficient and effective underdrainage is impossible, and then it is necessary to resort to the use of a foundation in order to distribute the concentrated wheel loads over a wider surface.

The foundation plays the same role in the case of roads as ballast in the case of railways. It is generally assumed that the downward pressure follows a line at an angle of 45° from the horizontal and is distributed over an area equal to the square of twice the depth of the total thickness of the surfacing and of the foundation. This is merely an assumption, but it is approximate enough in practice, and it has been found that the thickness of the foundation plus the thickness of the surfacing should not be less than 9 ins. in common soil for steel tire traffic.

A non-porous soil drained of ground-water will support a load of about 4 lbs. to the square inch; that is, about 1,300 lbs. per square inch applied to the surface of a coating 9 ins. thick per lineal inch of wagon tire. In practice a load of 700 to 800 lbs. per lineal inch of tire is assumed; that is, 2 to 2½ ins. per square inch on the substructure with a 9-in. thick surfacing including the thickness of the foundation.

Many good surfacings give bad results owing to the lack of a substantial foundation. For a soft tire traffic a foundation is not so necessary as for the traffic of wagons with narrow tires and without springs, the impact of which is very damaging to roads.

The foundation should be composed of stones not larger than 4 ins. and a filler in the voids is to be recommended whenever it is not very expensive. Gravel, as in the case of the railways, gives a good foundation and is even better than stone. Telford foundation should be resorted to only in case of necessity. Flat stones should not be used.

WATERWAYS.

Foundations are of the class of permanent works mentioned above. Bridges and culverts should also be so constructed as to be considered worthy of the same class.

Good concrete is well adapted for this kind of work. Concrete pipes should not be used in naturally wet clay soils, especially under heavy embankments. They should be of ordinary thickness and of a diameter not larger than 30 ins. under embankments higher than 14 ft. In these two cases box culverts would be preferable. Sand cushions should be employed to protect them against rock. It is absolutely necessary to give the head walls a firm seat, especially in clay soils and when the pipes are under an embankment. Concrete slab bridges are very useful as well as concrete girder bridges. The maximum economical span of a slab bridge is about 15 ft. and of a girder bridge between 15 and 30 ft. or more. Under high embankments arch culverts should be used.

Concrete trestle bridges are especially recommended for use on rivers where numerous deep gullies are found, as this type of bridge spans them very economically and artistically, thus dispensing with heavy embankments and deep cuts. The latter ought always to be avoided in this country on account of snow.

Many difficulties are encountered in fixing the dimensions of the culverts because the road engineer has, when deciding, to deal with the farmers. The same difficulties are met with when straightening tortuous, crooked roads and improving curves. These difficulties are not technical, but are of common occurrence in all countries, being unpleasant and unavoidable. The greatest care should be taken in the planning and construction of bridges and culverts for which the same general rules have to be followed as in the case of railways and of first class concrete work.

ROAD SURFACES.

The cost of the permanent work, that is, of the part of the outlay permanently invested in the case of roads, is a smaller fraction of the total cost than is generally supposed. Except in special cases, it never exceeds one-quarter or one-third of the cost of the surfacing proper, and in the case of special wearing coats this proportion is still smaller; consequently, great care should be taken in the choice and the laying of this part of the road which is short lived and so expensive.

The following points should be taken into consideration when making a choice:

(1) Not to attempt to risk surfacings on a substructure which is not sufficiently firm with or without foundation. Waterbound macadam is not excepted. (2) Not to use surfacing in places where floods or other causes could damage same. (3) Not to use costly surfacing when a less costly could reasonably do as well for the number of years the less costly is supposed to last. (4) Not to use costly surfacing when the amount of the total sum of the first cost of the sinking fund and of the maintenance of the said surfacing is higher than with a less costly wearing coat which would do as well.

There is a great variety of surfacings. Leaving aside the earth, sand, clay and ordinary gravel roads which are, as previously stated, very convenient in the majority of the rural roads where traffic is only local and light, the principal surfacings for highways and rural roads are: Waterbound macadam and waterbound gravel macadam; asphaltic macadam, penetration method; asphaltic concrete, mixing method; tar macadam, penetration method; tar concrete; cement concrete surfacing, penetration method (Hassam process); cement concrete surfacing, mixing method; and vitrified brick.

This is a rough classification. Many attempts have been made elsewhere to properly classify and name the road surfacings, but a complete understanding has not yet been arrived at. Of course, the composition of aggregate used with bituminous or tar cement, for instance, in the mixing and penetration methods, varies greatly and sometimes there is little difference between the aggregate used in the two methods. The names of the surfacing are very numerous, depending on some process in the execution of the work and on the nature of the cement and aggregate used.

Apart from the above surfacing some chemical products, such as rocmac, are found on the market, the function of these being to bind or help to bind stone in the macadam.

Tar, bitumen oils and other chemical or residual products such as calcium chloride and glutin should also be mentioned. Their functions are to protect the road surfacings against the wheels of vehicles, to give them a longer life and to lay dust. These preventives have to be renewed more or less frequently.

The author is merely speaking here of the surfacing and preventives generally used in rural road and highway work and not of the still more expensive paving used in cities.

Now what is to be said in regard to all the above named surfacings? There are such great and important interests concerned in bitumen, tar and cement surfacings, success and failure are in fact so mixed up and the experiments, as reported, have given such widely varying results, that it would be very difficult to give a categorical opinion.

ECONOMIC CONSIDERATIONS INVOLVED IN THE SELECTION OF SURFACES

With the economical aspect of the question in view the following conclusions can be drawn:

There is not at present any surfacing good for general purposes. Automobile traffic requires certain qualities for surfacing, and the narrow steel tires, heavily loaded wagons and motor trucks need others.

All kinds of surfacing can be used with advantage if judiciously and suitably chosen, well laid and maintained.

All surfacings require to be maintained with the greatest care.

A combination of steel tire and of soft tire traffic in different proportions requires in each case specially appropriate surfacing.

Bituminous macadam and bituminous concrete macadam, as well as tar, are very suitable for automobile traffic.

Experiments on concrete roads point towards a more general use of concrete surfacing, which does not seem to be affected by a cold climate.

Waterbound macadam, well built, is good for rural roads where automobile traffic is not heavy. Tarring and oiling lengthen its life and allow a medium heavy automobile traffic on it.

All that has been said above presumes that the substructure of the road has been sufficiently drained, that a substantial foundation has been provided for, and that the sum of the thickness of the foundation and of the surfacing are in keeping with the nature and the dryness of the substructure. The road is also supposed to be maintained with care.

In choosing a surfacing the following should be taken into account: (1) The nature and importance of the traffic both present and future; (2) the first cost of the surfacing; (3) the probable life of the surfacing; (4) the cost of its maintenance.

Such information is now easily obtained, and by adding in each case the interest on the first cost, the annual annuity for depreciation fund and the probable annual cost of maintenance of two or more surfacings, the total will indicate the one to be chosen. The first cost should be established as first class.

It may be added here that bituminous and tar surfacing require very great care and experience to be laid properly and that it is more difficult to lay them in this country than in a dry climate.

The annual period of work in the Province of Quebec is about 100 working days; rainy days are frequent as well as variations of temperature, which are at the same time very great.

For a good bituminous and tar surfacing the stone must be dry and not very cold when cement is added. The degree at which the bitumen or tar is to be heated depends to a certain extent on the temperature of the stone and of the atmosphere, and in the autumn the weather is often wet and cold.

WATERBOUND MACADAM.

With waterbound macadam, as well as other pavings and especially concrete paving, the quality of the stone plays an important role. Soft stone should be avoided as much as possible.

In the Province of Quebec trap and good rock for road purposes are scarce. Field stones are generally employed and the quality varies greatly as they are generally mixed with bad stones. Sandstone of inferior quality is very common; in some districts a kind of bad granite is found, while gravel and quarries with good material for road purposes are rare.

For waterbound macadam, the author advocates the two-course macadam—bottom course, 4-ins. loose stones of 2½ ins. ring, well rolled with a steam roller of at least 10 tons, no filler; top course, 4 ins. loose stones of 2½ ins. ring, an appropriate binder entered dry in the course with broom and roller in as great a quantity as possible. The first layer of the binder should be applied before the rolling is begun, and when it is not possible to enter more binder without the roller spraying is resorted to.

The road should be sprinkled until saturated, the sprinkler being followed by the roller. More screenings must be added if needed, and the sweeping, sprinkling and rolling continued until a grout has been formed of the screenings, stone dust and water to fill all voids and form a wave before the wheels of the roller. After that, enough screenings should be spread over the macadam to leave a wearing surface at least three-eighths of an inch thick.

With limestone a course-grained sand, as well as the dust of the crusher, is good, but with granite a limestone binder is generally required. In some districts of the Province only sandstone is available. A limestone binder would be suitable with these stones, but, unfortunately, in these districts no limestone is found, and in such cases the dust from the crusher or the sand is of no value and it is impossible to bind the macadam. Thus the only way to obtain a first bond is to use clay and the traffic does the rest, but for two or three years it is necessary to keep the ruts filled and the surface even. Experience shows that it is possible to obtain good results in this way.

The crown of the macadam is generally ¾ in. for a 16-ft. wide surfacing; for a bituminous or tar surfacing ½ in. is enough; for concrete ⅓ in. is suitable.

The choice of a macadam as a surfacing depends not only upon the nature and the importance of the traffic, but also on the quality and the price of the stone. When the stone is of bad quality and dear, it is often more economical to use a more expensive surfacing. In the opinion of the author, the best macadam would be of one-sized stone 2½ ins. ring, with an ordinary soil, a total thickness of about 9 ins. comprising the foundation; also macadam stone should not be mixed with an undue quantity of smaller stones or sand. For a wearing course such a mixture containing too great a proportion of small-sized stones and of screenings is not suitable. As a rule it is not uniform, the larger and smaller sizes have a tendency to separate and, if not mixed with care and laid with shovels, the resulting macadam will have a very irregular wearing quality, and ruts and depressions will consequently follow.

It is very difficult to obtain the one-sized stone at a suitable price in the Province of Quebec because few regular quarries sell macadam stone. It is necessary to use transportable crushers, but these give many flat chips which must be utilized. Meanwhile, everything should be done in order to obtain the standard cubical uniformly sized stone.

The preventives mentioned above are serviceable in many cases, but their cost compared with results is rather high and generally have to be applied at least every year. Tarring, however, has given very good results.

The question of the width of the surfacings is not yet completely settled. Meanwhile experience shows that a width of 16 ft. is suitable for a provincial road and for less important roads 14 ft. is convenient. Shoulders should have a width of at least 4 ft.

FINANCING ROAD IMPROVEMENTS.

The advent of automobiles, although an important factor in the question of roads, does not particularly affect the rural roads because automobiles will not for some years increase in such number in rural districts as to cause apprehension to the farmers on account of additional expense in maintenance. It only affects such roads in the sense that automobilists ask for better rural roads; while the difficulty lies in the fact that they desire many highways crossing the country which the farmers contend are not necessary to them and which they will not maintain. Who then is to pay?

The improvement of rural roads will benefit the whole country, as well as the farmers, the municipalities and the automobilists.

The provincial highways will also benefit the country, the farmers and the automobilists.

It thus seems justifiable to divide the cost of improvement and maintenance of the roads of the Province between the three interested parties.

What is to be the proportion for each of them? This is a very intricate question, but one which can be solved.

For speedy work, money is not at hand here as elsewhere. The system of long term bonds in the United States came first on the assumption that the next generation would benefit by the improvement as well as the present one. But complaints soon arose. One thing had been overlooked, and that was the difference between permanent work and passing or short lived work.

For permanent work long term bonds can

be used. Short lived work constitutes the larger proportion of surfacing. A good bituminous surfacing is said to last from 10 to 15 years with a moderately high traffic in spite of the maintenance. Consequently, for such surfacing only 10-year bonds would be advisable.

For this kind of surfacing, which on account of its high first cost is chiefly used on highways with an important traffic, long-term bonds should be avoided because, in spite of maintenance, long stretches have to be renewed at the same time. A macadam well maintained and without automobile traffic will last a long time, as has been proved by many examples in the old countries, provided the maintenance is complete and effective, the material for macadam being generally at hand and good road men being found easily along the road and put in charge of the work. In such cases longer bonds are to a certain extent reasonable.

Industrial Railway Used in Concrete Road Construction in Wayne County, Michigan.

When road construction is in full swing teams are usually scarce. Traction outfits serve when bridges along the route are strong, motor cars can be used where roads are good, and industrial railways are serviceable where much long hauling is necessary. An industrial railway is useable in almost any country and during all kinds of weather.

The Wayne County road commissioners have been using teams and traction engines in past work, but this year an industrial railway has been installed for use on the longer hauls. The equipment manufactured by the Orenstein-Archer Koppel Co. consists of locomotives, flat cars for hauling forms and track units, steel dump cars for hauling cement and aggregate and standard 15-ft. rail sections with steel ties. The track is laid either on the sub-grade or on the side close to the road. Material is loaded directly from gondolas into the steel dump cars, hauled directly to the site and dumped on the subgrade. The trains consist of 25 to 30 cars and at present are making a haul of three miles. With this system enough material is readily and economically provided at the work to keep men and mixer busy, whereas, on some of the other construction projects in Wayne County work is occasionally stopped because of lack of material.

Edward N. Hines, chairman of the Wayne County road commission, states that although it has not been possible to operate the railway at its maximum capacity because of lack of railroad switches, yet the saving on the Eureka Road alone, the project on which the equipment is now being used, will pay for half of its cost.

The 1914 Wayne County concrete pavement is 12 ft. wide, 6 ins. thick at the outside and 8 ins. thick in the center. Instead of wooden forms along the side, 6-in. channels, 12 ft. long, are being used. Expansion joints placed every 25 ft. are protected by Baker plates and filled with tar felt. Concrete is mixed in a ½-yd. Austin mixer, a batch consisting of 3 sacks of Universal Portland cement, 4½ ft. of sand and 9 ft. of gravel. The specification that the batch shall receive 16 complete turns and remain in the mixer for one minute is resulting in producing an excellent quality of concrete. The crew of 30 men is able to lay an average of approximately 500 ft., or 666 sq. yds., in a 10-hour day.

Six wheelers and shovelers alternate in their work for each successive block of concrete, a scheme that has proved efficient because the variety of work tends to prevent its becoming monotonous. When the stock piles are a relatively great distance from the mixer, each of the 12 material men loads and wheels his own barrow.

The commissioners have installed a system of tanks at the point where materials are received and are piping the water from the Huron River, a distance of 10 miles, to the site for use in mixing and curing. The tanks have sufficient capacity to insure a constant supply of water at all times for all purposes—a highly important item in concrete road work.

Fig. 2. Mixing and Placing Concrete. Note Steel Side Forms and Method of Finishing Surface.

Fig. 1. A Train of 30 Cars Hauling Aggregate for Concrete.

Some Practical Notes on the Design and Construction of Bituminous Surfaced Roads in England.

English practice with regard to the construction of tarred roads is considered to be somewhat in advance of that of other countries, owing possibly to the great amount of work accomplished of this character and the excellence of materials available. A paper read at the last annual meeting of the Institution of Municipal and County Engineers of England, by W. H. Maxwell, gives a number of practical suggestions for road work, and the paper is given here in part.

In the planning of new through routes, directness of line is usually an important feature to be considered, from an engineering and utilitarian point of view, but leads to monotony in the use of the road, and from an æsthetic standpoint compares unfavorably with winding roads.

As a general rule, it will be more advantageous to carry a new through route past the outskirts of a town, and connect up with some good branch road to the urban area, rather than attempt to carry the new main thoroughfare through the heart of a populated center, as the difficulties and costs of widening existing narrow roads through built-up areas are necessarily excessive, owing to the property and business interests disturbed. Very wide roads are not favored by shopkeepers, as they are not conducive to good trade—the bulk of pedestrian traffic usually keeping to one side of such a road.

Curves on a new main road should, of course, be as easy as circumstances will permit, but a radius of 100 ft. should be the minimum where fast through traffic is to be accommodated. On this curve, a person traveling along the center line of a clear 40-ft. roadway could see approaching traffic within the limits of the road-width about 120 ft. ahead. Under similar conditions, with a 150-ft. curve, a distance of about 150 ft. ahead could be seen. On rural roads, traffic invariably uses the center of the road by preference, and modern high speeds render an ample, unobstructed view essential.

The easing of curves invariably quickens the speed and reduces that degree of commendable caution in drivers which formerly existed. On a sharp curve the motorist is bound to materially reduce his speed or perish.

Curves should in all cases be freed from side sight-blocking obstructions, and for increased safety to fast traffic, the road surface on the outside curve should be given a suitable degree of super-elevation.

On some of his work Telford adopted a ruling or maximum longitudinal gradient of 1 in 35. Such a moderate gradient will present no impediment to fast driving, either up-hill or down, and in one on which tar-macadam and all modern methods of surfacing may be used with safety, but in hilly country will be difficult to maintain, except by much contouring and consequent increased length of route, or by heavy cutting and bridge work.

Level roads are to be avoided. If the longitudinal inclination is less than about 1 in 100, the surface water will be difficult to drain away and more cross camber must be provided. With a longitudinal gradient of 1 in 50, or sharper, the camber may be flattened considerably, as the needful surface drainage is obtained longitudinally. This flattening of cross-section should not, however, be carried too far, otherwise watercourses will speedily form down the center of the roadway on steep gradients.

Suitable cambers for different surfaces under ordinary conditions are: granite macadam, 1 in 25; tar-macadam, 1 in 30 or 1 in 40 on incline; creosoted deal paving, 1 in 36; hard wood and granite, 1 in 45; and asphalt paving, 1 in 50. The longitudinal fall of water channels should not be less than 1 in 100 for a granite channel, and 1 in 150 for asphalt.

IMPROVEMENT OF EXISTING ROADS.

Inadequate Foundations.—The majority of the old highways of this country have come into existence in a more or less haphazard fashion, and in the past the provision made for their maintenance has been uncertain and inadequate. The entire absence of foundations suitable for carrying modern weights brought upon the surface is revealed by the upward movement of the sides of the roadway into the ditches—a weakness to be observed generally throughout the country.

Vast sums are now being spent in laying down expensive so-called waterproof surface crusts of various kinds, in order to give an immediate show for the money expended, but the writer is of opinion that, wherever there is evidence of underlying weakness, money may be more advantageously applied by first putting in proper foundations and drainage. However excellent, well laid or expensive may be the surfacing material, it can never prove really satisfactory on a weak foundation. There will be gradual but constant movement of the road crust, local sinkages of the central portion and rising of the sides, and generally the annual expense of wear and tear of the surface will be greatly enhanced in cases where no solid and unyielding foundation exists.

The initial cost of such work is, of course, necessarily heavy, but where suitable foundations do not already exist, it is submitted that it is the only sound course to pursue, both from an engineering and financial point of view.

Thin Crusts.—The wisdom of recent practice in laying thin surface wearing crusts or armourings of asphalt and bituminous preparations over existing macadam surfaces is open to great question, except where an absolutely rigid and dry foundation can be relied upon. In the absence of this, most ordinary road surfaces are subjected to considerable movement under modern speeds and weights, and in these circumstances, thin wearing crusts are liable to fracture and disintegrate. Great caution in the selection of a suitable site is necessary, except where laid on a good concrete foundation, which, unfortunately in most cases, would make the cost of the work prohibitive.

Bituminous Methods.—In what are now known under the general name of "bituminous methods," the presence of coal-tar and pitch is the distinguishing feature, and the main object of all such processes is to exclude water from the crust of the road. These methods consist mainly in the revival and extended use of "tar macadam," "pitch grouting," "tar binders," and other like forms, and in suitable situations and conditions give very serviceable surfaces at a not unreasonable cost, but much discrimination is needed in their application.

The arch enemy of the tar macadam road is the traction engine, and this cumbersome vexatious contrivance is, unfortunately, rather in evidence in the writer's district, especially during the spring and early summer months. These engines, with destructive diagonal steel strips on the wheels, weigh over 16 tons on the road, and haul three lumbering wagons weighing about 12 tons each when loaded. Under this burden the very best of tar macadam work suffers substantial damage. Even after having been laid many months the material, owing to its plastic nature, will slightly soften on a hot day, sufficient to permit of its being crushed out of shape and torn up under traffic of the class named.

A well-made granite macadam road surface withstands this class of traffic very much better than tar macadam, or any of the bituminous processes.

To the enthusiast for tar macadam, in addition to the above caution, the writer suggests that consideration of the following points will help to keep him out of trouble:

1. The quality of tar available for such work is very variable and unreliable, and requires constant watching to avoid failure. 2. Good, fine weather is an important factor for successful work. If the weather is too cold there is great risk of too much tar being used, thus causing the tar macadam to become very soft and easily damaged during the warm weather. 3. Tar macadam is hopeless on a weak, yielding foundation. 4. It cannot be satisfactorily repaired during wet, cold weather. This is important in streets where much opening of trenches for gas, water, electric and other services is likely to be required. 5. Where there is much traffic a continual watch must be kept on the work for some time after it has been completed, which considerably adds to its cost. 6. Tar macadam is liable to "creep" during hot weather towards the sides of the roads, especially in country districts, where the lateral support of a curb and footpath is not usually available, and some provision to meet this tendency should be made. 7. The cost of tar macadam is, as a rule, much beyond that of a granite macadam tar painted surface, and its serviceable life cannot always be so accurately predicted.

The foregoing matters are mentioned, not with any desire to discourage the use of this type of road surface, but simply with the object of drawing attention to a few points which require consideration to ensure successful work.

With regard to the "pitch grouting" of road macadam, the writer is of opinion that this process has not yet been proved to be so satisfactory under like conditions of traffic, as first class tar macadam well laid. The work is expensive, its serviceable life is not great, the surface soon becomes deeply corrugated and carries much slippery mud during wet weather.

Coatings of tar macadam of a total thickness of 4½ ins. are sometimes laid with a layer of coarse material at the bottom, and finished with a fine grade for the surface. A coating of the thickness named is best laid in two layers, but there is no advantage in separating the fine and coarse grades; the same mixed grade material should be used for both coats.

Rolling of tar macadam is best done with a light roller (6 or 7 ton weight), and there is nothing to be gained by an excessive amount of rolling. Tar macadam may be laid on gradients as steep as about 1 in 25, and even sharper if the surface is kept clean. The degree of slipperiness experienced depends greatly on the weather and the skill of the driver.

It is a great mistake to lay the tar macadam, or any other bituminous road surface, over an existing macadam roadway as a foundation, without first lightly scarifying the surface all over and consolidating by rolling to a *uniform* condition before the bituminous material is laid. Where this precaution has been neglected the old inequalities and "pot holes" in the road crust will soon reappear on the surface of the newly laid coat, as the greater depth of bituminous material over the "pot hole" consolidates more than the thinner coating around.

Surface Treatment with Tar.—Experience of recent years has led to the settling down of much preliminary clamor about "dust-layers" to the very general use of coal-tar. This new demand has brought about a very substantial increase in the price of tar, and, with the continued extension of bituminous methods of road construction, the demand appears likely to exceed the supply, and to set a limit on this form of road improvement unless competitive processes are adopted.

The specially prepared tar for road surface painting used by the writer weighs 12.95 lbs. per gallon, or 173 gals. to the ton. This is heavier than the weights recommended in the English Road Board Specifications, but the tar is found to make very satisfactory work.

Tar painting is not usually very successful on roads with damp clayey sub-soils, in shady situations, or under trees. A dry sandy or chalky sub-soil is the most favorable for the work, and in these areas operations can be started earlier in the season.

The heavy, complicated, costly tarring machines of early tar painting days have almost disappeared in favor of much simpler plant, and hand work—the latter giving the best results in this class of work.

For town work granite chippings ⅜ in. to ⅝ in. gage make the best class of grit for covering the tar. Sand, though usually much cheaper, produces an increased quantity of mud and causes the tar painting to tear up more readily under heavy traffic.

In the author's experience the amount of money spent on road tarring is about equivalent to the saving obtained in ordinary maintenance and wear and tear on the roadways, so that no increase on the total cost of highways arises, while a greatly improved surface is obtained during the summer and autumn months. Tar painting gives most economical results on secondary roads, and other thoroughfares with light traffic, as, in such cases, the tarred surface remains in good condition for several years without repair, and very little attention of any kind is needed.

Repairs.—Systematic inspection of the roadways, and regular and prompt patching of depressions and "pot holes" is very desirable, especially on motor omnibus routes. For this work the writer uses a light roller of the convertible tractor type, which is well adapted for the purpose.

Under modern traffic conditions the highways require to be regularly patrolled and repaired, much in the same way as a railway track. Old screened road metal, of small gage, is well suited for patching, as it consolidates quickly. Ordinary macadam surfaces should not be patched with tar macadam as, after a little wear, a most unsightly and intolerably bumpy surface results, owing to want of uniformity in wear of the variegated surface.

When recoating a roadway, the thoroughfare should be closed wherever possible, and the whole width of road coated and rolled in one operation. Work of this class done in half-widths is seldom satisfactory, as rapid wear invariably occurs at the central joint. In cases where this system cannot be avoided it is best, if possible, to first treat about two-thirds of the width, so as to keep the joint out of the center of the road; but many roadways are too narrow to permit of this being done.

Weather conditions are among the most powerful factors influencing the wear and tear and deterioration of roads. Prolonged rain, and heavy traffic following the break-up of frost immediately succeeding a wet period are particularly destructive.

In town streets where a macadam surface has to be renewed about every two years, and patched frequently, a wood paved surface will probably be more advantageous. A maintenance cost of 20 cts. per square yard per annum is about the economical limit for macadam, and, from the point of view or traffic weight, a load of some 250 tons per yard of width per day is about the maximum for an ordinary macadam surface.

Steam Rollers.—In the opinion of the writer the usual so-called "10-ton" steam roller is much too heavy for the majority of surface recoating work. These rollers, when loaded ready for the road, often weigh nearer 13 to 14 tons than 10, and frequently cause much damage to the new metalling by crushing and weakening it during the process of consolidation. In some cases metal is put on the hard crust of the old road, without preliminary scarifying, and rolled down with a heavy steam roller, with the result that the stone, being severely crushed between hard surfaces, is permanently damaged at the outset, and the serviceable life of the new coat thus sadly reduced.

For much of his work the writer uses what is described by the makers as a 7-ton roller, of the convertible roller-tractor type. This machine is found to be of the greatest service for all classes of work as well as for haulage. The small tractor-roller can be moved quickly from job to job, and can be converted to a tractor in about a couple of hours. This roller, with awning, water, etc., fitted up ready for work on the roads actually weighs 9.42 tons.

MISCELLANEOUS DETAILS.

Mechanical Haulage.—Whatever may be the views held as to the desirability of public highway authorities employing steam, gasoline or other motor vehicles for haulage purposes, the writer has been practically compelled to do so, on account of the difficulty experienced, during the busy spring and summer seasons, in procuring sufficient suitable horses. It is usually impracticable to keep, during the relatively quieter winter months, a full stud of horses sufficient to cope with all work during the busier period of the year, and such work as street watering and road tar painting greatly accentuates the variation between winter and summer haulage demands. The hiring of the surplus summer requirements affords one way out where the horses can be got, but horse contractors are fast changing to mechanical haulage, thus greatly limiting the supply.

Slippery Road Surfaces.—In these days of improved road surfaces, tar painting, tar macadam and such like, complaints are perhaps a little more frequent in respect of slipperiness, and requests for gritting or sanding are often made. The best plan to overcome slipperiness is to keep the surface as *clean* as possible, by removing (and washing off if necessary) stiff pasty mud which is liable to accumulate during the foggy, damp weather of the winter months. Gritting and sanding greatly increases the production of this stiff slippery mud, as the material is speedily crushed by the traffic. The application of grit, therefore, should be as sparingly as possible, and cleansing of the surface should take its place.

Road Signs.—Generally speaking, there is room for improvement in road direction signs. Frequently they are so placed that an approaching traveler cannot read the sign without stopping, and even sometimes dismounting. The direction arm should be at the most favorable angle, the letters not less than 3 ins. in height, and the mileage stated in bold block figures to the nearest quarter. Strangers motoring long distances often find it impossible to quickly gather the name of the place they are passing through, and it would be a great convenience to have the name of the village or town boldly erected on the through roads near the commencement of the village buildings.

Street corner mirrors are costly to erect and maintain, and the moving reflected image is liable to mislead a motorist unaccustomed to them.

It is a mistake to multiply danger and caution signs unnecessarily. They should be confined to the most awkward spots, and be erected by a public authority. No private signs should appear on a public highway. If too numerous, the familiarity of their appearance leads to a general neglect of the warning intended to be conveyed.

STATISTICS OF TRAFFIC.

The expression of traffic records in tons per yard width of roadway does not, in many cases, give a true representation of the amount of wear and tear over a roadway. On roads through rural districts with a comparatively small amount of traffic, a very large percentage of vehicles keep to the center of the road—in fact, the less the volume of traffic the higher will be the proportion using the center of the roadway. Thus the sides suffer but little wear, while the centers soon become worn out. The average traffic record per yard width does not therefore correctly show the conditions which obtain and a minimum and maximum record is necessary to convey the true facts.

Much the same thing often occurs on very wide roads, as a portion of the width only is used by a high percentage of the traffic.

On busy town roads of medium width the traffic is very fairly distributed, and it is mainly to this class of road that the average tonnage per yard has any reliable significance.

On some roads, owing to the nature of local industries, the *night* traffic is quite an important item, and should not be omitted if a true record of actual conditions is to be obtained.

The collection of information and statistics, in reference to roads, traffic, materials, and other like matters, is a useful occupation from many points of view so far as it goes, but the road engineer should be cautious as to the conclusions he may safely draw from the collected data placed before him. So much depends on the actual conditions in any given case, and it does not by any means follow that because a road material has proved satisfactory or otherwise in one place it will necessarily do so in another. A new set of conditions will produce its own set of results, and the engineer must use his judgment in each case according to his local knowledge and experience.

Concrete Paving Between Car Tracks.

(Staff Article.)

Methods of treating concrete pavement between street car tracks vary in different cities. When a grooved rail is used the problem is comparatively simple; but with a T-rail several designs have been used. Figure 1 illustrates two methods that have been extensively used: (a) that followed by the city of Duluth, Minn., and (b) the Superior, Wis., method.

(a) (b)

Fig. 1. Treatment of Street Car Tracks on a Street Paved with Concrete. (a) Duluth Method, (b) Superior Method.

In the Duluth method the concrete is carried level from rail to rail, the depression next to the rail head being at an angle of 45° and sufficiently deep to clear the wheel flange. The Superior method is more commonly used and consists of simply a long bevel from the bottom of the rail head to a convenient point toward the center of the car tracks.

At Sheboygan, Wis., the Duluth method is used. The ordinary method of construction is as follows: A wooden strip, prepared by cutting diagonally a 2x2-in. piece, is tamped down along the inner edge of the rail head until level with the rail. The concrete already placed with a slight crown between the rails is struck with the strip in place. After striking, the strip is removed and the concrete broomed. A very slight trimming is necessary to form a good groove for the wheel flange. Up to the present time no trouble has been experienced from the entrance of water into the pavement along the joint.

At Mason City, Ia., the concrete is placed from curb to curb at one operation with practically no regard to the car tracks. Transverse expansion joints are placed between the rails in the same manner as if there were no car tracks. After placing the concrete street cars should not be permitted to use the tracks for at least four weeks. Where this plan was followed on 4,000 ft. of pavement having car tracks, no trouble has been experienced from the separation of concrete from the rails and the consequent admission of water into the subgrade.

The vibration of the rails under heavy traffic has not, so far, resulted in excessive damage to the pavement. It would seem that the rigid bond between the steel and concrete is an advantage in this respect, although it is probable that in the course of time separation must occur from temperature changes and other causes.

Weights, Volumes and Dumping Angles of Various Materials.

The following table of the weights and volumes of various bulky materials furnishes a rapid means of comparison and is useful for reference purposes. The dumping angles of some of these materials are as follows:

	Dumping angle in degrees.
Brick, coal gravel, sand (dry) and shingle	40 or less.
Rubble and clay	45
Loose, vegetable earth	28
Compact earth	50
Clay	16-45
Practically any material	55

Table I is a tabulation of weights and volumes.

TABLE I.—WEIGHTS AND VOLUMES OF VARIOUS MATERIALS.

Material.	Weight in pounds of Aggregate. 1 cu. yd.	2 cu. yds.	3 cu. yds.	4 cu. yds	5 cu. yds	Number of cubic yards of given material to weigh. 1 ton	1½ tons.	2 tons.	3 tons.	4 tons.	5 tons.	6 tons.
Ash, clinkers and cinders (mixed)	1,700	3,400	5,100	6,800	8,500	1.18	1.76	2.35	3.53	4.70	5.88	7.06
Asphalt	[illegible]	4,698	7,047	9,396	11,745	0.85	1.27	1.70	2.55	3.40	4.25	5.10
Brick (soft)	2,700	5,400	8,100	10,800	11,300	0.74	1.11	1.48	2.22	2.96	3.70	4.44
Brick (common)	[illegible]	6,000	9,000	12,000	15,000	0.67	1.00	1.33	2.00	2.66	3.33	4.00
Brick (hard)	3,375	6,750	10,025	13,500	16,875	0.59	0.88	1.18	1.77	2.36	2.95	3.04
Brick (pressed)	3,650	7,300	10,950	14,600	18,250	0.55	0.82	1.09	1.64	2.18	2.73	3.28
Brick (fire)	3,900	7,800	11,700	15,600	19,500	0.51	0.77	1.03	1.54	2.06	2.56	3.08
Cement (Rosendale)	1,620	3,240	4,860	6,480	8,100	1.23	1.85	2.47	3.70	4.94	6.17	7.40
Cement (Portland)	2,100	4,200	6,300	8,400	10,500	0.95	1.43	1.90	2.86	3.80	4.76	5.72
Charcoal (hardwood)	500	1,000	1,500	2,000	2,500	4.00	6.00	8.00	12.00	16.00	20.00	24.00
Charcoal (pine)	[illegible]	992	1,458	1,984	2,430	4.12	6.17	8.23	12.34	16.46	20.57	24.68
Clay	[illegible]	6,400	9,600	12,800	16,000	0.63	0.94	1.25	1.88	2.50	3.12	3.76
Coal, anthracite (lump)	1,875	3,750	5,625	7,500	9,375	1.07	1.60	2.13	3.20	4.26	5.33	6.40
Coal, anthracite (broken)	1,782	3,564	5,346	7,128	8,910	1.12	1.68	2.24	3.37	4.48	5.61	6.74
[illegible]	[illegible]	3,506	5,259	7,012	8,765	1.14	1.71	2.28	3.42	4.56	5.70	6.84
Coal, anthracite (stove)	1,736	3,472	5,208	6,944	8,180	1.15	1.72	2.30	3.45	4.60	5.74	6.90
Coal, anthracite (chestnut)	1,693	3,386	5,079	6,772	8,465	1.18	1.77	2.36	3.54	4.72	5.90	7.08
Coal, anthracite (pea)	1,646	3,292	4,938	6,584	8,230	1.22	1.82	2.43	3.65	4.86	6.07	7.30
[illegible]	[illegible]	3,000	4,500	6,000	7,500	1.33	2.00	2.66	4.00	5.33	6.66	8.00
Coal (Cumberland)	1,450	2,900	4,350	5,800	7,250	1.38	2.07	2.76	4.11	5.52	6.90	8.28
Coal [illegible]	[illegible]	2,720	4,080	5,440	6,800	1.47	2.20	2.94	4.41	5.88	7.35	8.82
[illegible]	[illegible]	1,720	2,580	3,440	[illegible]	2.33	3.49	4.65	6.98	9.30	11.63	13.96
[illegible]	[illegible]	6,000	9,000	12,000	[illegible]	0.67	1.00	1.33	2.00	2.66	3.33	4.00
[illegible]	[illegible]	4,300	6,450	8,600	10,750	0.93	1.39	1.86	2.79	3.72	4.65	5.58
Earth (loose)	2,000	[illegible]	6,000	8,000	10,000	1.00	1.50	2.00	3.00	4.00	5.00	6.00
[illegible]	[illegible]	6,000	9,000	12,000	15,000	0.67	1.00	1.33	2.00	2.66	3.33	4.00
[illegible]	[illegible]	4,000	6,000	8,000	10,000	1.00	1.50	2.00	3.00	4.00	5.00	6.00
[illegible]	[illegible]	[illegible]	11,400	15,200	19,000	0.53	0.79	1.05	1.58	2.10	2.63	3.16
[illegible]	[illegible]	[illegible]	4,200	5,600	7,000	1.43	2.14	2.86	4.29	5.72	7.14	8.58
[illegible]	[illegible]	5,000	7,500	10,000	12,500	0.80	1.20	1.60	2.40	3.20	4.00	4.80
Mortar	2,400	[illegible]	7,200	9,600	[illegible]	0.83	1.25	1.67	2.50	3.33	4.16	5.00
[illegible]	[illegible]	5,800	8,700	11,600	[illegible]	0.71	1.06	1.41	2.12	2.82	3.53	4.24
[illegible]	[illegible]	3,900	5,850	7,800	9,750	[illegible]	1.54	2.05	3.08	4.10	5.13	6.16
[illegible]	[illegible]	4,200	[illegible]	8,400	10,500	0.95	1.43	1.90	2.86	3.80	4.76	5.72
Quartz (broken)	2,700	5,400	8,100	10,800	13,500	0.74	1.11	1.48	2.22	2.96	3.70	4.44
[illegible]	[illegible]	4,320	6,480	8,640	10,800	0.93	1.39	1.85	2.78	3.70	4.63	5.56
[illegible]	[illegible]	6,400	9,600	12,800	16,000	0.63	0.94	1.25	1.88	2.50	3.12	3.76
[illegible]	[illegible]	6,000	9,000	12,000	[illegible]	0.67	1.00	1.33	2.00	2.66	3.33	4.00
[illegible]	[illegible]	9,200	13,800	18,400	23,000	0.44	0.65	0.87	1.31	1.74	2.17	2.62

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design and to Methods and Cost of Construction

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A Chemical Method of Gaging Stream Flow.

A most interesting article in this issue explains the new method of stream gaging by titration with which a number of European hydraulicians have for three of four years been experimenting. Any summary of this article that can be made here would help little toward a clearer understanding of the process. A fact that should be emphasized here, however, is that this process is of far more than academic interest. Actual turbine flow tests by titration are now in progress in this country. In a letter to the editors, Mr. Blaauw, one of the translators of the article which we publish, refers to and comments on these American tests as follows:

Just now Mr. Ward is applying this chemical method at Massena, N. Y., for the testing of four 5,000 HP. turbines. In this case the method will be checked up by the current meter and the pitot tube, but it is the intention to do testing in the future only by the chemical method. I believe that the chemical method will enable engineers to save considerable time and money. Three years ago, at Massena, N. Y., six turbines were tested at the enormous total cost of $6,000, representing almost a whole summer's work. The accuracy aimed at in that case cannot compare with the accuracy obtained by the chemical method, and—so it seems to me—the cost could have been kept down at least 50 per cent. For the tests for which Mr. Ward has gone to Massena, great accuracy is required (one-tenth of 1 per cent is aimed at) in order to settle the question of bonus or penalty, which in this case runs into the thousands of dollars.

Soliciting Public Confidence in Railway Securities.

Prospective railway construction and renewals depend upon the success with which railways can sell their securities. Public confidence in railways is essential to the successful marketing of railway securities. A very large number of people have lost confidence in railways and in their managers. It is true that a few maladministrators of railway properties are in large measure responsible for this condition, but it is just as true that railway managers very generally have contributed toward it and it is supremely true that whatever may be held to be the cause and the allotment of responsibility for this cause the time has come when to restore public confidence in railways a change is necessary in their principles of financial management.

Any action by the railways, which indicates change of this sort is noteworthy, and one therefore observes with interest the meeting last week with President Wilson of a committee of six high officials of railways to urge his aid in restoring public confidence in railway securities. This committee consisted of Chairman Trumbull of the Chesapeake & Ohio and Missouri, Kansas & Texas boards, Samuel Rea of the Pennsylvania, Daniel Willard of the Baltimore & Ohio, E. P. Ripley of the Atchison, Topeka & Santa Fe, Hale Holden of the Chicago, Burlington & Quincy, and Fairfax Harrison of the Southern. The specific request of the committee as reported in the dispatches was as follows:

That the president will call the attention of the country to the pressing necessity for support of railroad credit by co-operative and sympathetic effort of the public and of all governmental authorities, and that the railroads be relieved so far as possible of further immediate burdens involving additional expense.

That the president will urge a practical recognition of the fact that an emergency is upon the railroads which requires in the public interest that they have additional revenue and that the appropriate governmental agencies seek a way by which such additional revenues may be properly and promptly attained.

The emergency upon the railways is the necessity of meeting a vast amount of maturing obligations by further issues of securities. To enable themselves to sell these new securities, the railways want permission from the Interstate Commerce Commission to increase rates and consent of the Federal Reserve Board to accept railway securities at face value in the transaction of the banking business.

As their affairs stand at present there is not much doubt of the need of the railways. In appealing for public sympathy, however, it is not enough for the railways to exhibit the extent of their debts maturing; the public wants to know how this indebtedness came about, to know where the money owed was put. Referring to the appeal to President Wilson of the committee named above, the dispatches from Washington state:

Admitting that the public no longer has much faith in railroads and their management, the committee threw its cards on the table and asked President Wilson to address an appeal to the nation and to government agencies to be more charitable and more favorable towards the carriers in the future.

Doubtless the committee was less suppliant than this statement indicates, but unless one misjudges public opinion supplication without justification and without indication of reform will not be very effective in securing the faith which, it is complained, is now lacking. Reform is needed in the attitude of the public toward the railways, but it will not come permanently until some reform in railway financial management occurs or is compelled.

The Work of the Maine Highway Commission.

The law under which the present highway commission of the state of Maine is working may be said to be characterized by its clearness and correlation between different sections, avoiding conflicts of authority and difficulty in interpretation. A number of its features are worthy of mention. The division of the highway system into state and state aid roads, the former, to be constructed wholly by the state, the latter, jointly by the state and municipality, is in accord with modern ideas of highway economics. The provision for maintenance, while insufficient, is definite and is to be commended. The budget system of preparing plans and requesting aid a year in advance of construction is worthy of note. A sliding scale of state aid by which the relatively poorer municipalities receive a larger proportion of aid than the more wealthy communities should tend to produce uniformity in type of construction—a matter of importance on through roads. The provision for the retirement of bonds with money derived from a vehicle tax is a direct method of taxation and results obtained are worthy of study.

While the present commission has not been in power a sufficient length of time to demonstrate the possibilities of the highway law, a definite conservative beginning has been made on the road system of the state. For the most part, Maine is a sparsely settled state. In the southern portion, however, large cities exist and the road traffic is heavy, consequently, while in the southern section road surfaces adapted to heavy traffic are used, by far a larger portion of the roads will be surfaced with a material economical for use in thinly settled regions.

An excellent type of gravel road has been developed in this state. By reference to the abstract of the gravel road specifications contained in this issue it will be noted that both a two and three course type of construction are used.

The system of state roads contemplated will give ready access to all portions of the state. Direct roads are provided to the important fishing and hunting regions, a fact which will doubtless be appreciated by sportsmen. Moreover, an outlet is provided for the produce of the residents.

The progress made by the state of Maine toward a conservative and effective system of roads to provide for the needs of the people should furnish an incentive to other states to undertake their work in a like manner. An excellent start has been made and before many years a large mileage of improved road in this state will be available to aid in the development of internal resources.

Fire Escapes for Such Buildings as Charitable Institutions and Hospitals.

The difficulty of providing adequate fire escapes for various types of buildings has long been recognized by architects and engineers, yet one cannot feel that a satisfactory solution of the problem has been made. It is at once recognized that each class of building must be considered by itself in working out systems of fire escapes. In residences and in buildings where the number of occupants is relatively small, the outside stairway and ladder type of escape provides facilities as elaborate as are justified, although such escapes are unsightly. In commercial buildings, factories and charitable institutions, where the number of persons in a single building is large, it has been found extremely difficult to provide fire escape facilities which will meet the demands made upon them in time of extreme danger. It is in charitable institutions and hospitals that the necessity for the highest type of fire protection devices is felt. For such buildings outside stairways, chutes and even elevators have not proved satisfactory, as the inmates are at least partially helpless and cannot make the best use of such fire escapes.

An important step toward providing better fire escape facilities has recently been taken by John A. Kingsbury, commissioner of charities of New York, when he instructed H. F. J. Porter and A. L. A. Himmelwright, engineers, to proceed with the development of a special system of life protection in case of fire for the hospitals and other buildings of the department of charities. The system of "horizontal escape" recommended consists of introducing a dividing wall across the building, extending from basement to roof, with a doorway fitted with a fireproof door at each floor. In case a fire should occur at any floor on one side of the wall, the people would simply pass through the doorway into the safe section, an alarm signal being used to give notification of the fire. The occupants could then reach the ground using the elevators and stairways of the safe section, as under normal conditions. This system of horizontal escape has been used to some extent in commercial and factory buildings, although in many cases dividing fire walls have been provided without doorways. The cost of installation of such a system is usually low, in fact the system can be installed in buildings which contain an ordinary division wall at a very small expense, the thickness of the wall being increased, where necessary, to form an effective fire barrier and an opening provided with a fireproof door. It is seen that the efficiency of such a system depends to a considerable extent upon the doorway, the door for which should swing in both directions.

The officials of the department of charities and of the fire department of New York recommended the adoption of this system after an investigation which has been carried on since the first of the year. The department of correction has also asked permission to have the work extended into its buildings, and Bellevue hospital has already had its buildings surveyed and has asked for an appropriation for a similar installation. The need of better fire protection facilities is emphasized by each disastrous fire; and the increased activities of public service commissions in providing for the safety of the public will undoubtedly result in the installation of more effective systems of fire escapes than those now in common use.

BUILDINGS

Design, Construction and Unit Costs of the Power House for the New Smelter of the Arizona Copper Co., Ltd., Clifton, Ariz.

The Arizona Copper Co. has recently constructed a new smelter at Clifton, Ariz., which is one of the most complete in this country. Construction was begun in February, 1912, and the work was completed in February, 1914, the total cost of the improvement being $2,105,020. Figure 1 shows a layout of the smelter and gives some conception of the extent of the improvement. The drawing also shows the 10-ft. contour lines. In this issue we shall describe the design and construction features of the power plant and shall give detailed cost data applicable to the material and labor items. The article is based on a paper by E. Horton Jones, in the Bulletin of the American Institute of Mining Engineers. The de- [illegible] Copper Co.

[illegible]

The main portion of the power house (for location see Fig. 1) has a length, between outside column centers, of 276 ft. and a width of 52 ft., with an extension on one side having dimensions of 26 ft. 9 ins. by 92 ft. The height of the main portion of the building, [illegible] is 48 ft., while that of the extension is 34 ft. 9¾ ins. A traveling crane having a capacity of 20 tons and a span of 50 ft. serves the main section. The building has a steel frame and 8-in. tile walls, resting on the concrete basement walls. The concrete roof is waterproofed with a composition of tar, cement and coal oil. The concrete basement floor has a thickness of 4 ins. and was cast on the ground, in 6-ft. squares with sand joints. The foundations for the turbines, blowing engines, compressors and exciters extend from the basement to the main floor level. The space between these foundations and between the walls and the foundations, at the main floor level, has a reinforced concrete floor supported on steel I-beams. The total weight of the structural steelwork in the power house is 254.29 tons.

Figure 2 (a) shows a plan of the power house and gives the location of the engines, transformers, compressors, pumps and turbines, and Fig. 2 (b) shows a cross section of the building.

Figure 3 (a) shows a cross section of the power house and gives details of the structural framework and of the concrete foundation of the columns. The columns are spaced 23 ft. on centers, the main part of the building consisting of 12 bays, and the extension of 4 bays. The width of the main section is 52 ft., center to center of columns, and that of the extension, 26 ft. 9 ins. From the base up to the crane girder, a distance of 37 ft., the columns have a width of 24 ins., and consist of 4 5x3x5/16-in. angles with 2½x½-in. double lacing. Above the crane girder the column width is reduced to 15 ins. The outside columns of the extension have a width of 15 ins. and consist of 4 5x3x5/16-in. angles with 2¼x¼-in. double lacing. The columns are braced longitudinally with two lines of horizontal struts, each of which consists of 4 3½x2½x¼-in. angles, 24 ins. back to back, with 2¼x7/16-in. double lacing, and one line of 12-in. 20.5-lb. channels, to which the main flooring framing is attached. The positions of these struts are indicated in Fig. 3 (a). The crane girder consists of a 26-in. 90-lb. Bethlehem I-beam, supporting a 50-lb. crane rail.

Figure 3 (b) shows a detail of a typical column foundation. The columns are anchored to the concrete foundations with four ¾-in. bolts, 2 ft. long.

Figure 3 (c) shows a detail at the apex of the roof, and Fig. 3 (d) shows a detail at the gable end. These drawings indicate the type of roof construction used.

Figure 3 (e) shows the end framing of the main portion of the power house, and Fig. 3 (f) shows the end framing of the extension.

In general, the windows were equipped with steel sash, and with No. 18 "Caldwell" sash balances, although 40 of the small windows have wooden sash. About one-half the concrete sills were cast in place and the remainder were molded as separate members. These sills have a section 8x10 ins. for the windows and 3x10 ins. for the fixed sash at the top of the building. They are reinforced with three ⅝-in. rods.

Six 48-in. "Burt" ventilators, with square bases, are placed along the apex of the roof.

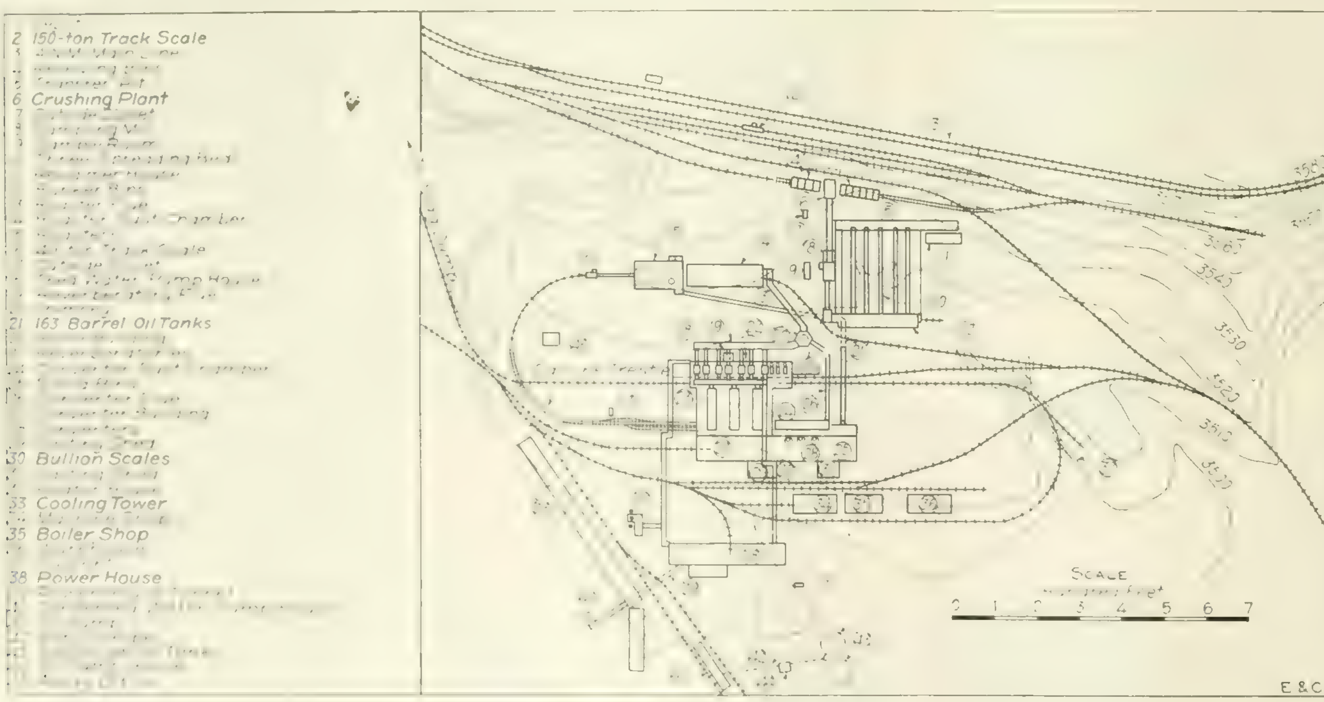


Fig. 1. Layout of New Smelter of Arizona Copper Co., Ltd., Clifton, Ariz.—Power House is Building No. 38.

CONSTRUCTION FEATURES.

Excavation.—The excavation required for the power house was about 55 ft. wide, 280 ft. long, and 10 ft. deep. The material encountered was red clay and boulders on top, with sand and gravel beneath, the latter materials being used for concrete. Powder was used, followed by plowing, picks, shovels, fresnos and carts. The excavated material was hauled 450 ft.

Column Foundations.—The piers for the steel columns are about 3x4x3 ft. The concrete was hand mixed, in the proportions of 1 part cement and 7 parts sand and gravel, and was transported about 150 ft. in wheelbarrows. Considerable difficulty was experienced in keeping out the sand and gravel, which constantly fell into the pits from the sides of the excavation. About 50 per cent of the vertical surface required the use of forms.

Foundation Walls.—The concrete for the walls below the main floor level was cast as a vertical slab, bearing against the columns of

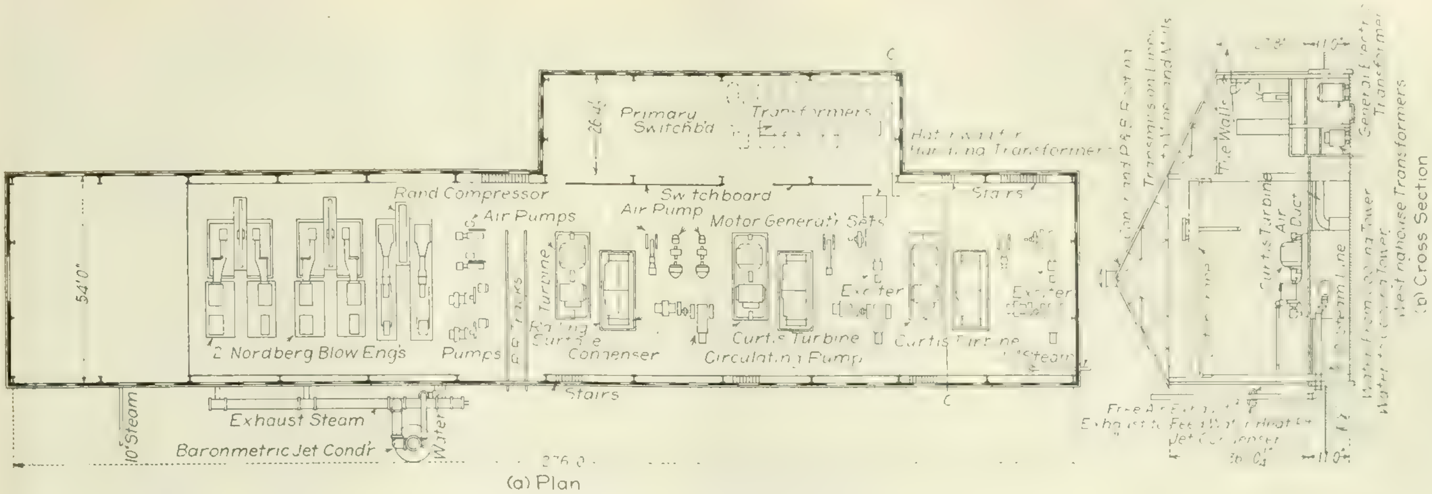


Fig. 2. General Plan and Cross Section of Power House for Smelter of Arizona Copper Co., Ltd. Showing General Dimension of Building and Arrangement of Equipment.

the building. These walls are 12 ins. thick at the top, 20 ins. thick at the bottom and 11 ft. high. They were designed as a reinforced concrete slab (using ¾-in. rods) to withstand earth pressure on a 20-ft. span. The concrete

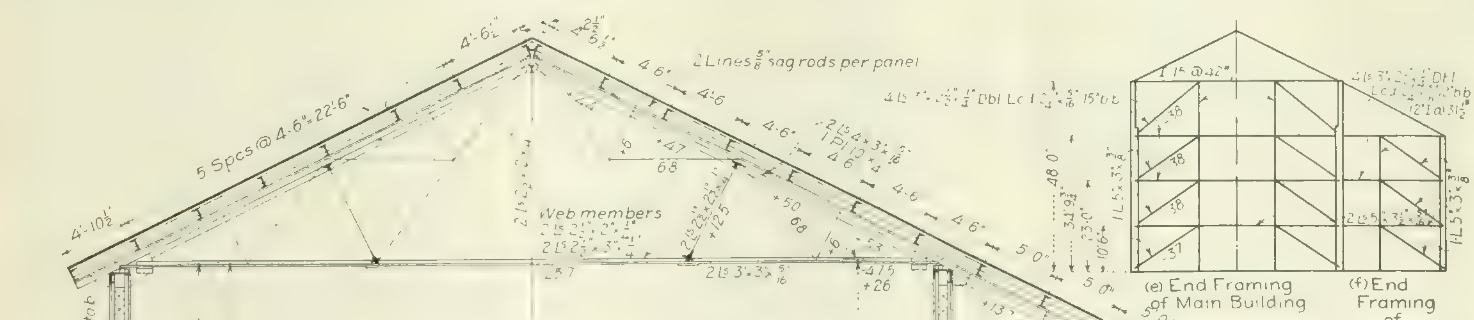


Fig. 3. Cross Section and Details of Power House for Smelter of Arizona Copper Co., Ltd. Showing Structural Features.

was machine mixed in the proportions of 1 part cement to 5 parts sand and gravel, and was wheeled a distance of about 150 ft. About 95 per cent of the vertical surfaces of the walls required the use of forms. One coat of "Wadsworth Howland Bay State" cement was applied to the basement walls.

Basement Floor.—The basement floor was cast on the ground in 6-ft. square blocks, with sand joints. The total thickness of the floor is 4 ins. After the concrete was laid a coat of cement grout was applied, and then the floor was painted with two coats of "Toch Bros." cement filler.

Exterior Walls.—The exterior tile walls are 8 ins. thick and about 40 ft. high. The tile walls were laid directly on the concrete basement walls. The mortar used consisted of 1 part cement, 1 part lime and 3 parts sand, while that used at the outside of the wall contained sufficient lamp-black to give a pleasing appearance. All joints were struck. The coping of the wall was formed by bolting a 2x4-in. timber to the top course of tile and another to the under side of the roof. Metal lath was placed between these timbers and plastered with cement mortar. The depth of the coping is 18 ins.

Main Floor Slab.—The concrete slab at the main floor level of the power house consists of the "Berger" multiplex plate laid on steel I-beams and covered with concrete reinforced with ½-in. and ¼-in. rods. In laying this floor the plate was laid on the beams and was not wired. The rods were then laid in the concrete at right angles to the grooves of the plate. The concrete was machine mixed in the proportions of 1 part cement to 7 parts sand and gravel, the ¾-in. top finish being mixed in the proportions of 1 part cement to 2 parts sand. The thickness of the floor, from the bottom of corrugations of the plate to the top of the finish, is 5 ins.

The "Berger" plate, which is exposed beneath the floor, was cleaned of rust and painted with two coats of linseed oil and white lead, which required the use of low scaffolds. The top surface of the floor was given two coats of "Tock Bros." cement filler and one coat of gray cement paint, after the floor had dried out.

Roofing.—The "Berger" plate used in the roof construction was hoisted by means of a single pulley and hand rope. When laid in position it was wired to the purlins with No. 10 wire. The concrete was hoisted to the eaves at various points and transported to position in hand buckets. The concrete proportions were 1 part cement to 5 parts sand and gravel. The concrete was troweled smooth and was then painted with a composition of tar, cement and coal oil. For this waterproofing coat, 11 bbls. of tar, 54 gals. of coal oil and 7 sacks of cement were used.

The reinforcing plate was cleaned and painted with two coats of linseed oil and white lead. The painters worked from swinging scaffold attached to the roof purlins. The moving of these scaffolds from place to place was slow and difficult.

Wood strips were embedded in the concrete around the base of each ventilator and across the roof at the juncture of the main building and the addition. A cement coating was then applied to the concrete, followed by an application of hot maltha, with three sheets of felt embedded in the maltha. Another application of hot maltha was then applied, followed by a course of three-ply P and B roofing paper.

Painting Sash and Woodwork.—The sash, doors and miscellaneous woodwork were coated with three coats of linseed oil and white lead.

EQUIPMENT.

The various engines, compressors, turbines, etc., were set on concrete foundations, the top of the foundations being at about the main floor level.

The equipment in the power house includes the following:

2 Nordberg blowers; designed to compress 10,- [illegible]

3 2,000-kw. Curtis turbines, direct-connected to 2,500 k. v. a., 6,600-volt, 60-cycle, 3-phase, [illegible]

[illegible] for raising voltage from 6,600 to 13,200 volts [illegible]

[illegible] in

2 Ridgeway, tandem, compound, slide-valve engines, direct-connected to 75-kw., 125-volt, [illegible]

3 dry vacuum pumps for surface condensers, [illegible]

[illegible] to supply 250-volt direct current. [illegible]

[illegible] and 13 ft. 2 ins. high [illegible]

[illegible] 10-in.; lines from boilers, 8-in.; lines to engines, 4 to 8-in. [illegible]

[illegible] turbines; pipes range from 3-in. to 42-in.

Water pipes; includes 30-in. cast iron suction [illegible] 16-in. wood-pipe lines from equalizing tank to [illegible]

Table I gives, for each item, the total cost of labor and of material, the quantity of material and the total unit costs. The costs here given are actual costs and were compiled by an efficient cost organization. Each cost item has been judged solely on its merits for use by an estimator. All delays due to changes in plans, delayed shipments, labor troubles, changes in working hours and rates, and variable weather conditions have been included. In compiling the cost data the Dewey decimal [illegible]

TABLE I.—TOTAL COSTS OF LABOR AND MATERIALS, QUANTITIES OF MATERIALS AND UNIT COSTS OF POWER HOUSE AND EQUIPMENT.

Ref. No.	Account.	Total labor cost.	Total material cost.	Total labor and material.	Quantity of material.	Total unit cost.
9001	Excavation	$7,727.56	$ 69.09	$7,796.65	7,313 cu. yds.	$ 1.07
9002	Building foundation piers	1,699.92	1,460.02	3,159.94	231.7 cu. yds.	13.64
.1	Building foundation walls	3,735.78	3,628.81	7,364.59	508.5 cu. yds.	14.48
.2	North tunnel	1,350.79	1,230.37	2,581.36	180.3 cu.yds.	14.32
.3	Concrete drain	205.68	227.37	433.05	34.6 cu. yds.	12.52
.4	Basement floor, concrete	916.41	1,347.78	2,264.19	12,130 sq. ft.	0.19
.45	Basement painting	81.45	48.81	130.26	830 sq. yds.	0.16
.6	Preparation of concrete for painting	891.73	42.69	934.42	2,459 sq. yds.	0.38
.7	Painting concrete	195.84	301.61	497.45	2,459 sq. yds.	0.20
9003	Steel structure	....	....	23,773.30	254.29 tons	93.49
.1	Tile walls	3,856.83	4,510.20	8,367.03	14,343 cu. ft.	0.58
.11	Unloading tile	332.40	0.17	332.57	522.70 tons	0.64
.12	Wall coping	372.69	107.05	479.74	732 lin. ft	0.66
.2	Doors, windows and frames	974.38	3,319.93	4,294.31	4,044 sq. ft. opening	1.06
9003.21	Concrete sills	596.33	120.96	717.29	964 lin. ft.	0.74
.3	Ventilators	125.60	439.76	565.36	6 ventilators	94.23
.4	Main floor columns	236.93	626.44	863.37	68 columns	12.70
.41	Main floor slab concrete	1,267.91	3,341.61	4,609.52	10,210 sq. ft.	0.45
.42	Painting underside of main floor	181.88	147.58	329.46	2,679 sq. yds.	0.12
.43	Painting top of main floor	95.56	199.32	294.88	1,134 sq. yds.	0.26
.5	Roof, Berger multiplex plate	420.83	3,063.18	3,484.01	214.83 squares	16.22
.51	Roof concrete	1,723.10	958.51	2,681.61	214.83 squares	12.48
.52	Roof tar	172.70	127.73	300.43	214.83 squares	1.40
.53	Roof, downspouts and tile drain	286.17	240.44	526.61	905 ft.	0.58
.54	Roof painting, underside	692.84	324.55	1,017.39	6,813 sq. yds.	0.15
.55	Roof, P. & B. roofing	577.68	1,317.08	1,894.76	214.83 squares	8.82
.60	Painting sash	290.09	16.72	306.81	299 sash	1.02
.61	Painting woodwork	29.50	4.06	33.56	89 sq. yds.	0.38
9004	Crane	131.89	1,723.27	1,855.16	1 crane	1,855.16
9005	Well grading	1,558.07	517.68	2,075.75	2,600 cu. yds.	0.80
.1	Shaft sinking	765.02	612.10	1,377.72	45 ft.	30.61
.2	Timbering	57.61	....	57.61	45 ft.	1.28
.31	Aldrich pump installation	74.56	16.62	91.18	....	......
9006.01	Nordberg blowers, foundation	774.06	3,020.83	3,794.89	686.3 cu. yds.	5.53
.1	Nordberg blowers, cost and inst'l'at'n	1,641.62	32,514.02	34,155.64	2 Nordbergs	17,077.82
.2	Nordberg blowers, painting	327.57	57.65	385.22	2 Nordbergs	192.61
9007.01	Turbines, foundation	959.08	1,432.70	2,391.78	196.5 cu. yds.	12.16
.1	Turbines, cost and installation	2,297.70	79,586.49	81,884.19	3 turbines	27,294.73
[illegible]	Turbines, painting	286.15	41.02	327.17	3 turbines	109.06
[illegible]	Turbines, air pipe making	547.68	200.75	748.43	103 ft.	6.27
.4	Turbines, air pipe erection	232.57	64.24	296.81	103 ft.	2.88
[illegible]	Transformer trucks and transfer table	121.63	538.08	659.71	15 trucks	43.98
.52	Auto transformers	735.60	12,044.91	12,780.51	10 transformers	1,278.05
9008.01	Condenser foundations	291.08	285.18	576.26	50.3 cu. yds.	11.45
9008.1	Condensers, cost and installation	415.31	19,563.55	19,978.86	3 condensers	6,659.62
.2	Condensers, painting	30.00	5.86	35.86	3 condensers	11.95
9009	Jet condenser hot well, excavation	[illegible]	[illegible]	29.72	46 cu. yds.	0.65
.01	Jet condenser hot well, foundation	[illegible]	69.99	136.26	16.5 cu. yds.	8.26
.02	Jet condenser hot well, supporting structure and tank	...	....	945.74	5.76 tons	164.18
.03	Jet condenser hot well, cost and erection	128.97	494.68	1,078.65	1 condenser	1,078.65
.12	Jet condenser hot well, dry vacuum pumps	285.51	2,860.01	3,145.52	2 pumps	1,572.76
.13	Jet condenser hot well, pumps, painting	30.00	5.86	35.86	2 pumps	17.93
.21	Circulating pumps, foundation	560.01	708.93	1,268.97	210 cu. yds.	6.04
.22	Circulating pumps, cost and erection	366.90	3,535.68	3,902.58	2 pumps	1,951.29
.23	Circulating pumps, painting	30.00	5.86	35.86	2 pumps	17.93
9010.01	Air compressor, foundation	840.98	1,246.54	2,087.52	238.3 cu. yds.	8.76
.02	Air compressor, [illegible]	642.90	148.67	791.57	.....	.....
.03	Air compressor, painting	10.58	24.49	35.07	.....	.....
.04	Air compressor, all piping except steam	298.46	160.65	459.11	.....	.....
.05	Air compressor, wrecking and transportation	457.77	136.06	593.83	.....	.....
.06	Air compressor, installation of air receivers	49.47	1.43	50.90	.....	.....
9011.01	[illegible] pumps, 2 circulating pumps, foundation	1,439.67	1,875.43	3,315.10	373 cu. yds.	8.89
.02	2 exciters, cost and installation	491.01	6,118.26	6,609.27	2 exciters	3,304.64
.03	3 dry vacuum pumps, cost and installation	147.26	3,190.10	3,337.36	3 pumps	1,112.45
.04	[illegible] cir. pumps and engines, cost and installation	389.32	8,729.37	9,118.69	3 pumps	3,309.56
.05	2 exciters, painting	86.01	14.65	100.66	2 exciters	50.33
.06	[illegible] painting	50.00	8.79	58.79	3 pumps	19.59
.07	3 cir. pumps, painting	81.69	14.65	96.34	3 pumps	32.11
9012.01	2 motor gen., 1 air pump, 1 cir. pump, foundation	269.52	[illegible]	955.31	107 cu. yds.	8.93
.02	2 motor generators, cost and install'n	319.06	6,830.33	7,149.39	2 generators	3,574.69
[illegible]	2 motor generators, painting	[illegible]	[illegible]	35.86	2 generators	17.93
[illegible]	Transfer table pit, concrete	[illegible]	58.23	82.36	12 cu. yds.	6.86
.01	Switchboard, concrete compartments	1,472.21	510.48	[illegible]	1,469 sq. ft.	[illegible]
.02	Switchboard, cost and erection	2,730.53	15,520.57	18,215.10	.....	.....
[illegible]	Steam piping north and south mains, [illegible]	249.65		[illegible]	[illegible] cu. yds.	0.89
.01	Steam piping, foundation	578.24	945.97	1,524.21	194.5 cu. yds.	7.84
.02	Steam piping, steel supporting structure			[illegible]	86.81 tons	88.64
[illegible]	Steam piping, hangers and anchors	1,030.68	337.26	1,367.94	153 rods	8.94
.04	Steam piping, cost and erection	2,286.31	18,622.25	20,908.56	3,401 ft.	6.15
.05	Steam piping, covering and erection	266.71	[illegible]	6,079.94	3,401 ft.	1.79
[illegible]	Exhaust pipe, cost and erection	1,745.71	8,715.66	10,461.37	1,541 ft.	6.79
.01	[illegible] painting	85.05	51.19	136.24	1,541 ft.	0.09
.05	Exhaust pipe, covering and erection	318.25	830.56	1,148.81	746 ft.	1.54
.10	Air piping, cost and erection	363.19	554.16	917.35	.....	.....
.11	Air piping, painting	31.56	18.66	[illegible]	.....	.....
.2	[illegible] foundation	63.09	102.81	165.90	18.3 cu. yd.	9.07
.21	Exhaust pipe, supporting structure	197.27	57.93	255.20	.....	......
.22	[illegible]	20.32	....	[illegible]	29 cu. yds.	0.72
9016	Water pipe, excavation and backfill	1,485.10	0.24	1,485.34	2,406 cu. yds.	0.62
.01	[illegible] and erection	3,747.79	16,437.88	20,185.87	.....	.....
.02	Water pipe, painting	230.59	25.54	256.13	.....	.....
				$382,068.91		

[illegible] average wage scale was paid. The men worked 9 hours per day, except the brick masons, brick tenders and cement finisher boss. Mexican common labor was employed in some cases. Where this class of labor was paid in Mexican money the letter "M." precedes the quoted rate of wage.

Brick masons, 8-hr. day	$6.50
Brick tenders, 8-hr. day (M.)	2.25
Carpenter boss	5.00

Carpenters	4.50
Cement finisher boss, 8-hr. day	$3.50 to 5.00
Concrete boss	4.50
Concrete mixers (M.)	2.25
Labor bosses	4.00
Laborers (M.)	2.00
Pipe fitter boss	5.50
Pipe fitters	4.50
Pipe fitter helpers	3.00
Teamsters	$2.00 to 3.00
Teamsters, fresnos and slips (both A. and M.)	$2.00 to 2.50
Teamsters, plow (both A. and M.)	2.25
Steam fitters	4.50
Steam fitter helpers	3.00
Enginemen, compressors	3.50
Enginemen, stationary	3.00
Machinists	4.50
Machinists' helpers (both A. and M.)	3.00
Machine shop boss	5.00
Boilermaker boss	5.50
Boilermakers	4.50
Boilermaker helpers	3.00
Electrician boss	5.00
Electricians	4.50
Electrician helpers	3.00
Blacksmiths	4.50
Blacksmith helpers (both A. and M.)	$2.50 to 3.00
Water boys (both A. and M.)	2.00

The power house contains 32,096 sq. ft. of floor space. The cost of the building only, including items in Table I from 9001 to 9003.61, inclusive (exclusive of item 9002.2) was $77,452.56, making the cost, per square foot of floor space, $2.41.

The volume of the power house is 784,000 cu. ft., making the cost of the building, per cubic foot, 10 cts. The same items are here included as noted for the cost per square foot.

The total cost of the power house and its equipment was $359,590.10, making the cost of the building equipped $11.20 per square foot, or 46 cts. per cubic foot. In these costs are included accounts 9001 to 9004, and 9006.01 to 9016.01, inclusive, deducting one-half of accounts 9014 to 9014.05, inclusive.

Some General Principles Governing the Writing of Building Codes.

The importance to the engineer of securing adequate and impartial building codes for our cities is being realized to a much greater extent than formerly. In the past engineers have not interested themselves sufficiently in this work to secure proper representation on committees having in charge the drafting of building laws. The result has been that the engineering requirements of the building codes of various cities differ widely and are often unsatisfactory. The following data on the writing of building laws were abstracted from a paper by John A. Ferguson, engineer, Bureau of Building Inspection of Pittsburgh, in the Proceedings of the Engineers' Society of Western Pennsylvania. The author also gives definite suggestions for a building code for Pittsburgh, Pa., which were considered in our issue of Sept. 2, 1914.

THE LEGAL STATUS OF BUILDING LAWS.

In order to understand thoroughly the constitutional power back of building laws it is well to glance at the general plan of our government. The United States is the union, under the constitution, of the several states for common preservation and protection, the government having the right only to regulate matters in which the several states are interested, such as the making of treaties, the declaring of war and the regulation of interstate relations. The United States government has power to enact only such laws as are directly or by implication given to it by the constitution.

The states in general have the right to enact all such laws as are not specifically prohibited either by state or national constitution. However, the states cannot enact so-called "special legislation," such as regulating county, village or township affairs, incorporating cities or villages, changing or amending the charter of any town or village, or regulating the jurisdiction of police magistrates, etc. Consequently, the state can enact laws of a uniform character throughout the state, but must delegate to the municipality, etc., the right to enact such legislation, through the proper channels, as may be required by purely local conditions.

Among the various doctrines as to the right to enact or execute building laws there is the police power. This power is not defined by law, as it would tend to lessen its effectiveness. Owing to the lack of definition the legislature and those executives who are entrusted with the care of public safety can exercise broad powers. All regulation by legislation or by departmental rule governing the public health, safety and morals is based upon this police power. It is about the broadest power possessed by the government.

The right of eminent domain is said to cover the right of the state to enter private property and suppress or change its use where obnoxious to the public health, safety or morals. If property is not taken in this case there can be no claim for compensation. This last power is invoked when wrecking a structure to prevent a spread of conflagration, when an owner refuses to raze a dangerous structure, or when entering a property for the purpose of inspection.

Whenever a series of acts becomes a public nuisance the aggrieved parties have the legal right to abate it. However, the necessary steps must be taken through the properly constituted authorities.

It has been a maxim of common law and a fundamental principle with respect to real as well as personal property since the days of the Roman law that a person has the right to use his own property only to the extent that he does not thereby injure that of his neighbor. It will be seen that the public has a very definite and real right to take the necessary steps to prevent, as far as possible, the economic loss, as well as the loss of health, life and morals, of individuals. To accomplish this it is necessary to regulate fire prevention and protection, to see that sufficient exits are provided to prevent loss of life in panics, to provide against overcrowding in tenements, sweatshops or places of public assembly, to conserve the public health and morals and so to regulate the business relations of individuals as to conserve equal rights and justice to all in the making and enforcing of laws regulating the construction and occupation of buildings.

Since the state legislature cannot pass special legislation to make state laws different in one section from those of another, all legislation passed should be such that it will be proper to enforce it anywhere in the state, leaving to the local governments the right to add the laws required to provide for the purely local conditions. In order to conform to this the local government should have the power to organize its own enforcing body, the cost of which should then be chargeable to the city or town, and the personnel be appointed from those in the locality who are familiar with its peculiar needs. No part of such cost should be placed on the state to be distributed over the territory less able to pay for it and not benefiting therefrom. Every person in such an organization should be answerable to the body which has the appointing power for the proper conduct of his work. Reports of as general or detailed a character as desirable should be made to the state.

An additional reason for such arrangement can now be brought forward. It is based upon the fact that any body organized for the purpose of transacting business will completely fail to achieve its proper effectiveness if that body is extended too far and becomes unwieldy or overorganized, or if its personnel is chosen from among those whose interest is foreign to the work which is to be performed. This would become true in the case of localities where a large amount of building work is done.

Thus it is easily seen that, for proper effectiveness, any locality large enough to afford its own police officers should choose them from among its own members and provide for their discipline free from any but the most general supervision by the state. Personal supervision, such as giving state officers the right to remove a city officer without formality of preferring civil service charges, could result in demoralizing influences. Since this is not the case with the purely police officers it should be apparent even to the layman that the case of those charged with the enforcement of building laws is exactly the same as any police officer.

GENERAL REQUIREMENTS AND [illegible] A BUILDING COMMISSION.

As a result of the great diversity of opinion concerning what should and what should not be incorporated in building laws (and this diversity seems to be the result of the fact that there is so little widespread exact knowledge of the subject) it is well to outline at the beginning the kind of men who should be chosen to do this work, and the attitude of mind with which they should approach the work set before them. In the first place, it should be plainly understood that only those who are known by general reputation to be the best technicians in their various lines, as well as the broadest minded in the community, should be invited to assist in so exacting a piece of work. Then they should approach their task as if every policy and every general requirement or detail were to be arranged from an entirely new field and must be studied out with a broad view of the needs of the public rather than the especial hobbies of any individual, keeping in mind always that the law is only to conserve to the general public, life, health and property and not in any way so to restrain the individual that he cannot exercise the greatest amount of initiative possible in the conduct of his business.

Members of a commission for the revision of building laws should be so chosen as to have at least one representative for every business interest which would be largely affected by the laws it is proposed to formulate, and too much stress cannot be laid upon the fact that only the best and broadest minded from each interest represented should be chosen as members of such a commission. If there are distinct divisions in some of these interests, then the commission should have the power to employ men especially versed along such lines to report upon them, and to draft provisional matter in the proper form for the commission to review and take such action as they see fit.

A commission should have power to procure evidence, to call before it those having special information upon any subject, and to employ others, having experience, to advise them and even to provide outlines or briefs showing what should be in the laws. The best results always seem to have come from following the method of having one man cover the ground and others to check him up carefully. A commission should have the power to subpoena witnesses, and those especially interested, to testify under oath, and to hear the cases of the various interests, allowing them to submit proposed drafts of the various portions of the building law affected in each case.

Examinations of the various kinds of organizations and the results of the work of the various commissions recently appointed have shown that it is almost always advisable to invite various local technical and public organizations to look over the proposed laws and to add their suggestions and co-operate with the commission by furnishing sets of standard practice or any information of which they are in possession. By having all provisions fully discussed, any claims made later that partiality was shown or that special interests influenced the results are forestalled, and there is less occasion for future amendments, which may prove especially troublesome. It would be possible to gain much strength along this line by providing a permanent "Board of Appeal," selected by appointment from among representative men of the broadest ability in the community, and by requiring all such matters to pass through their hands before the law-making body would be called upon to act. If the building inspector then had the power to make rulings which would not become operative until endorsed by the Board of Appeals, on all those matters so difficult to cover in the body of a building law, but which are bound to come up, many matters now settled by the building inspector would either be shown to be incorrect and the decision changed, or would be held correct and enforced. Such a Board of Appeal would then be a kind of permanent commission for the revision of building laws. It should also act in the usual manner to decide questions when the interested parties feel that the building inspector has not arrived at the correct decision. As is usual, where professional services are performed, the members of this board should be paid for their time according to the work which they do.

As it will now be seen, those who are invited to perform the work of writing building laws are not taking the place of the law-makers. Such matters are not laws until they have passed the legislative body, either municipal or state, as provided by statute, the status of a commission being that of the professional man engaged to perform work which the legislator finds himself poorly prepared to do. In view of the vast amount of work to be done in all cases and due to the fact that those who are fitted to do the work have prepared themselves at great expense of both money and time in order to sell their services and special knowledge, it is not right for the public to demand their time without fair return commensurate with the service performed. Compensation in the form of a lump sum would be found to have a beneficial effect. Members of a commission should be chosen from among the leading members of technical and other societies. The following professions and vocations should be represented:

2 architects.
1 structural engineer.
1 sanitary engineer.
1 general contractor.
1 real estate dealer.
1 physician.
1 lawyer.

As the chairman of any body is the one upon whom the bulk of the responsibility is placed he should be either an engineer or an architect.

REQUIREMENTS OF BUILDING LAWS.

In the writer's judgment the best building law is the one which is built up around a careful and comprehensive series of definitions which classify all building requirements for health, public safety, public morals and fire protection. Such a series of definitions should be the whole attitude of the law, not the idea of restriction or the attitude of "thou shalt not." Each use to which a building is to be put, or may be put, should be defined and classified according to the hazard to be overcome and the minimum requirements set forth in clear, readable, everyday English.

Definitions which classify the different types of construction should come first, while definitions which grade the various structures according to their intended use and occupancy should come next. These requirements should be followed by a series of definitions and specifications governing the use of all building materials, taking each in turn and completing the requirements for each before undertaking the next one. A general type of construction governing a peculiar combination or use of building materials should be treated under a separate heading, viz.: "Steel or Reinforced Concrete Structural Framing," "Use of Terra Cotta Tile Construction," "Electric Wiring," "Plumbing," etc.

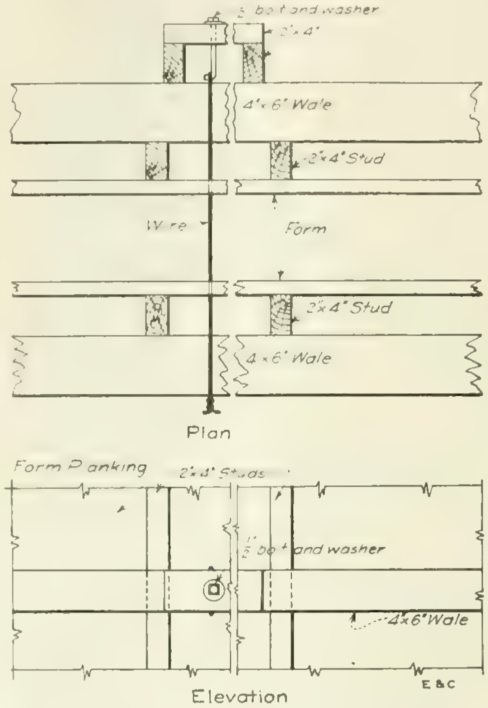


Fig. 1. Details of a Simple Wire Tightener for Concrete Forms.

When this has been completed it should be followed by a definition naming the class and grade required for every use to which structures are ordinarily put. This section should be treated as a more detailed classification. It should contain all the requirements, specified as carefully as may be, as to manner of fireproofing, materials of construction, and general safeguards to be thrown around the occupants for their well-being and safety. This last should contain the requirements as to safety from fire, proper entrances, exits, etc., as well as the general sanitary regulations. It should also specify the amount of space per person, ventilation, light, and proper placing of conveniences, and the numerous stipulations covering special hazards, such as the storage of explosives, the placing of automatic sprinklers, etc.

In this connection it will be found desirable to make special layouts for the treatment of such structures as tenements, apartments, workshops and factories, places of public assembly, theaters, garages, etc., which have a peculiar hazard. Everywhere it should be borne in mind that an architect, engineer, contractor or owner should be placed in possession of all necessary information to enable him to design and execute his work in conformity with the laws without any loss of time or fear of refusal of permit therefor.

ADMINISTRATION.

Finally, the value of any law depends, not so much upon what it contains, as upon the ability, common sense and efficiency of those engaged in its administration and enforcement. It might be possible to write a theoretically perfect law and not find one piece of construction work correctly done under it. One of the prime requisites is the choosing of capable and broad-minded men to do this work, assisted by efficient, conscientious inspectors. Much can be done toward insuring a proper administration of the laws by arranging the administrative portion, so that an intelligent man will catch hold of the right idea. Then conditions will be favorable for good building construction. On this account the administrative sections should receive very careful study.

Wire Tightener for Concrete Forms.

A number of devices are in use for tightening the wires used in concrete forms, several of which are of metal construction and are patented.

A simple wire tightener for concrete forms which is easily constructed is illustrated in Fig. 1. Its cost is very small, as short ends of form lumber can be used in its construction.

We are indebted to Mr. J. W. Caldwell, superintendent of Public Works of Hawaii, for an illustration of the device shown in Fig. 1. This form of tightener has been used with satisfactory results for several years on construction work in Hawaii.

WATER WORKS

Notes on the Design and Construction of Elevated Water Tanks.

At the latest annual convention of the American Society of Mechanical Engineers there was a fire protection session. At that session there were several papers presented by the sub-committee on fire protection. One of these papers, presented by Mr. W. O. Teague of Boston, was entitled, "The Need of More Care in the Design and Construction of Elevated Tanks." Mr. Teague discussed the subject primarily from the fire protection standpoint, but the greater part of the abstract of his paper here given, as taken from the Journal of the Society for April, 1914, relates with equal force to tanks erected to equalize water pressure and to provide some storage in municipal water works installations.

Elevated tanks for fire protection systems were first made of wood, but there are now as many being made of steel. Wooden tanks have been built up to 100,000 gals. capacity, although they are rarely larger than 60,000 gals., for above this capacity the steel tank is cheaper and more practicable. The cost of a 60,000-gal. tank of wood or steel erected on a 75-ft. steel tower is about $3,000. Steel tanks are built in large sizes, one of the largest being 1,200,000 gals. capacity; this one is 50 ft. in diameter and 90 ft. high, and is supported by a steel tower 130 ft. high.

Failures of tanks in service, involving loss of life and destruction of property, have shown the need of more care in their design and construction. To insure the best results, the following features should have attention.

WOODEN TANKS.

The tightness and durability in the wooden tank depends chiefly upon the quality of the lumber and the details of its construction. Selected tank stock only should be used consisting of white cedar, cypress, white pine, Douglas or Washington fir, or redwood, and the lumber should be free from sap, loose or unsound knots, worm-holes and shakes, and be thoroughly air-dried. Both the staves and bottom are usually made up of 2½-in. stock dressed both sides, for tanks up to 16 ft. in diameter and 16 ft. deep; for larger tanks 3-in. stock is used. Plank for this purpose should be full length without splices.

The strength of the wooden tank depends principally upon the size and spacing of the iron hoops. The importance of the matter of the hooping will be appreciated when it is realized that overstressing of even one hoop may result in bursting of the tank. The wooden tank being originally merely a development of the barrel where flat hoops were necessary to permit of tightening by driving them toward the enlarged middle, it was natural to use also flat hoops for the tank and the tank was also made tapered so that the hoops could be tightened by driving, although later they were tightened principally by hoop lugs. It was claimed that the tapered shape had also the advantage of preventing the hoops from dropping down over the tank, if it was allowed to remain empty and the staves to shrink from drying.

The tapered shape of tank is not important, however, since a tank which has been allowed to dry up has been seriously damaged thereby and cannot be made tight without extensive repairs, sometimes necessitating the rebuilding of it. In fact, most tanks are now made without taper and the hoops are found to remain where placed. The tapered tank costs somewhat more to build since the staves must be fitted more carefully and the design undoubtedly would have been entirely discarded long ago, except that some architects and purchasers believe a tapered tank presents a more pleasing appearance. The amount of taper is so small, being usually 1 in. per foot, thus giving a batter of ½ in. per foot to each side of tank, that its absence is hardly noticeable except on very high and small diameter tanks. The only objection to the tapered tank, however, is its extra cost.

In the early studies of this subject many serious failures of tanks were traced to weakening of the flat hoops by their rusting at the back where they bore against the staves, due to moisture from rain being retained between the surfaces of the hoop and staves. These failures were largely unpreventable, as it was difficult properly to inspect the condition of the hoops, and also impossible to paint them while the tank was in service. The use of hoops of round rod without welds has remedied this trouble as their surface is nearly all exposed for inspection and painting, and also they are not so subject to corrosion since the exposed surface of a round rod is much less than that of a flat bar or band of the same cross sectional area.

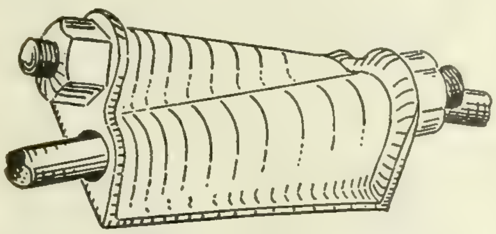


Fig. 1. Preferable Design for End Lugs in Malleable Iron for Round Rod Tank Hoops.

Another point of weakness in the flat hoop is at its connection to the cast iron lugs which is usually made by riveting. The use of round rod hoops, however, permits of a satisfactory connection to the lugs, but at first many tank failures resulted from the use of light cast iron lugs. These are now made of malleable iron, the best design being shown in Fig. 1. The hoops are so placed on the tank that the lugs do not come in a vertical line.

Round rod hoops are so spaced that the stress will not exceed 12,500 lbs. per square inch when computed from area at the root of the thread. The proper spacing can readily be found from the following formula:

$$\text{Spacing of hoops in inches} = \frac{\text{Safe load for given hoops (lbs.)}}{2.6 \times \text{diameter (ft.)} \times \text{depth (ft.)}}$$

The depth used is the distance from overflow to point where hoop is to be located. The top hoop is placed 2 ins. from the top of the staves and the spacing between hoops should in no case exceed 21 ins. An extra hoop or two is placed at the croze to take the additional strain due to the swelling of the bottom planks.

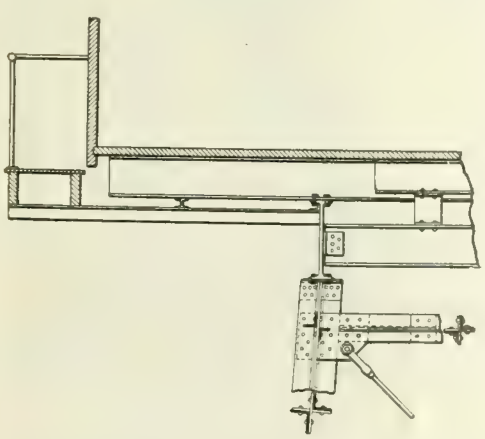


Fig. 3. Detail of Arrangement of Steel I-Beam Grillage for Support of Tank.

The tank roof, since it in no way serves to retain the water, has usually been nothing more than a makeshift cover. In the early days a single flat roof was used on outdoor tanks, but this held the snow and ice and required strong joist supports to keep it tightly in place. The snow also interfered seriously with the opening of the hatch to give access to the interior of the tank. A conical roof was then built over the flat one which remedied these difficulties. It also greatly increases the efficiency of the roof in preventing radiation of heat from the tank water in winter, as it provides a dead air space between it and the flat roof in addition to the one between the latter and the water, thus reducing the cost of heating in freezing weather. The conical roof also gives a better appearance to the tank top. A well-built roof is shown in Fig. 2. It should be tightly fitted around the tank top to maintain the dead air spaces.

Much trouble has resulted from leakage in the wooden tank, because it has not been firmly supported. The wooden tank is locally weak, not being of unit construction, and the lack of firm support has permitted working of the joints. It is supported only from the bottom, none of the weight being carried by the staves. Wooden beams were first used as supporting members, and these were placed on the roof of a building or tower as a grillage, and the tank bottom set on them. In time the wood rotted because of moisture from the tank bottom, permitting the tank to settle and causing leakage; there was also danger of collapse of tank because of this weakening of the joists. The use of steel I-beams as grillage members as shown in Fig. 3 avoids these difficulties. The beams should not be spaced over 18 ins. clear between edges of flanges, and the tank bottom is placed directly on the steel.

STEEL TANKS.

The simplest form of steel tank is the flat bottom one and tanks of this type give satisfactory service, provided the bottom is supported by a steel grillage as in the case of the wooden tank. One possible source of trouble is from corrosion of the bottom, and to prevent this in so far as possible the bottom plates are made somewhat thicker than is necessary for strength alone, and the grillage I-beams are of a height and spacing to permit of inspection and painting of the bottom. When the tank is to be placed on a concrete tower, it may rest directly on a reinforced concrete slab with the bottom thoroughly grouted in place with neat cement.

The preferred form of a tank to be placed on a steel tower is that having a hemispherical or elliptical bottom. The construction in this form is cheaper than for the flat bottomed tank, as the bottom is self-supporting and a steel grillage is unnecessary. The entire bottom is also accessible for inspection and painting, and corrosion is reduced to a minimum since the plates are exposed to the air.

Plates for use in steel tanks are made somewhat thicker than is necessary for strength in order to make them durable against corrosion. The minimum thickness is ¼ in., except that ⅛-in. plates are used for roofs. The plates composing the lowest cylindrical ring are 5/16 in. thick for 60,000-gal. tanks and larger, and the bottom plates 5/16 in. thick for tanks 75,000 gals. and larger.

One of the weaknesses of steel tank construction in the past has been poorly designed connections of the tank shell to the posts of the supporting tower. When the posts have a batter, as is usually the case, the inward thrust due to the horizontal component of the weight is provided for by a circular girder consisting of a ¼-in. plate 24 ins. wide, attached to the lowest cylindrical ring by an angle and stiffened by angles or a channel at the outside edge, as shown in Fig. 4. The posts also connect to the tank shell at this point and the design is such that the load will be transferred from the shell to the center line of the posts so as to avoid eccentric loading. A number of tanks built without circular girders have failed by the posts crushing in the tank plates. Others with the girder, but having eccentrically loaded connections to the posts, have failed by bending of the upper posts.

As the hydrostatic pressure on the tanks is comparatively small, it is not necessary to provide standard riveting for the thickness of plates used. The joints of the plates should be riveted so that the unit stresses on the net section of the plates and rivets will not exceed 7,500 lbs. for shearing and 20,000 lbs. for bearing. The horizontal joints are single lap riveted, except between the lowest

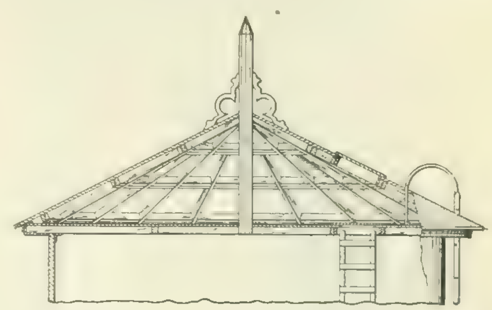


Fig. 2. Example of Good Design of Double Roof for an Elevated Water Tank.

cylindrical ring and the bottom, which are double lap riveted. The vertical joints also are single lap riveted except those in the lowest cylindrical ring, which are double lap riveted. The rivets are entered from the outside and driven from the inside and inside seams caulked. One of the strong features of the steel tank is that when once made tight, it gives practically no trouble from leakage.

TOWERS FOR ELEVATED TANKS.

Towers to support wooden tanks were originally built of wood, but with the increases in size of plant, buildings and extensions of them, considerably larger tanks and higher towers were required, and the builders, realizing the inadequacy of wooden construc-

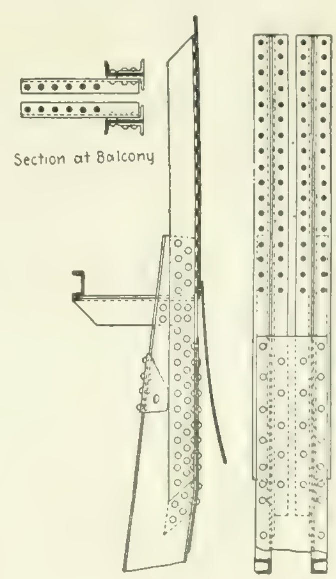


Fig. 4. Detail of Attachment of Tank Shell to Tower Post and Circular Girder Construction.

tion under these conditions, began to make towers of steel. The managements not being experienced in structural steel designing, naturally selected the simplest design possible for the towers. The posts and girts consisted usually of two angle irons, placed apex to apex and strapped together at intervals of several feet by tie-plates shop-riveted to the angles. The column sections were spliced by angles which were shop-riveted at one end to the post; the other end was field-bolted in erecting the tower, as this was the simplest form of connection and the easiest one to make. Furthermore, it had the advantage that the bolting could be done by the regular erectors, which made it unnecessary to have first-class mechanics in the erecting gangs and the carrying of special tools. This, however, was not good construction and the manufacturers are now field-riveting these connections.

The struts were at first connected directly to the posts by bolts. This construction is objectionable because the bolts are apt to work loose and it does not brace the parts. The construction now used is that of gusset plates riveted to the posts and girts. The wind rods were also connected directly to the posts at

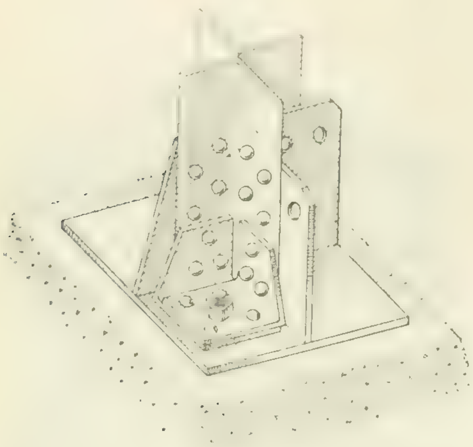


Fig. 5. Typical Construction of Footing for Angle Iron Column.

the girts. The bolt holes, as originally inserted through the post angles, weakened the posts, since they reduced the net section. The rods are now connected to the gusset plates. The arrangement of these parts is shown in Fig. 3. The diameter of bolt and thickness of plate are proportioned to provide proper bearing strength. The posts and girts of steel towers erected to support steel tanks, and to some extent wood tanks, are now largely made of channels latticed on both sides or having a plate on one side. Other shapes such as the Bethlehem H-beam and two channels with an I-beam between to form an H-section are also used to some extent.

Competition in the manufacture of these structures has resulted in the use of too high unit stresses and as a result the posts, figured on a conservative basis as represented in case of other structural work such as bridges, had a factor of safety of less than 4 and sometimes as low as 2½, and failure has resulted. To obtain safe towers it became necessary, therefore, to set maximum allowable stresses. The loading of the structure consists of the weight of the structural and ornamental steel work, platforms, roof, piping, etc. The live load consists of the weight of the total volume of water; the movable load on the platform is assumed to be 30 lbs. per square foot, plus the wind load. The wind pressure is assumed at 30 lbs. per square foot on a flat vertical surface and the wind load on the tank is taken as this pressure times 6/10 the projected area of tank and roof, and in the case of steel tanks, the curved bottom. The total wind load on the posts, struts, wind rods, ladders and riser boxing is assumed at 200 lbs. per linear foot of height of tower.

All parts of the structure are proportioned so that the sum of the dead and live loads shall not cause the stresses to exceed those allowable. The principal stresses in such a tower structure are axial compression on gross section of columns and struts, axial tension on net section of wind rods, bending on extreme fibers or net section of rolled shapes, built sections and struts, and shearing of rivets. The axial compression on gross section of columns and struts is determined from the following expression: $17{,}100-57\frac{L}{R}$, where L is the unsupported length of the members from center to center of connection in inches, and R the least radius of gyration in inches; the ratio $\frac{L}{R}$ should not exceed 125 for columns and 150 for struts and minor members and the maximum compression allowable as thus determined is 12,000 lbs. per square inch. The axial tension on net section of wind rods must not exceed 12,500 lbs. per square inch; the bending on extreme fibers or net section of rolled shapes, built sections and struts, 16,000 lbs. per square inch, and the shearing for shop-driven rivets, 10,000 lbs. per square inch and field-driven rivets 7,500 lbs. per square inch.

The lower ends of the posts have not been as carefully designed as their importance requires. Frequently, in angle iron towers particularly, no special attempt has been made to properly distribute the load to the base plate attached to the post footing. Cast iron plates were first used and the concentrated loading caused these to crack, resulting in collapse or in throwing the structure dangerously out of plumb with possibility of failure of the foundation under this post. The present use of steel plates has improved conditions, but the design must be such as to distribute the load to them as shown in Figs. 5 and 6, which are designs that are being used quite generally.

In anchoring the columns to the foundations, the diameter of the bolt at root of thread should be such as to withstand the maximum uplift due to the wind with tank empty, and to resist the shearing force at base plate. The bolts should be made from round wrought iron or mild steel rods without upsets.

FOUNDATION AND SUPPORTS.

The foundation piers to support steel towers are usually made of concrete, consisting of one part portland cement, three parts clean sand and five parts broken stone. They are usually pyramidical in shape and proportioned to suit soil conditions. The allowable bearing pressures on soil will range from 1 to 5 tons per square foot, depending on the quality of the soil. Where the soil is moist or rather loose, a girt should be provided at the base of the tower to prevent spreading of the posts. The allowable bearing pressures for footings should not exceed 400 lbs. per square inch for portland cement concrete and 200 lbs. per square inch for ordinary brick work with portland cement mortar, except when tank is to be rested on building walls, when the bearing plate should be figured on the basis of 125 lbs. per square inch.

The weight of the foundation pier when buried at least two-thirds of its height should be equivalent to the calculated net uplift due to wind pressure with the tank empty, that will be transmitted to it; otherwise it should be 1½ times that amount.

Where the tank structure is above a building, and the building walls are depended upon to act as supports, great care should be taken to determine that the construction is safe against collapse. In many cases, tanks are supported by building walls not originally built to carry them, but where a sprinkler system was later installed it was considered more convenient and cheaper to use the walls than to erect a detached tower for the tank. This has frequently been done without making a thorough inspection first of the condition of the walls, and, largely through ignorance, the necessary care was not taken to distribute the load. Many failures have consequently resulted and there are no doubt numerous cases of this kind where the tanks are apt to fall at any time.

Inspection should be made of the quality and condition of the brick and mortar or other material used in the construction. The wall foundations should be examined as to construction and bearing on soil or rock. The condition of the bond between abutting walls should be noted and a general inspection made for sizable cracks in the walls. The thickness of walls and size and spacing of window and door openings should be measured and calculations then made to determine if the load of tank, water and trestle can be safely distributed over the walls. All unnecessary openings should be bricked or otherwise solidly filled in, and it may be necessary to sacrifice some openings to obtain the required strength. When the walls cannot be altered to support the load, the additional support required can be obtained by carrying steel beams down inside the walls to a solid foundation, provided these do not interfere with occupation of or processes carried on in the building. Otherwise it will be necessary to provide a separate steel tower.

The proper strength of foundations is especially important because of the greater probability of loss of life from the falling of a tank from above a building as compared with the falling of a tank on a detached tower. The monetary loss is liable, of course, to be also much greater, as the water will undoubtedly wreck the building and cause heavy water damage. The building departments of cities endeavor to obtain proper construction, but unfortunately they do not always succeed. The possibility of trouble is increased because of the divided responsibility of the tank builder and the architect. The former seldom concerns himself as to the strength of the supporting walls, assuming that the latter has given the matter proper attention, so he goes ahead and erects the tank according to contract.

GENERAL FEATURES.

Tank fittings should receive careful attention to insure the reliability of the equipment. The discharge or riser pipe is more serviceable if made up of cast or wrought-iron pipe, flanged or coupled, than one made up of bell and spigot pipe, since the latter is apt to leak at the leaded connections, necessitating removal of the frost-proof boxing to permit of repairs. A tank and tower is constantly swaying from side to side and this tends to loosen up leaded joints. Furthermore, the increased rigidity of the flanged and coupled pipe permits the use of a minimum number of tie rods. There are usually four rods connected, one to each post, at girt connections.

The connection of the discharge or riser pipe to wooden tanks has usually been made

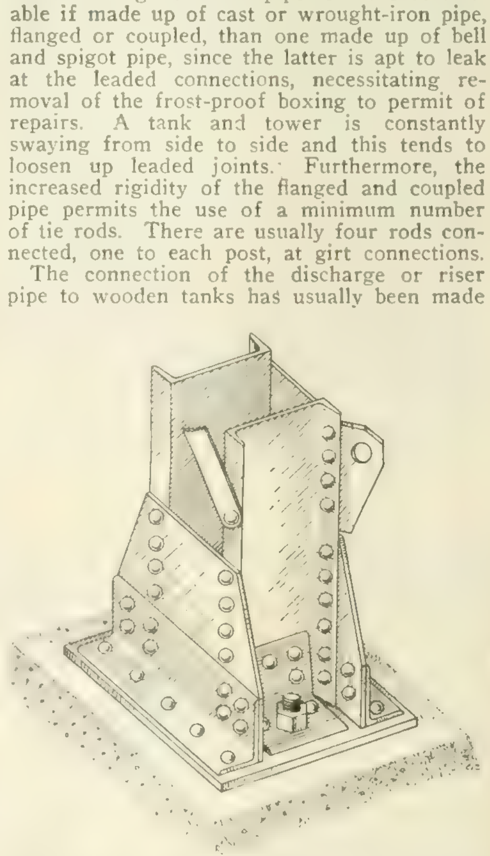


Fig. 6. Typical Construction of Footing for Channel Column.

by extending the pipe through ordinary cast-iron slip flanges bolted to the tank bottom on each side of the opening. The hole in the planks was cut larger than the size of the pipe to form a packing space which was filled when parts were first assembled.

A better construction was used for steel tanks having a stuffing box and gland. Both types of joints were found to be unserviceable, however, the former because the joint could not be tightened when leakage occurred, and the latter principally because iron to iron parts rusted together, which resulted in the breaking of some pipe fitting and the emptying of the tank. Examples of properly designed expansion joints forming tank connections for wooden and steel tanks are shown in Fig. 7. These have a bronze gland and ample clearance between the iron parts to prevent binding by corrosion. The packing space is large and the joint is extended within the tank bottom to form a settling basin, to prevent sediment getting into the yard

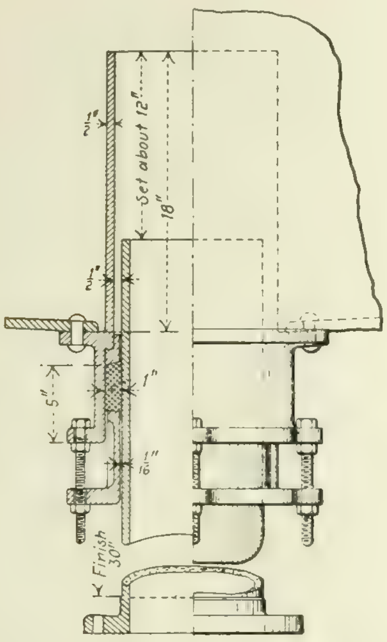


Fig. 7. Example of Proper Design of Expansion Joint for Steel Water Tank.

pipe and clogging the sprinklers at time of fire.

A tightly constructed frost boxing should be placed around the discharge or riser pipe, and arrangements made for keeping the water heated by a hot water heater or a steam coil in the bottom of the tank. Designs of three-ply, two air-space boxings are shown in Fig. 8.

A tank level indicator or telltale is necessary to give a positive indication that the tank is full at all times. After many serious fires it has been learned that the tank had been partially or wholly empty at the start of the fire, and the lack of water had handicapped the fire protection devices. Tanks may be left empty due to neglect, but usually so because of false indication of the telltale. The most used type of device for this purpose is the float in the tank water, operating a target sliding on a scale fixed to the outside of tank. Obviously, these are subject to sticking due to their mechanical construction and exposure to snow and ice in freezing weather. The ordinary pressure gage has been largely used, but cannot be positively depended on, since it is seldom, if ever, tested and the parts stick, causing false readings. There are several types of electrical telltales operated by a float, but these are complicated and easily gotten out of adjustment. Attention is also necessary to maintain the electrical current.

The most reliable telltale is undoubtedly the mercury gage, an adaptation of which for this purpose is shown in Fig. 9. This gage was developed by the laboratories of the Associated Factory Mutual Fire Insurance Companies. It should be placed indoors where it will be observed and cared for. The mercury pot is then piped to the riser pipe on the tank side of the check valve, and the gage board adjusted after filling the mercury pot. The gage is readily tested by opening the pet cock on the water pipe. If water continues to flow under constant pressure, the apparatus is in operative condition; otherwise, the pipe is clogged or there is a valve closed.

The painting of steel tanks and towers and of the iron hoops of wooden tanks is very important to prevent corrosion. Steel plates and shapes should be given the usual priming coat at the shop. The surface of the metal should be thoroughly cleaned of mill scale, rust and grease and perfectly dry before applying the paint. A good paint for the first coat is made by mixing 20 lbs. of red lead and 10 lbs. of zinc oxide with 3 qts. of boiled linseed oil, the red lead and zinc oxide being ground in. This amount of paint will cover about 50 sq. yds. of surface. A second coat should be applied after structure has been erected. For this a more durable oil or asphaltum paint should be used.

The inside of a steel tank should be repainted, usually every two years, or oftener, if the paint shows signs of peeling or wear. The outside of the tank and the tower should be repainted at about five-year intervals. The surface should be carefully cleaned either by sand blast or by steel brushes or scrapers.

The iron hoops of wooden tanks should receive a priming coat after assembly. They should be repainted when necessary. The advisability of painting wooden tanks exposed to the weather is an open question, although a large percentage of the tanks are painted. There is no doubt but that paint protects wood under ordinary conditions, but the objection raised to its use on tanks is on the ground that the tank water percolates through the staves and is prevented from evaporating as it is held under the paint and this is likely to set up dry rot in the wood. It is well known, however, that dry rot does not occur when wood is completely immersed, but rather when it is in a moist condition in the presence of some heat. This objection is not considered well-founded and as a rule the tanks are undoubtedly preserved by painting.

Use of Fire Protection Tanks.—The life of properly constructed equipments depends largely upon the care and attention given to them by property owners. The tanks should be only for fire protection. The practice of using a foot or so of water from the top of the tank for mill purposes is objectionable as the tank collects a larger amount of sediment from the water which is constantly being supplied than it does when used for fire service only. This sediment is likely to settle in the sprinkler pipes and either clog them completely, or, if the sprinklers are open, seriously interfere with their discharge. If water is drawn from the bottom for mill purposes the tank may be empty when needed for fire service. Furthermore, the fluctuation in water level is apt to result in shrinkage of the upper ends of the staves of wooden tanks, causing leakage and hastening corrosion in the steel tank by the repeated wetting and exposure of the sides to the air.

DISCUSSION BY C. S. PILLSBURY.

In discussing Mr. Teague's paper Mr. C. S. Pillsbury of the Chicago Bridge and Iron

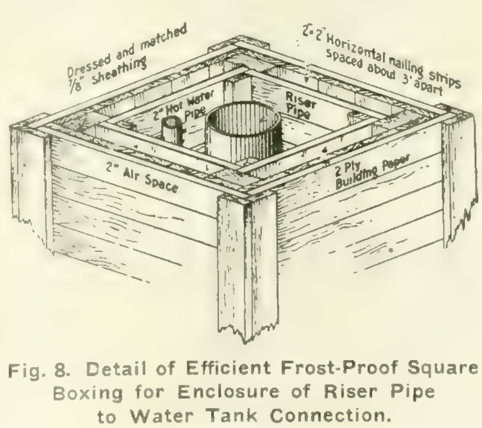


Fig. 8. Detail of Efficient Frost-Proof Square Boxing for Enclosure of Riser Pipe to Water Tank Connection.

Works referred to the practice of his company in the design of tanks and towers. He spoke in part as follows:

It is of great importance to design the details so they will develop the full strength of the main members. To those familiar with railroad bridge shop practice, this statement will seem superfluous, but, as a matter of fact, practically every failure of an elevated tank can be traced to an eccentric top post connection, insufficient provision for the horizontal thrust at the top of the posts, poorly made column splices, or some other oversight due to inexperience. To prevent excessive stresses in the tank shell, the number of posts must be proportioned to the diameter and depth of the tank, and provision must be made to resist the torsional moment due to the curvature of the sides. It is also necessary to provide for a number of severe forces

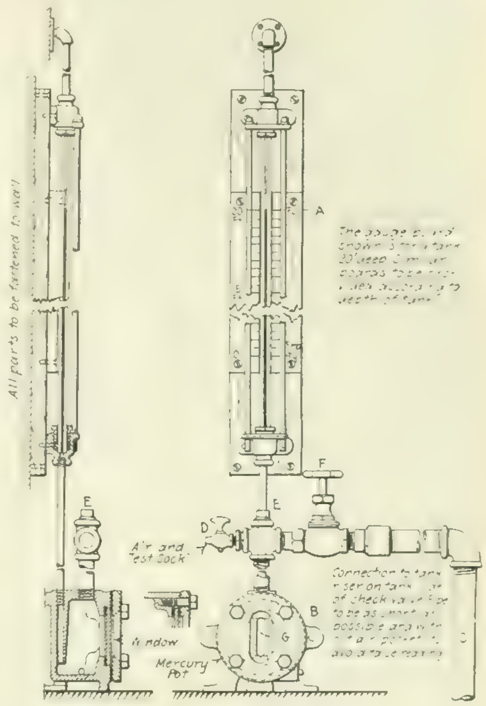


Fig. 9. Details of Mercury Gage for Indicating Water Level in Tank.

other than gravity and wind loads. This last statement is well emphasized by the illustration of a water tower loaded down with ice due to the continuous overflowing of the tank in cold weather. It is only a carefully designed structure that can undergo such treatment without injury.

Following are the stress formulas used by the Chicago Bridge & Iron Works:

Tank Sides and Bottom.—The stress in pounds per linear inch in the sides of a cylindrical tank is $2.6 \times H \times D$, where $D =$ the diameter of the tank in feet and $H =$ the head of water in feet. The maximum stress in a hemispherical bottom is $1.3 \times H \times D$ and in an elliptical bottom $2.3 \times H \times D$. The last two formulas are closely approximate, H and D being the same as before, except that in this case H should be taken as the total depth of the tank.

Posts.—The vertical component of the dead load post stress is equal to the total water and metal load divided by the number of posts. The vertical component of the wind stress equals the following:

3-post tower.................... $\dfrac{M}{0.75\,D}$

4-post tower.................... $\dfrac{M}{1.00\,D}$

6-post tower.................... $\dfrac{M}{150\,D}$

8-post tower.................... $\dfrac{M}{[illegible]}$

where $M =$ moment of wind about panel point at the bottom of the post section considered and $D =$ diagonal of tower at the panel point about which moments are taken.

Rods.—There is no dead load stress in the rods. The wind stress equals the following:

3-post tower............0.500 (V-V') Sec. A
4-post tower............0.707 (V-V') Sec. A

6-post tower ....1.000 (V-V') Sec. A
8-post tower ....1.307 (V-V') Sec. A
where

V' = vertical component of post stress in panel above;
V = vertical component of post stress in same panel as the rod, and
A = angle rod makes with the vertical.

Struts.—Except where the batter of the posts changes there is no dead load stress in the struts. The wind stress is approximately the horizontal component of the rod stress in the panel below.

Maintenance of Hydrants and Gates at Holyoke, Mass.

An important duty of the superintendent of water works is to so maintain the hydrants and valves in the water system under his charge that, when called upon, they will be in condition properly to perform their functions. To insure this condition of preparedness for emergencies the superintendent should, by means of inspections, keep posted on the exact condition at any time of every hydrant and valve in the system. Procedure in this matter varies somewhat as between various water departments. The present article describes the procedure at Holyoke, Mass., and is based upon a paper before the annual meeting of the New England Water Works by Patrick Gear, superintendent of water works at Holyoke.

In the city of Holyoke there is kept a card index of each of the 700 hydrants. This index gives the location, make, date set, size, cost, repairs and date of repairs for each hydrant. No one except a member of the fire department is allowed to operate hydrants. The water department has a man attend all fires, day or night, to see that the hydrants are working all right and to render such assistance as he can should there be any trouble with them. After the fire is over he is required to inspect them and see that they are properly closed and file a report of the same on cards furnished him. The water department does not allow any other city department to operate the hydrant in any way for any purpose. Its employes attend to all the opening of hydrants for street or sewer flushing and for puddling and charges are made only for the water used. Contractors and builders are charged $1 a day for the opening and shutting of hydrants in addition to the cost of the water used. To plumbers, for flushing sewers, etc., the department furnishes a man and hose and charges $1.50 for his time and for the water used besides.

About twice a year all the mains are flushed by opening the hydrants and any trouble experienced in any hydrant is reported and rectified. Besides this the hydrant inspector makes a general inspection of all the hydrants each spring and fall and reports the condition of each as he finds it. For his convenience the city has been divided into four zones. He is furnished with a loose leaf card book.

During the winter special attention is paid to such hydrants as experience has proven are most likely to give trouble. The inspector takes with him a string to which a piece of lead is attached which he drops into the hydrant to find out if there is any ice there. So that the string may not get caught in the barrel of the hydrant, the inspector has an eye turned in a piece of wire. This he puts into the nozzle of the hydrant and the lead drops without any trouble as the string passes through the eye. In ordinary winters there is no trouble, but in severe ones an occasional hydrant may freeze. If the inspector finds ice in the barrel he thaws it out with hot water. If he finds water he pumps it out with a small hand pump he carries for that purpose, and then puts in salt to keep it from freezing. Knowing the depth of each hydrant he opens certain ones which are not deeply set to see if the branch leading from the main is frozen. Should it prove to be frozen the department takes out a 25-HP. steam boiler, connects it with about 25 ft. of hose to a ¾-in. pipe, and puts steam down through the ground to the frozen pipes in holes 2 or 3 ft. apart until the ground is warm enough to thaw out the pipes.

Where hydrants are set in wet ground the employes plug the drip and pump the water out after the hydrant is used. Whether it is good fortune or good care that accounts for it, Holyoke never yet has found a hydrant frozen when opened for a fire. About every five years or so the hydrant barrels are painted a bright red and the top a white metallic color.

Where the operating nut and stuffing box are not brass, the lubricating and packing should be carefully attended to, and an extra man is sent with the inspector to do this as the latter is not able to do it alone. Most of the trouble comes from the hydrants of older makes.

The importance of good gates is very fully realized by all. They control the flow of water to the hydrant, to the meter and to each of the various fixtures which from time to time have to be repaired or replaced. They ought therefore to be of the best material and of the best workmanship.

If it is necessary to shut off water for any reason and the gates are not tight, there is trouble. It is then necessary to go back another block to shut off. If the gate leaks at the stuffing box it is necessary to dig up the street to pack it; if a gate closes so hard that the spindle breaks it must be dug up and repaired. After cleaning out the rust and putting in the spindle the gate is all right and practically as good as new. Mr. Gear suggests that a gate would be longer free from rust and dirt and could be more easily taken apart and cleaned if it was constructed with a brass gland, brass lined stuffing box, top of gate brass lined where the shoulder of the spindle rests, brass bolts and nuts in the stuffing box and slotted for easy removal, the rings in the body set out ½-in. or more and with a space of 1-in. on the sides and 2 ins. on the bottom to keep the dirt away from the gate and rings. A gate so made ought to cost but little more than those at present on the market.

RIVERS AND HARBORS

Stream Gaging By Titration; Comparative Tests of New Chemical and Standard Mechanical Methods of Gaging Stream Flow.

Translated by [illegible] and R. E. Ward, 1118 Empire Building, Pittsburgh, Pa.*

INTRODUCTION.

It is known among hydrographers that the gaging of certain Alpine streams cannot be effectively and properly done by the means at present employed. Indeed it is often impossible to find a cross section suitable for gaging by means of a current meter. Even when a suitable profile is found, the possibility of alluvial deposits changing the cross section remains. Another effect of the alluvial material is the increased friction on the apparatus, thereby changing the constants for a longer or shorter period. This last phenomenon may be observed in the measurement of turbines, using water charged with alluvial material. Some of the current meters have shown, after high water gagings in Alpine streams, a change of constants, due to wearing by sand. Furthermore, the loss of water by infiltration in gravel, etc., cannot be measured by the current meter as we have had the occasion to observe.

The growing importance of the correct gaging of Alpine streams for establishing storage basins has led me to look for some method other than current meter. The result was "The Chemical Method"[1] published in detail by Boucher and Mellet in 1910[2] and mentioned in 1909 by Cote and Bellet[3]; Cote and Bellet measured the discharge of one of the turbines in the plant at Vallorbe. Although having complete confidence in the titration method, this test did not seem conclusive to me because the turbine in question was not tested at the same time by any other method. In other words, the results were not comparative.

The chemical method has also been tried or mentioned by Van Iterson[4], E. Lemaire[5], Ch. E. Stromeyer[6], etc., but never to the author's knowledge have comparative tests been made. At my request, Prof. Mellet was willing to collaborate in the new tests which Mr. Lütschg and I were to undertake at the plant at Ackersand, near Viège.

This is the only plant in Switzerland which has a tailrace equipped with a curtain or screen for measuring the total discharge of turbines according to the method of Prof. Anderson of Stockholm. Moreover, this tailrace, on account of its careful construction, furnishes an excellent opportunity for accurate current meter measurement. The tailrace terminates in a weir. We were therefore able to make comparative tests with the salt solution, the current meter, the curtain, and the weir.

In performing the chemical tests, we chose the simplest case; i. e., the case where we were sure the mixture was perfect. The results presented here are of the first series of tests. At the request of Mr. Boucher, the preliminary results of our study have been sent to the Société Hydro-technique de France, meeting Nov. 28, 1912. The same results were also sent to the Société de Physique de Genève, on Dec. 4, 1912.

To our minds the tests have been made so conclusive that we have decided to use the titration method for gaging Alpine streams, in which the water is always in constant commotion. We have already made gagings by this method, but as we have not yet tried it under all circumstances we deem it wise to defer publication until we can show the limitations of the new method. The various solutions resulting from the tests at Ackersand have been analyzed by Mr. Mellet in his laboratory at the school of chemistry at Lousanne.

GAGING BY CHEMICAL MEANS.

If a constant quantity of concentrated solution of sodium chloride (kitchen salt)[7] be injected into a stream or into the penstock of a turbine, and samples of water be taken after the passage through the turbine, or at a certain distance from the point of injection into the stream, the degree of solution may be used as a means of measuring the discharge.

*"Jaugeages par Titration" par le Dr. Léon W. Collet, le [illegible] R. Mellet et [illegible] Lütschg, ingenieur, "Communications du Service de L'Hydrographie Nationale," Berne.

[1][illegible] T. Schloesing in 1863 in the "Comptes rendus de l'Academie des Sciences" of Paris. (See Houille Blanche, Decembre, 1912, page 325.)

[2]Gaging by titrations by S. Boucher and application of the titration of the chlorides for discharges, by Dr. R. Mellet, "Bulletin technique de la suisse romande," No. 11, 10 Juin Lausanne, 1910.

[3]E. F. Cote et H. Bellet. "La mesure du debit dans les essais de turbines hydrauliques." Grenoble, Gratier & Rey, editeurs. 1909.

[4]Genie Civil. Tome XLIV, No. 26, page 411.

[5]Genie Civil. Tome LVIII, No. 24, page 497.

[6]Proceedings of the Institution of Civil Engineers, Vol. CLX. 1904-1905. Part III.

[7]This salt has the advantage of being cheap and easily obtainable everywhere.

The concentrated solution may be called the initial solution, and the diluted solution, obtained in the samples, the final solution. The discharge may be easily determined by the fact that the ratio of the discharge of the initial solution to the discharge of the final solution is inversely proportional to the ratio of their concentration.

$$\frac{D}{d} = \frac{c}{C}$$

For example; if we pour per second into the stream 0.1-liter of initial solution, containing 300 grs. of salt per liter, it is at the rate of 30 grs. of salt per second. If, after mixing the water of the turbine or stream, we find the final solution contains only 0.3 gr. per liter, we know by the proportion above that the discharge is 1,000 liters per second. To be exact, the discharge sought is the discharge of the final solution diminished by the initial, or 1,000 — 0.1 liters.

Three conditions are necessary for the success of the gaging: (1) A constant rate of flow of the initial solution; (2) perfect mixing; (3) accurate titration of the salt solution.

1. CONSTANT FLOW OF INITIAL SOLUTION.

One condition absolutely necessary for successful applying of the chemical method is that the flow of the salt solution shall be strictly constant and exactly determined. Figure 1 shows a home-made apparatus which insures the inflow of the initial solution at a constant rate. The greater the height h between the jet which discharges the salt solution and the level of the overflow, the greater the certainty of obtaining a constant discharge. As a rule, this difference of height should be at least 1,000 times the allowance of overflow, in order to assure accuracy of 0.1 per cent.

Ordinary kitchen salt contains impurities which, by obstructing the jet, modify the discharge, and thus vitiate the results. It is necessary, therefore, to use a filtered solution, and absolutely clean containers.

The discharge of the jet should be carefully gaged, both before and after the tests by means of carefully calibrated containers. Water or a different salt solution must not be used, because the density and viscosity has a notable influence on the discharge.

At Ackersand we made two tests on the turbines, the first by injecting the salt solution directly into the turbine and the second by injecting it into the tailrace. (See K and L, Fig. 2.) In both cases we were able to keep a constant inflow of the initial solution by means of the overflow of cylinder C (see Fig. 1). In the first test the height h was 3.965 meters, in the second only 0.350 (no more was possible).

It is convenient to choose the discharge jet so that the ratio of the discharge of the initial solution to the approximate discharge of the turbine or the stream shalll be 1 to 10,000[8].

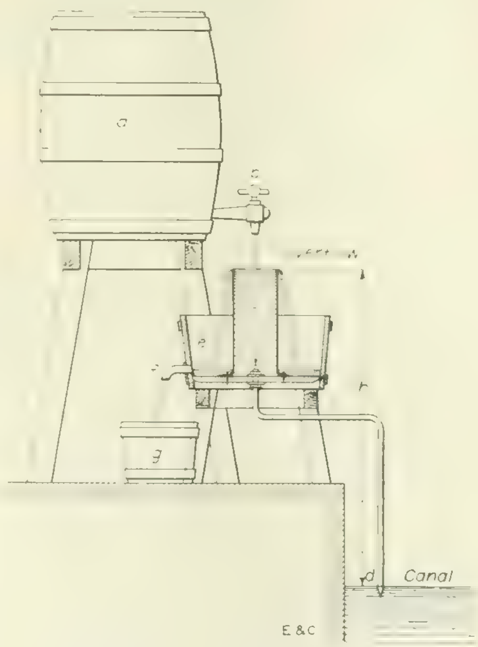


Fig. 1. Home-made Apparatus for Applying Solution in Gaging by Titration.

2. PERFECT MIXTURE.

In a case as shown at K, in Fig. 2, the mixture is perfect. In the tests on the plant at Day, by MM. Boucher and Mellet, sample No. 1 of the final solution at sixth minute gave a discharge of 262.5 liters per second. Sample No. 2 at the ninth minute gave 262.3 liters per second. The difference of about 0.1 per cent indicates well a perfect mixing.

In Alpine streams there is almost certain to be perfect mixture as is easily proven by fluoresceine. As MM. Boucher and Mellet has observed, the chemical method is not applicable to a large stream of slow flowing water. Tests by these authors made with fluoresceine showed that the color flows in veins and streamers. Under such circumstances, no reliable samples can be taken.

3. ACCURATE TITRATION OF THE SALT SOLUTION.

Some years ago the application of volumetric titration of extremely diluted salt solutions was published by us.[9] This method, much simpler than the gravimetric method, nevertheless enables us to obtain results nearly exact. By it the discharge can be determined with an accuracy of about 0.1 per cent.

The following is a description of the titration method employed in our tests:

A. *Preparation of the Reactive Agents.*—(1) Solution of silver nitrate: 1.5 to 2 gr. per litre; (2) solution of sodium chloride: about 1 gr. of salt per litre; (3) solution of potassium chromate: about 5 gr. for 100 cm.^{3} of water.

B. *Dilution and Preliminary Concentration of the Solution for Titration.*—(1) Initial solution: Measure with a burette 5 cm.^{3}; dilute to 500 cm.^{3} in a calibrated flask, and with the aid of a pipette, take out 10 cm.^{3} of this diluted solution for titration. (2) Water taken from the feed water of the turbine or water from the stream to be gaged: Evaporate exactly 1 litre in a porcelain evaporator from 10 to 20 cm. in diameter. Evaporate by means of a water bath, pouring the water in as fast as it evaporates until the litre is reduced to about 10 cm^3. (3) Final solution: Make the evaporation in the same manner as for the water just described. The volume to be taken is also a litre, if care has been taken to make the inflow about 1/10000 of the discharge sought (determined approximately).

C. *Titration of the Three Solutions.*—Make the titration in a porcelain evaporator (for the evaporated solutions use the same evaporator which served for the evaporation). To the small volume (10 cm.^{3}) to be titrated in the evaporator, add two drops of the chromate solution, then allow the solution of silver nitrate to run in slowly by means of the burette.

First there is formed in the liquid colored

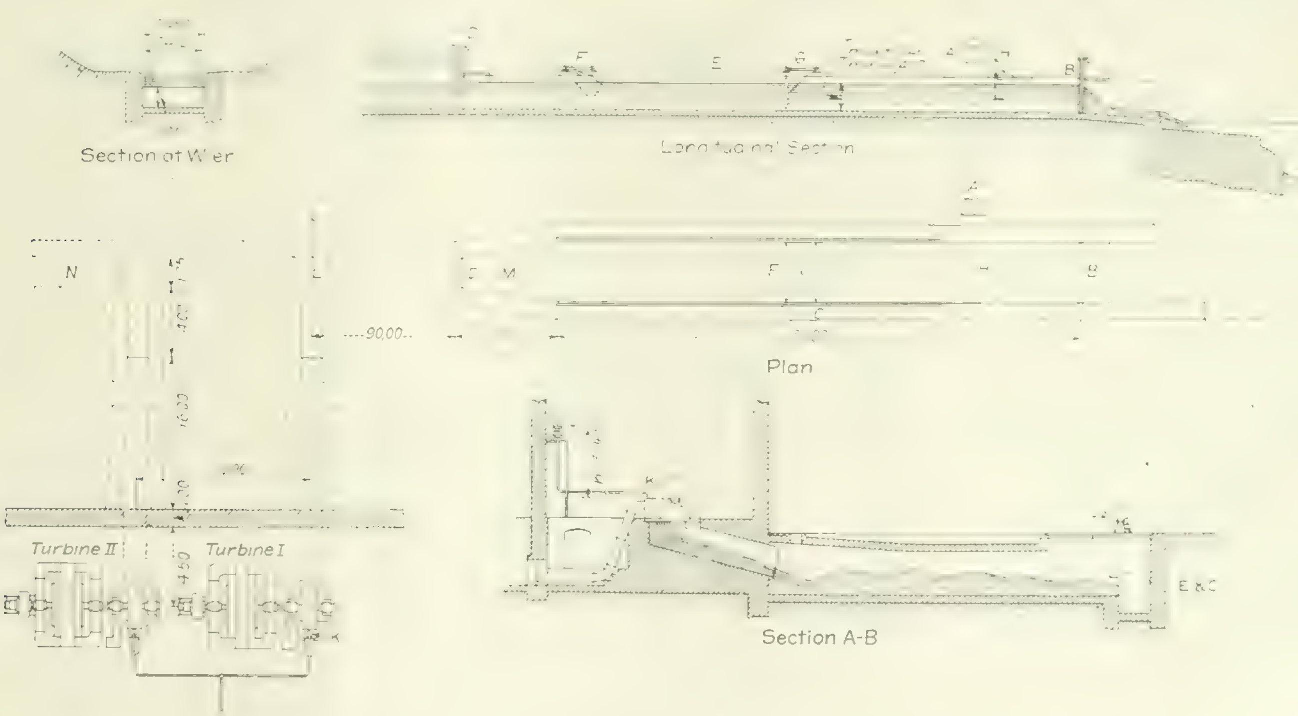


Fig. 2. Plans of Turbine Plant Tested by Titration Method.

[8]If the discharge to be gaged is very large one can work with a ratio much smaller than 1/10,000, but in this case it is necessary to treat the Finale Solution to be titrated accordingly. (See under B, paragraph 3, Final Solution.)

[9]Application of the titration of chlorides for gaging of discharges by [illegible] Mellet. "Bulletin technique de la Suisse romande." 1910. No. 11 (10 juin), page 125.

lemon by the chromate a white precipitate. If the coloration of the mixture does not change after the addition of the 10 cm.³ of silver nitrate solution contained in the burette, add another drop of the chromate. Then continue dropping the silver solution from the burette and so on, adding one drop of chromate after each addition of 10 cm.³ of silver nitrate. The operation is completed as soon as the mixture in the evaporator shows a light orange tint.

In order to note this change accurately, place beside the liquid to be titrated a second evaporation of the same size, containing a telltale liquid. The latter is prepared by mixing equal volumes of the chlorate solution and nitrate solution, and adding just enough chromate solution to give the mixture a yellow color of the same intensity as that of the mixture to be titrated. This tell tale mixture, keeping its lemon color, makes it possible to determine the exact moment when the other mixture changes its tint.

Make note of the number of cm.³ of nitrate solution which is necessary to titrate each of the three solutions. These numbers of cm.³ refer to 1 litre of the second and third solution, but only 1/10000 litre of the first. It is therefore necessary to multiply this last figure by 10,000 in order to obtain comparable results.

D. *Calculation of Discharge.*—Let

N_1 = Number of cm.³ of silver nitrate solution necessary to titrate 1 litre of initial solution;

n = Number of cm.³ of silver nitrate solution necessary to titrate 1 litre of the water of the turbine or from the stream taken before the test;

N_2 = Number of cm.³ of silver nitrate solution necessary to titrate 1 litre of the final solution;

d = The discharge of the initial solution;

D = The discharge of the turbine stream in litre per second;

Then this formula is sufficient:

$$D = d\frac{N_1}{n} - d \text{ liters per second.}$$

DISCHARGE MEASUREMENTS OF TURBINES OF THE [illegible]

(VALAIS).

The gross head is 754 meters and the net head from 675 to 700 meters. The turbines are of the Pelton type of 5,500 HP. and 500 R.P.M. Each machine has its own tail race which discharges into a common discharge canal, running at right angles to the individual races. (See Fig. 2.) Samples of water could be taken only from the outlet of the tunnel of this canal; therefore the salt solution had to be poured into the two turbines simultaneously. Each of the turbines had in its casing an opening, covered by a mud-plug which permitted the salt jet to be introduced directly into the turbine jet.

1. GAGING BY TITRATIONS.

Gaging No. 1, 11 *a. m.*—The initial solution was introduced into the two turbines as follows:

[illegible] litres per second
[illegible] litres per second

d = ...0.114525 litres per second

Salt solution available about 190 litres (about 57 kg. of salt). Time of discharge of salt solution, 20 minutes. The samples of the final solution were taken at the outlet of the subterranean canal at the 7th, 9th, 12th, 15th, [illegible]

Results. 1st. Initial Solution.—Five cm.³ of the salt solution were diluted to 500 cm³. From it 10 cm. were taken for titration; 52.25 cm.³ of this silver solution were necessary for titration.

$$N^1 = 422500 \text{ cm}^3.$$

Results. 2nd. [illegible]—One litre was evaporated and 1.6 cm.³ of the silver solution were necessary for titration.

$$n = 1.6 \text{ cm}^3.$$

Results. 3rd. Final Solution.—One litre of the various samples was evaporated.

Sample 7th min., needed for titration 37.02 cm.³ [illegible]

Sample 9th min., needed for titration 37.43 cm.³ sil. sol.

Sample 12th min., needed for titration 37.39 cm.³ sil. sol.

Sample 15th min., needed for titration, bottle broken in transport.

Sample 20th min., needed for titration 37.23 cm.³ sil. sol.

Sample 25th min., needed for titration 1.66 cm.³ sil. sol.

The sample of the 25th minute (5 minutes after the salt solution ceased) contained scarcely more salt than the normal amount. The 7th, 9th, 12th and 20th show the mixture perfect.

The mean is $N_2 = 37.2675$ cm³.

Calculation of discharge: $D = d\dfrac{N_1}{N_2 - n}$

$$d = 0.114525\frac{422500}{37.2675 - 1.6} - 0.11 = 1356.5.$$

Note. The common tail race of the two turbines extends back, forming a cul-de-sac, or pocket (see *N*, Fig. 2). This pocket is filled with water and contains at least 200 m³. The water is kept in constant motion by the violent discharge of the second turbine. There is, therefore, a constant mixing of a small quantity of the water from this turbine with the water in the pocket. This fact was verified by putting fluoresceine in the second turbine. The green coloring gradually spread throughout the water of the pocket. During the introduction of the salt solution into the turbine, part of this solution is lost in the pocket, while some of the water from the pocket is mixed in the discharge. This change goes on in a regular manner until the final sample solution has the same content of salt. The effect of this process is, however, the same as if the turbines, during the test, had constantly and regularly a greater discharge than the actual. The result $D - 1356.5$ *l/s* is therefore *too large*. However, the samples taken are concordant and show that the mixture was perfect and the method a good one.

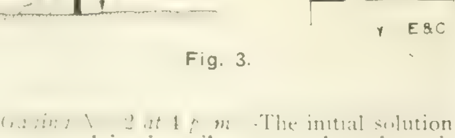


Fig. 3.

Gaging No. 2 at 4 p. m.—The initial solution was poured in the tail race canal at the exit of the discharge of the first turbine (see Fig. 2). The salt solution was not, therefore, affected by the dead water in the cul-de-sac, spoken of in the first test. Discharge d = 0.121183 litres per second.

The salt solution available was about 100 litres (about 30 kg. of kitchen salt). Time of discharge of solution was 7 minutes. Samples of the final solution were taken at the exit of the underground canal (gate lifted) at the 5th, 6th, 7th, 8th, 9th and 12th minutes.

Results. 1st. Initial Solution.—Five cm³ of the salt solution were diluted to 500 cm³ and 10 cm³ taken for titration. Necessary for titration: 43.3 cm³ of silver solution. $N_1 = 433000$ cm³.

Results. 2nd. Feed Water of the Turbines.—One litre was evaporated. Necessary to titrate: 1.6 cm³ silver solution; $n = 1.6$ cm³.

Results, 3rd. Final Solution.—One litre of each sample was evaporated.

Sample 5th min., necessary to titrate 41.6 cm.³ sil. sol.

Sample 6th min., necessary to titrate 40.2 cm.³ sil. sol.

Sample 7th min., necessary to titrate 42.0 cm.³ sil. sol.

Sample 8th min., necessary to titrate 41.8 cm.³ sil. sol.

Sample 9th min., necessary to titrate 31.4 cm.³ sil. sol

Sample 12th min., necessary to titrate 2.4 cm.³ sil. sol.

The sample of the 9th minute (2 minutes after the salt solution ceased) contained less salt than the preceding, which is to be expected. That of the 12th (5 minutes after the cessation of the salt solution) contained scarcely more than normal (also to be expected). The samples for the 5th, 7th and 8th minutes are concordant, therefore the mixture is perfect.

Among the concordant samples that of the 6th does not agree as well as the other three. It contains a little less, which can only be explained by a small increase, sudden and momentary, in the discharge of the turbines. This difference seems sufficient to consider separately two items as follows:

First, the mean discharge during the test, taking account of the four values below.

Second, the exact discharge of the 5th, 7th and 8th minutes during which the discharge was very constant, and abstracting that of the sixth minute. We have, therefore, to consider two values of N_2 as follows: First, total mean of the concordant samples: $N_2 = 41.4$ cm³. Second, mean for the 5th, 7th and 8th minutes: $N_2 = 41.8$ cm³.

CALCULATION OF THE DISCHARGE.

First, mean discharge during the test: $D = d\dfrac{N_1}{N_2 - n} - d = 0.121183\dfrac{433000}{41.4 - 1.6} - 0.12 = 1318.3$; $D = 1318.3$ — litres per second.

Second, exact and constant discharge for the 5th, 7th and 8th minutes: $D = 0.121183\dfrac{433000}{41.8 - 1.6} - 0.12 = 1305.2$; $D = 1305.2$ litres per second.

Remark.—The test is interesting because the case is analogous to an Alpine stream. In fact, a perfect mixture was obtained by pouring the salt solution into the whirlpool caused by the water of the first turbine arriving at right angles in the common canal.

2. GAGING BY CURRENT METER.

For the test a very sensitive instrument was used (No. 740) constructed by Ott, at Kempton. The speeds registered varied between 0.15 m. and 0.28 m. per second. The meter was calibrated before and after the test. For a cross section 3.484 meters long we chose eight ordinates upon which six measurements were taken, making a total of 48. The speeds being very regular, the cross section was divided evenly by the ordinates. Every five minutes the height of the water surface was measured.

We made only a single gaging by the current meter, which lasted 1 hr. and 4 mins., while the gaging by the salt solution was made in the morning in 20 mins., and in the afternoon in 7 mins. On account of the variation of the water level during these tests, we were obliged to proportion the discharge shown by the meter to the water level at the time of the gaging by the salt solution. This was done by means of a discharge curve drawn from data of previous gagings. The results follow: Gaging No. 1: $D = 1317.5$ *l/s*. Gaging No. 2: $D = 1291$ *l/s*.

3. GAGING BY MEANS OF THE CURTAIN.

The hydroelectric plant at Ackersand is the only one in Switzerland possessing an installation for gaging by means of a curtain. The principle is very simple. A curtain with the same section as the tail race is suspended from a carriage running on rails. A pull of 0.8 kg. is sufficient to set the carriage in motion, therefore the resistance offered by the rails is small. The curtain is lowered perpendicularly into the current and is carried away by it. Every meter an electric contact registers on a drum the distance as a function of the time. A simple reading, therefore, gives the mean velocity.

For a detailed description of this device see "Vergleichsversuche mit Flugel und Schirmopparate zur Bestimmung von Wassermengen" by M. Lütschg.[10]

The results obtained by means of the curtain are as follows: Gaging No. 1: $D = 1303$ litres per second. Gaging No. 2: $D = 1299$ litres per second.

4. GAGING BY MEANS OF THE WEIR.

As said before, the tail race has at its end a thin walled weir with end contractions, Fig. 3. This weir, serving as a gate for regulating and

[10]Mitteilungen der Abteilung fur Landeshydrographie. No. 2. Bern. 1913.

emptying the tail race, has two steel rods which product slight eddies and affect the results to a small extent. Nevertheless, to be complete, the results of the gaging are produced below. We have applied the Frese formula[11]:

$$Q = \frac{2}{3} n b h \sqrt{gh}$$

$$n = n_0 \left\{ 1 + \left[0.25 \left(\frac{b}{h} \right)^2 + \zeta \right] \left(\frac{h}{H} \right)^2 \right\}$$

$$n_0 = 0.5755 + \frac{0.017}{h + 0.18} - \frac{0.075}{b + 1.20}$$

[11]Zeitschrift des Vereins Deutscher Ingenieure, 1890. Band XXXIV. Hefte No. 49-52.

$$\zeta = 0.025 + \frac{0.0375}{\left(\frac{h}{H} \right)^2 + 0.02}$$

Applying this formula the following results were obtained:

Gaging No. 1: $D = 1339$ l/s.
Gaging No. 2: $D = 1312$ l/s.

CONCLUSION.

Gaging No. 1—Salt solution poured into the turbine:

	Salt S.	Meter.	Curtain.	Weir.
l/s	1356.5	1317.5	1303	1339

Gaging No. 2—Salt solution poured into the tail race:

	Salt S.	Meter.	Curtain.	Weir.
l/s	1305.2	1291	1299	1312

We have explained before why, in Gaging No. 1, the result by the salt solution is too high. From what precedes we can draw the following conclusions:

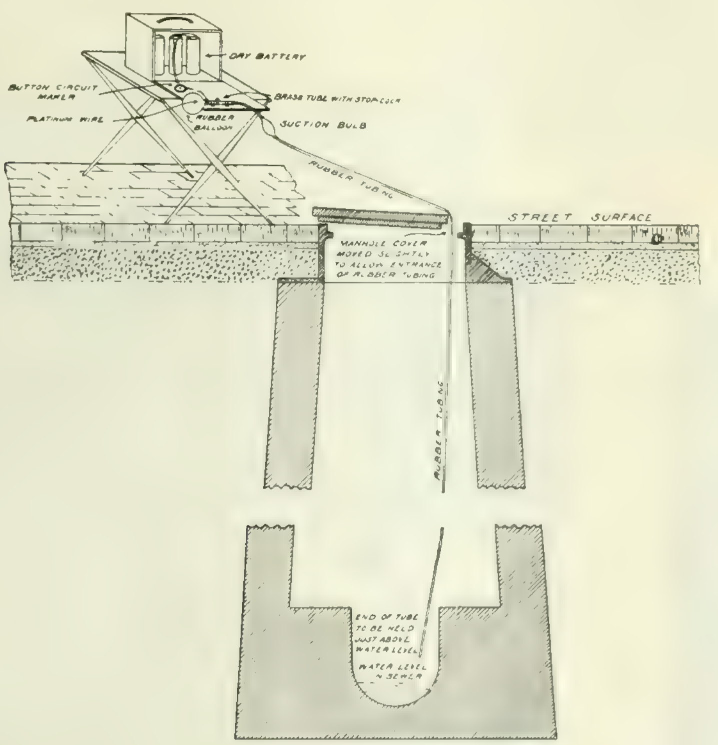
The method of titration, rapid and exact, permits us to determine easily the discharge of high head turbines. It seems applicable in certain cases to test low head turbines. We hope to be able to try this out in the future. Comparative gagings which we have made at Morge (Valais) and on the Salanfe (Rapids of Pissevache, Valais) have proven to us that this method may be used in gaging Alpine streams. We have published nothing about this case yet, because, being more complicated than that of the turbine, it must be studied more fully. We hope in the future, taking everything into account, to be able to fix the limits of this method.

SEWERAGE

Apparatus for Testing Sewer Atmosphere for Explosive Gases.

Laboratory experiments indicate that when gasoline enters a sewer in small amounts it rapidly passes into the vapor state. In such cases there probably would be so little gasoline left in the sewage that it could not be detected there, even in cases where sufficient gasoline had been introduced to form an explosive mixture. In other words, it appears that the thing to test is not the liquid sewage, but the atmosphere just above the sewage. This point was brought out in a study and investigation of explosions in sewers made by the Department of Public Works of New Haven, Conn. The results of this study were made public by Henry J. Kellogg, assistant city engineer, who had charge of the investigation, in a paper before the Connecticut Society of Civil Engineers. The substance of that paper was published in ENGINEERING AND CONTRACTING of Feb. 25, 1914. The information in the present article is taken from the discussion of the original paper by its author, Mr. Kellogg. The apparatus here illustrated and described was designed by Prof. Arthur L. Dean of Yale University, New Haven, Conn.

In testing the sewer air, as above suggested, the investigators first intended to get samples of sewer atmosphere by entering manholes with jars filled with water, then emptying the bottles near the surface of the liquid sewage. The jars would thus be filled with air from below, and, if immediately closed with stoppers fitted with sparking plugs the contents could be tested for explosiveness by a firing battery. But by the time a man could get the jar filled with the suspected atmosphere ventilation would have changed the gaseous condition in that locality in the sewer. If the dangerous gas or gases were still present at the manhole in sufficient quantity to be a menace, after such ventilation, it would be no place to send a man.



CROSS-SECTION OF TYPICAL MANHOLE

Sketch of Apparatus for Testing Sewer Atmosphere for Explosive Gases.

That difficulty has been overcome by the simple, ingenious and apparently practical apparatus designed by Professor Dean and shown in the accompanying sketch. The long rubber tube is let down into a manhole through the small opening by just starting off the cover, after sounding for depth to find the length of tube needed to reach nearly to the surface of the stream. The bulb is worked until both the rubber tube and the bulb are filled with the sewer atmosphere. Then a small, collapsed, rubber toy balloon is slipped over the firing end of the short piece of brass tubing to which the long rubber suction tube is also attached. Projecting from the brass tube into the balloon are the ends of two insulated wires from the dry cell battery, connected by a fine platinum wire. When the balloon is well filled by further pumping, the stop cock in the brass tube is shut, to prevent backfiring, and the circuit is closed by pressing the button. When the platinum wire becomes heated if there is an explosive mixture in the sewer the contents of the balloon are exploded.

The apparatus was tested at the laboratory by holding the receiving end of the long rubber tube over a beaker into which had been poured a few drops of gasoline, and it worked successfully every time. It showed further that the only thing that could be damaged was the little balloon.

The object of extending the tube to the bottom of the manhole is to get the heavier gasoline vapor. It is not necessary to sound for the lighter illuminating gas. The odor of the latter discloses its presence, but the testing apparatus can be used to find out if enough were present to cause an explosion

Sewage Treatment in Germany by Means of the Riensch-Wurl Rotating Screen.

It is generally considered in Europe that the Riensch-Wurl patented rotating screen represents the highest development yet reached in the mechanical treatment of sewage. This screen has quickly established itself in Germany and in nearly all of Europe, and at this time the number of installations of this screen is over three times as great as that of all other types of sewage screen. The Riensch-Wurl rotating screen has now been introduced into the United States, in fact at least two of our largest cities are now experimenting with this type of screen. Accordingly we are here giving a description of the construction and operation of the screen, our information being derived from an article by Dept. Engineer Endris, Engineer with the city of Hamburg, Germany. The present article describes the screen in general and has particular reference to the experimental and later full scale installations at the city of Dresden.

The construction and method of operation of the screen can be plainly seen in Figs. 1 and 2. The device consists of a circular screen provided with slots, the disc being suitably located at an inclined angle of about 10° to 30° in the sewer, completely closing the cross section of the canal. As Fig. 2 shows, the upper part of the disc extends above the water. The profile of the disc always conforms closely to the surfaces of the canals and has the advantage that the surface offered to the water entrance increases in the proportion of the rapidity of the increase in the canal profile, which is not the case in a rectangular pipe. The diameter of the screen is chosen to suit the volume of sewage and ranges from 4 ft. to 26 ft. The capacity of the screen depends upon the rapidity of flow, and the size of the slits, and is between 1½ and 1,200 gals. per second. The effluent of the main sewer is conducted diagonally against the screen. Shortly after the immersion of the rotating screen, it is covered with suspended matter, and during its travel through the water forms a sort of sludge filter, by which particles considerably smaller than the width of the slits are retained. On its exit from the water the screen is covered with solid substances of all kinds, as shown in Fig. 2. The solid substances are cleaned off above the surface of the water by a set of rotating brushes mounted on a spider, which revolve around the spider shaft and also on their own axes. In consequence of a special construction the bristles of the brushes are not pressed against the screen plate with too much force, nor is the dirt squeezed through the slits, but the screen plate is gone over lightly and every spot is brushed several times. The brushes carry the sludge into a circular trough, from which it is further transported by buckets, belt conveyor, tilting carts, or other device.

In Fig. 1 the separator screen consists of two firmly connected parts, a smooth ring part, which lies diagonally in the sewage stream, and a hat-shaped frustum of a cone. The screen cone is cleaned by a special brush, the circular screen surface being in turn cleaned by revolving brushes.

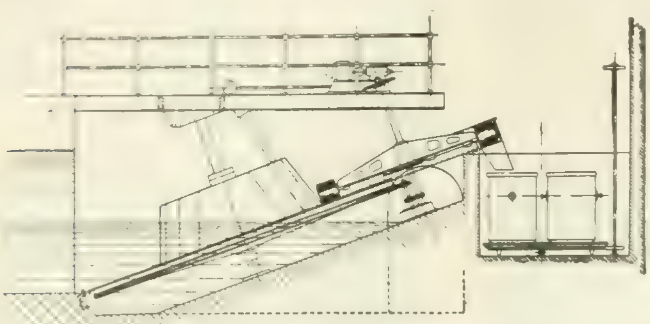


Fig. 1. Diagramatic Outline of the Elements of the Riensch-Wurl Screen.

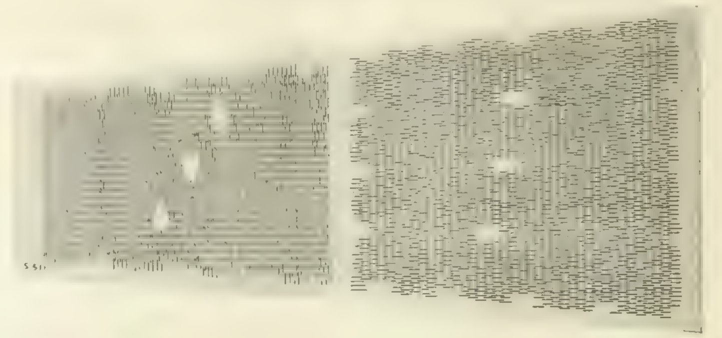


Fig. 3. View of Typical Screening Plates of the Riensch-Wurl Rotating Screen.

DETAILS OF CONSTRUCTION.

The Screen Plates.—The usefulness of the screen depends particularly upon the construction of the screen plates which separate the solids from the fluids. The screen plate illustrated in Fig. 3 is used in the Riensch-Wurl screen. Plates of this type are made of bronze or brass containing slits of cone shape, widened out on the under side and about 1/32 in. to 7/32 in. wide. The screen plates are so arranged on the iron framework as to cause the least possible loss of head in the water.

The screen surface, being made of large plates, insures a smooth and uniform surface without any projections on which sludge might possibly accumulate, while the close arrangement of the perforations provides the most advantageous possible use of the screen for the passage of water.

The Frame Work.—The frame work, shown in Fig. 1, is remarkably well arranged and is braced rigidly on all sides. All the load carried in a single one of the arms of the screen is cared for by a structural iron framework and is borne up by the main shaft itself. In the smaller and the medium sizes, up to 16 ft. in diameter, the framework is secured to the carrying shaft and is hung to a ball-bearing support which is supported by the service bridge.

The Ball Bearing Support.—This support is constructed as a pivot over the surface of the water. In the larger types, however, 16 ft. in diameter and over, the shaft is stationary and the framework of the plates rotates in a ball-bearing support arranged around the shaft and likewise above the water. In this way all the weight is taken up by the supports outside of the surface of the sewage and is easily cared for. In the sewage water itself, except when the water level is high, a steady bearing is provided for the lower part of the main shaft. This consists of a roller bearing equipped with stuffing boxes.

Thorough lubrication, together with the slow rotatory movement (0.3 to 0.2 rotations per minute) and the small power consumed insures freedom from trouble.

The Brushes.—The manner in which the brushes are attached to a central brush body with radiating arms makes the cleaning of the discs particularly effectual. While in all other screen arrangements the screen surface is cleaned only once at each rotation the distribution of the brushes in the Riensch-Wurl system permits the cleansing of the screen plates as frequently as desired. Normally the screen plates are cleaned from four to five times each rotation in such a manner that a clean brush passes over the previously brushed surface, as shown in Fig. 4. The brushes describe intersecting paths on the plates so that no single spot is left untouched by them.

In order to insure uniformity in the pressure of the brushes they are suspended movably from the carrying arm as shown in Fig. 5. The middle shaft of the brush body is arranged eccentrically to the shaft of the carrying arm and is dragged along with it. By means of this special construction the brushes do not press against the screen by the pressure of the driving gear, but glide along easily over the screen plates while the brushes rotate on their own axes. Counterweights within the brush body permit the regulation of the pressure of the brushes on the screen plate so that all parts of it may

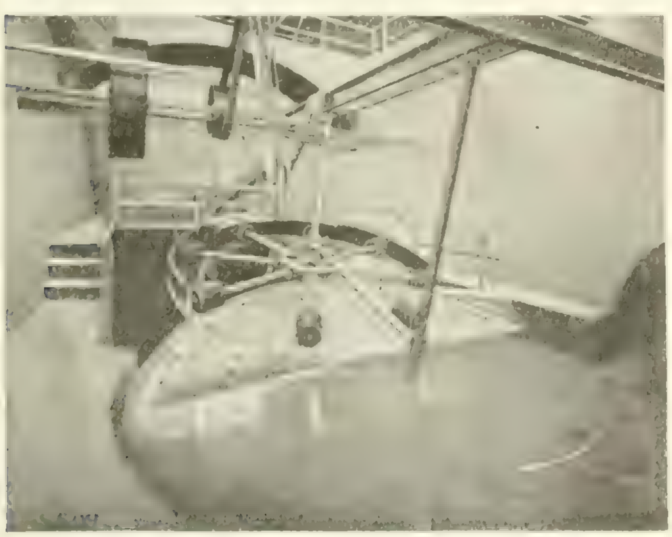


Fig. 2. View of the Experimental Riensch-Wurl Screening Plant Installed at Dresden, Germany.

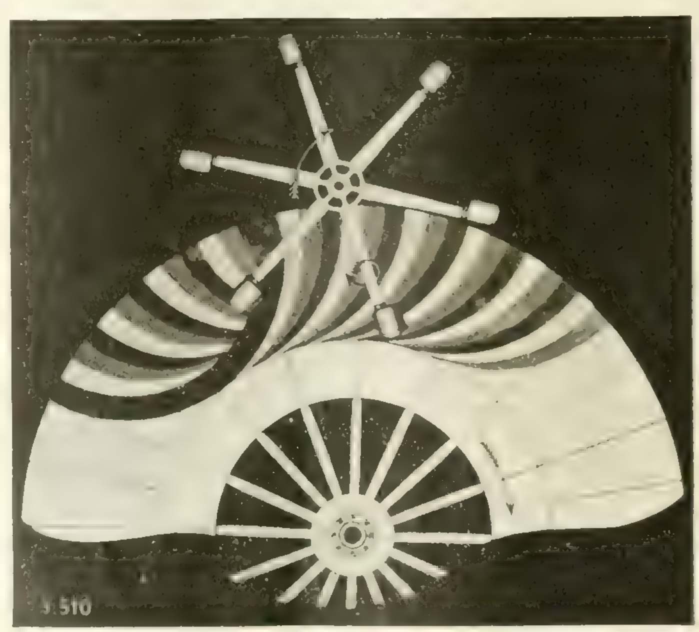


Fig. 4. Diagram Showing Brush Cleaning Sphere Development of the Riensch-Wurl Screen.

be uniformly and thoroughly cleaned. This device is the result of practical experience.

The Conical Brush—This brush which cleans the raised cone attached to the ring of the screen is attached to the service post and is adjustable. In the larger types there are two or more brushes which may be connected or disconnected as desired, according to the height of the water. Thus the single conical

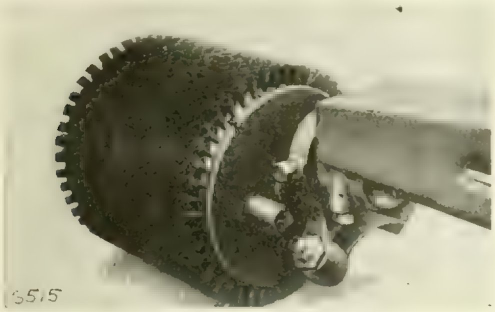


Fig. 5. Design of Riensch-Wurl Brush Suspension.

brush may be readily dispensed with in dry weather, at a saving of the mechanism and of the brush as well. The brushes as they rotate push the sludge in front of them. The eccentric attachment of the brushes and their upward pressure precludes the possibility of the openings being clogged.

The Driving Power of the Disc.—The driving power may be supplied by belt or by electricity, according to local conditions. All parts which require attention are visible at all times and are arranged on a platform which extends over the screen and contains machinery is in motion. In the smaller types the transmission wheel for the screen frame is made of a simple cast iron gear and has been found very satisfactory, the tooth consisting of easily interchangeable segments. For the larger types a gear wheel is built up of hardened steel pins. The clearance between the screen and the adjoining edge of the conduit must be very exact, so that the space between the discs and the stationary rim corresponds exactly to the width of the openings in the screen. For this purpose a wrought iron ring is secured to the masonry and to it various segments are attached. These may be adjusted exactly to the circumference of the screens.

THE ADVANTAGES CLAIMED FOR THE RIENSCH-WURL SCREEN.

The screens move noiselessly, and the action is particularly neat, as there is no splashing or dripping onto the floor of the screen room. The motion is a uniform rotatory one, and vibratory movements are entirely avoided.

The machinery is at all times visible and accessible to the attendant and is easily controlled by him. There are no nooks and corners in which sediment may decompose, and the carrying off of the screenings is easily supervised.

All rotating parts are above water and may be easily reached while the machinery is in motion. Only one single guide bearing may occasionally be under the level of the sewage, but if constructed as indicated above this presents no difficulty and has proven entirely satisfactory.

The cleansing process takes place above water. Its peculiar mechanism affords frequent and thorough cleansing of the screen.

The seal, as already described, is most precise and can be regulated to the minutest degree.

is required by the Riensch-Wurl screen system. The same surface is therefore obtained at a smaller cost.

The high efficiency of the Riensch-Wurl separator discs is especially noteworthy. These are now made in sizes, having capacities ranging from 2½ to 1,200 gals. per disc. The working capacity of the disc is essentially greater than has hitherto been generally supposed.

The low cost and the small amount of power used are additional meritorious features of the system. Compared with other systems, especially the wing-screens, a particular advantage presented by the Riensch-Wurl system is the small amount of power consumed. In actual use, the Riensch-Wurl screens of Bremen (14 ft. diameter) used 0.5 HP. In Dirschau the cost of 18 hours' constant service for one month was only $2.30, so that the Riensch-Wurl screens required only 0.2 HP. Every one of the large Riensch-Wurl screens of Dresden (26 ft. diameter) running at full speed used only 2½ HP.

Finally, special emphasis must be laid upon the minimum amount of loss of head in the stream flow that the Riensch-Wurl screen effects, a natural consequence of the large screen surface. In most cities this is a matter of much importance, since the fall of the principal intercepting sewers is usually only a slight one and must be used to the best advantage.

QUALITY OF THE RESIDUE LEFT BY THE RIENSCH-WURL SCREEN.

The quality of the residue and the ease with which it can be handled is one of the most important features of the Riensch-Wurl screen. While the residue obtained from basin or tank plants and the like, contains a high percentage of water and presents an offensive and not easily manipulated fluid, the constant brushing of the screen plates, in the system here under consideration causes the residue to lose much of its water content. This sludge is low in water content and is innocuous and easily transported. Unlike the basin sludge, which is impregnated with alkaline salts, the products of the screen retain complete manurial value and are readily disposed of to farmers. They are easily composted and take up little room, doing away with the necessity of large sewage fields with their costly care and the unpleasant accompaniment of foul odors and troublesome flies. The

Fig. 6. Interior View of the Sewage Treatment Plant at Dresden, Comprising Four Riensch-Wurl Screens Each 26 ft. in Diameter.

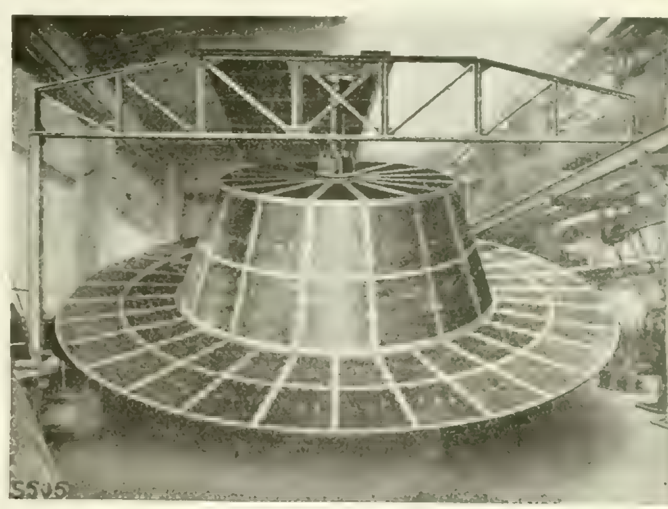


Fig. 7. View of One of the Riensch-Wurl Separator Screens, Built for Dresden, while on Erecting Frame at the Factory.

the lubricating apparatus as well as the principal parts of the machinery.

Lubrication.—Particular stress is laid on proper lubrication. All rapidly moving bearings are provided with ring lubrication and all ball bearings are continually kept running in oil. Central lubrication with grease is provided for the slowly moving parts, and all parts requiring lubrication have special oil cups attached which may be refilled while the

The large available screen surface is one of the greatest advantages of the system over other systems. A maximum of 4/5 of the entire filtration surface is available for the flow of the water, while in band-screens or drum-screens at most only 1/3, and in wing screens or the like only 1/5, of the entire screen surface is available. In other words, for similar results other systems require three to five times as much area of screen surface as

sludge absorbs no moisture from the air or from rains. The sewage plants of Dirschau, for example and of Christiana, in Norway, are situated in the midst of populated areas without giving rise to any of these undesirable features. The sewage purification plant of Stettin, partially constructed by Dr. Endris, is likewise situated in the midst of dwelling houses along the piers with the pleasure boats, which ply between the watering places of the

North Sea. The residue can be easily carted away in a cleanly manner by street carts, in the same way as for the necessary street catch basins, and at a low cost. For instance, suppose a city has 300 cubic meters of dry material to dispose of. By this screening method this separated solid matter would show an average water content of 63 per cent, and therefore there would be only 1,130 cu. yds. of residue to be disposed of, which besides obviating the cost of transportation would yield a small sum of money. On the other hand, if this solid material were separated by a basin plant, its 95 per cent of water would necessitate disposing of 8,000 cu. yds. of sludge instead of 1,130 cu. yds. and moreover being saturated with water after every rainfall would require long periods of time for drying.

These figures adequately demonstrate the superiority of the Riensch-Wurl Screen over a basin construction in all large plants, while any additional plants subsequently required may be made smaller and at a considerably less expense.

The volume of the wet sludge, according to the water content should be considered. Thus, sludge with 95 per cent water spreads over twice as much space as does that with a 90 per cent water content, and eight times as much as of a 60 per cent water content.

THE DRESDEN PLANTS.

The first large plant of this type was installed as a test plant at Dresden. The experimental results were so favorable that the city council decided to equip the sewage filtration plant of the city with Riensch-Wurl screens. The experimented screen, see Fig. 2, had a diameter of 14 ft. and reduced 100 gals. of sewage per second. The perforations for the flow of the water were 1/16x13/16 in. In 1909 this disc was equipped with independent brushes and found definite use in the abattoir yards of the city of Dresden.

The treatment plant at Dresden is particularly interesting on account of its large size. As seen in Fig. 6 this plant consists of four screens, each 26 ft. in diameter. They care for a maximum flow of 4,500 gals of sewage per second, an average of 1,120 gals. per second for each screen. The screens are made of perforated bronze plate, the perforations being 1/16 in. wide, and are so arranged as to control a water level of 8 ft. All the sewage of Dresden flows through this installation, without further treatment, into the Elbe. It is the largest sewage clarification plant in the world. The first two screens were placed in July, 1909, and the remaining two in July and September of 1911. During the drought of the summer of 1911 the flow of the Elbe for a period of three months was only 1,765 cu. ft. per second. The filtration fully met the requirements of the situation.

FIELDS OF APPLICATION.

The screen here described finds a place as a complete filter for sewage in all cities with favorable river flow. The Dresden plant above described is a good example of this class.

A second important field of application of this screening system is as a pre-cleanser in basin and tank installations. This form of mechanical pre-filtration of the greater part of the sludge considerably reduces the cost of the total treatment.

These screens are also sometimes installed to protect sewage pumps.

In the smaller sizes the Riensch-Wurl screen also serves for the clarification of the wastes of factories and mills, as well as for reclaiming fibres and other waste. It is well adapted for wool scouring plants, cloth factories, spinning mills, tanneries, canning factories, and for the purification of river water for large condensers in central power plants.

Mr. William L. D'Olier, 503 Morris Bldg., Philadelphia, Pa., has acquired the patent rights for this screen in America.

ROADS AND STREETS

Recent Specifications and Standards of the Maine Highway Commission.

(Staff Article.)

An act of the Maine legislature passed in 1913 provided for a highway commission of three members appointed by the Governor, and a chief engineer appointed by the commission. The chief engineer is the executive officer of the commission and employs such assistants as are necessary to supervise road work in the state. A system of state roads (built wholly by the state from a bond issue) and state aid roads (built jointly by the state and municipalities) is to be laid out, constructed and maintained by the commission. A sliding scale for state aid appropriations is provided, varying from \$2 appropriated by the state for each dollar contributed by a town having a valuation of \$200,000 or less, to \$0.75 for towns having a valuation in excess of \$1,600,000.

For the construction of state roads serial 41-year bonds, the interest and sinking fund for which are provided for by the automobile license fees, may be issued at a rate not to exceed \$500,000 per year, to the amount of \$2,000,000. An additional sum of \$300,000 yearly is appropriated by the state for construction, administration and maintenance purposes. The numerous details of organization and financing are well worked out.

In the Current News Section of ENGINEERING AND CONTRACTING for April 22 will be found a description of work proposed and under contract and a map of 125 miles of road, plans for which have been prepared by the commission. The specifications and standards of construction for this work, while conservative and following well established practice, present a number of interesting features.

Specifications.—The Notice to Contractor, Proposal, Contract, Specifications and Bond are bound in the order named in a booklet 8½ ins. by 10 ins. in size. The notice contains an itemized estimate of quantities and the time honored "rights to reject any and all proposals" is modified by "whenever in the opinion of the Commission good cause exists therefor." A unit price form of proposal is used.

Excavation is classified as earth, rock and borrow and all surfacing materials are measured in place after rolling, gravel being paid for by the cubic yard and macadam by the square yard. Metal and vitrified culvert pipe, inlet gratings and bituminous material are furnished by the commission, the contractor doing the unloading and hauling. The contract is specific and rather short. In excavation stumps are removed to a depth of 18 ins. and rock to 12 ins. below finished grade.

TABLE I. PLAIN CONCRETE RETAINING WALLS.

Table of Dimensions and Quantities.

Height, H	Base, B	Footing, F	D	Top, W	Cu. yds. per lin. ft. of wall.	Cu. yds. lin. ft. of footing.	Total cu. yds. per lin. ft.
4	1'- 8"	2'- 8"	1'-00'	12"	0.197	0.098	0.295
5	2'-00"	3'-00"	1'- 2"	12"	0.277	0.125	0.402
6	2'- 4"	3'- 4"	1'- 3"	12"	0.370	0.154	0.524
7	2'- 9"	3'- 9"	1'- 4"	12"	0.486	0.185	0.671
8	3'- 3"	4'- 3"	1'- 6"	12"	0.629	0.236	0.865
9	3'- 8"	4'- 9"	1'- 8"	12"	0.777	0.288	1.065
10	4'-00"	5'-00"	1'- 9"	12"	0.926	0.324	1.250
11	4'- 4"	5'- 4"	1'-10"	18"	1.188	0.362	1.550
12	4'- 9"	5'- 9"	2'-00"	18"	1.388	0.426	1.814
13	5'- 3"	6'- 3"	2'- 1"	18"	1.625	0.482	2.107
14	5'- 8"	6'- 8"	2'- 2"	18"	1.858	0.535	2.393
15	6'-00"	7'-00"	2'- 3"	18"	2.083	0.583	2.666

Fig. 1. Standard Cross Sections for Gravel [illegible] in Maine.

Vegetable matter is removed for fills less than 18 ins. deep. Borrow is measured in excavation. The free haul for excavation is 2,000 ft. with ½-cent. allowance beyond this distance. Stone base of large stones have outlets 3 ft. wide into side ditches at intervals of approximately 200 ft. 'V' drains have similar outlets. These outlets in the case of gravel drains are protected with 1 sq. yd. of sod. Earth shoulders are separately defined and are to be of selected material. Stones 6 ins. to 9 ins. in size bedded in 6 ins. of sand or gravel are used in cobble gutters. Guard rail is given two coats of white paint. Joints in side drains are wrapped with a 4-in. strip of burlap. Gravel surfacing is to contain from 15 to 25 per cent of binding material and is laid in two or three courses. Gravel must not be dumped on the roadbed. Piles of 5 cu. yds. are left on the roadside at 500 ft. intervals for maintenance purposes.

Bituminous binder is paid by the gallon in place for penetration work. The hardness, toughness and per cent of wear of stone is specified. Macadam base is constructed of 2 to 3½ in. stone and second course of 1 to 2¼ in. stone, circular gage measure. Binder is applied at 250°-300° F. for tar products and 300°-350° F. for fluxed native and oil asphalt products. Application is made at the rate of 1½ gals. per square yard, using pouring pots or a pressure distributer. A seal coat of from ½ to ¾ gals. per square yard is used. The specifications as a whole are well written in unusually good language.

Cross Sections.—In Fig. 1 are shown standard cross sections for various types of roads. Two-course gravel road is laid in courses of equal thickness. In three-course road the first two courses have a compacted thickness of 3 ins. and the third course 2 ins. Stones larger than 1½ ins. in diameter are not permitted in the top course. For bituminous macadam the bottom course is 3½ ins. thick after compacting. This course is filled with sand or screenings, the top ½ in. being left open. The second course is 3 ins. thick, loose measure, and is left open and porous.

TABLE II.—REINFORCED CONCRETE RETAINING WALLS.

Table of dimensions and quantities.

H	T	A	B	W	C	D	E	Cu. yds. per lin. ft.	Bars U square. Size.	Length.	Spac.	Bars V square. Size.	Length.	Spac.	Bars X square. Size.	Length.	Spac.	Bars Y square. Size.	Length.	Spac.	Bars Z square. Size.	Length.	Spac.	Approximate Wt. of Steel per lin. ft.
6'	1'-3"	3'-8"	2'-3"	9"	6"	6"	12"	0.371	½"	1'-10"	12"	½"	6'-9"	12"	½"	1'-6"	12"	½"	3'-6"	12"	½"	*	12"	21 lbs.
8'	1'-5"	4'-9"	2'-9"	9"	6"	8"	12"	0.521	½"	2'-00"	10"	½"	8'-9"	10"	½"	2'-3"	10"	½"	4'-6"	12"	½"	*	12"	28 lbs.
10'	1'-7"	6'-00"	3'-3"	9"	6"	10"	12"	0.690	½"	2'-3"	8"	½"	11'-3"	8"	½"	3'-3"	8"	½	5'-9"	12"	½"	*	12"	39 lbs.
12'	1'-9"	7'-3"	3'-9"	9"	12"	12"	12"	0.880	¾"	2'-8"	5½"	¾"	13'-3"	5½"	½"	3'-6"	5½"	⅝"	7'-00"	8"	½"	*	12"	92 lbs.
14'	1'-11"	8'-6"	4'-3"	9"	15"	14"	15"	1.125	¾"	3'-3"	4"	¾"	15'-3"	4"	¾"	4'-3"	4"	¾"	8'-3"	8"	½"	*	12"	144 lbs.
16'	2'-1"	9'-9"	4'-9"	9"	18"	16"	16"	1.418	1"	3'-6"	6"	1"	17'-6"	6"	1"	5'-00"	6"	¾"	9'-6"	6"	½"	*	12"	200 lbs.

*Length of bars Z depends on length of wall.

Note.—The reinforced cantilever type of wall should be considered only for heights from 12' to 18'.

For heights up to 12 feet vertical reinforcing bars (V) should extend entire length of wall.

For heights over 12 feet, ⅓ of bars should extend entire height, ⅓ of bars ⅔ of height, and ⅓ of bars ⅓ of height.

The above table is for twisted square bars. Other deformed bars of same sectional area may be used.

STANDARD STRUCTURES.

Blue prints of standard details and structures are bound in a convenient size booklet. The outside dimensions of the pages are 11 ins. by 16¾ ins., the space within the border lines being 10½ ins. by 15¼ ins. in dimensions.

Retaining Walls.—In Fig. 2a is shown a type of gravity retaining wall and Table I gives the quantities and dimension for walls of various heights. Figure 2b illustrates a cantilever type of wall, data for this design being given in Table II.

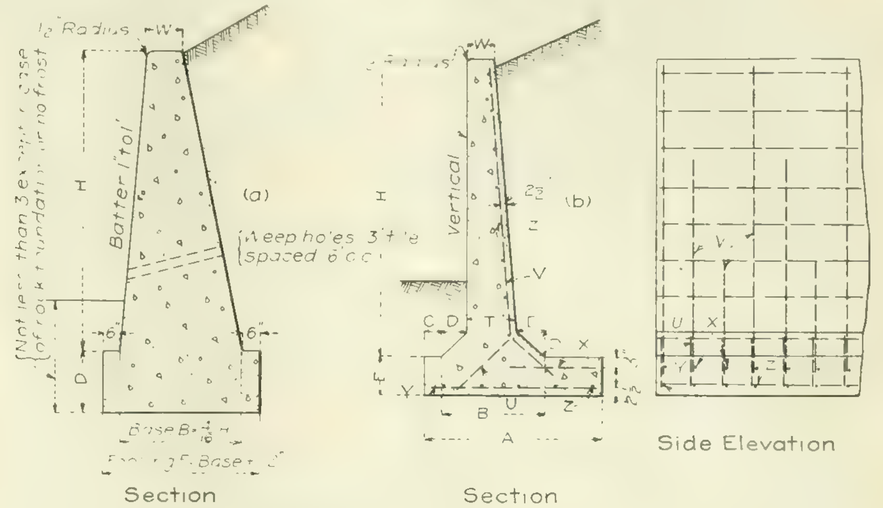


Fig. 2. Standard Retaining Walls Used by Maine Highway Commission.

..*Pipe Culverts.*—An interesting feature of the details for pipe culverts is the provision of a table showing the length of pipe needed with various rates of fall from end to end of culvert for different depths of fill. This tabulation, given in Table III, obviates the necessity of computing lengths and is a convenience. Fig. 3 shows a typical pipe culvert layout, the dimensions of the headwalls for various size pipe being given in Table IV.

Concrete Culverts.—Three mixtures of concrete are used: 1:2:4 for reinforced concrete and masonry measuring 9 ins. or less in thickness, ¾ in. stone being the maximum size permitted in the aggregate; 1:2½:5 for abutments and wing walls, stones being limited to 1½ ins. in size; and 1:3:6 for footings and cut-off walls, stones up to 3½ ins. being permitted. Figure 4 shows a standard design for a 5 ft. by 4 ft. box culvert and Fig. 5 shows details for culverts of other sizes. Table V gives dimensions and quantities for the culverts illustrated in Figs. 4 and 5.

The standard forms of guard rail in use are shown in Fig. 6, together with the details of a drop inlet used in town suburbs and on side hill roads.

SPECIFICATIONS.

An abstract of the general specifications of the highway commission is given. The specifications are brief and clearly written.

material shall be furnished and placed so as to make the surface conform to the required sub-grade. All soft clay and spongy material which will not consolidate under the roller shall be removed to a proper depth and paid for as excavation. The space thus made shall be filled with suitable material.

No allowance will be made for excavation beyond or below the width, lines and grades shown on the plans or called for by the specifications. All connections with driveways and branch roads shall be excavated to the lines and grades given by the engineer. The contractor shall remove all obstructions such as walls, fences and buildings within the limits of the work. All manholes, catch basins, water gates and similar structures which are below grade or project above grade shall be raised or lowered to fit the finished grade of the road. Except as hereinafter provided, the item of excavation shall include all such removals and alterations.

Rock Excavation.

Rock excavation shall include hard ledge rock or boulders of more than ½ cu. yd. volume. Boulders of ½ cu. yd. or less volume and soft and disintegrated rock which can be removed with pick or shovel will be classified and paid for as earth excavation.

Rock excavation shall be made to the lines indicated on the typical rock cross-section showing the side slopes, said slopes to be one-quarter to one; and 12 ins. below the lines indicated on the cross-sections showing the finished surface.

Rock excavation for underdrains, pipe culverts, and masonry will be made six inches in width or depth beyond the outside of the pipe or masonry, except that a minimum width allowed for small pipes shall be eighteen inches. Allowance for rock in gutters will be made on the basis of the width of gutter and 12 ins. in depth below the proposed surface, allowance for rock in catch basins to be made on the basis of 6 ins. outside of the outer walls.

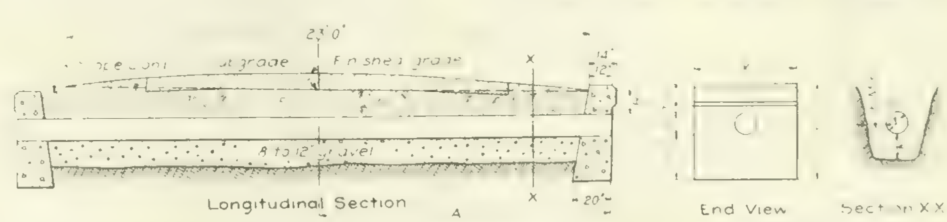


Fig. 3. Pipe Culvert Details Showing a Method of Placing.

Excavation.

Excavation shall include the grading of the roadway, including shoulders, ditches and side slopes the entire length of the work, to conform to the lines and grades shown on the plans. All trees, stumps, brush, roots and sods within the lines of the improvement shall be grubbed and removed. All stumps shall be excavated to a depth of at least 18 inches below finished grade. Wherever such excavation shall have been made, a sufficient amount of suitable

Borrow.

When the material excavated inside the location, or right of way, suitable for filling, is not sufficient in quantity to properly form the subgrade of the road, including shoulders, such necessary additional material shall be obtained and placed by the contractor from borrow pits or other sources approved by the engineer.

All measurement of earth work shall be made in excavation. Material obtained from excavation within the limits of the location or slopes or in grading side roads, and driveways and used in embankments, or for all other purposes, shall be paid for as excavation, and all material obtained outside the location

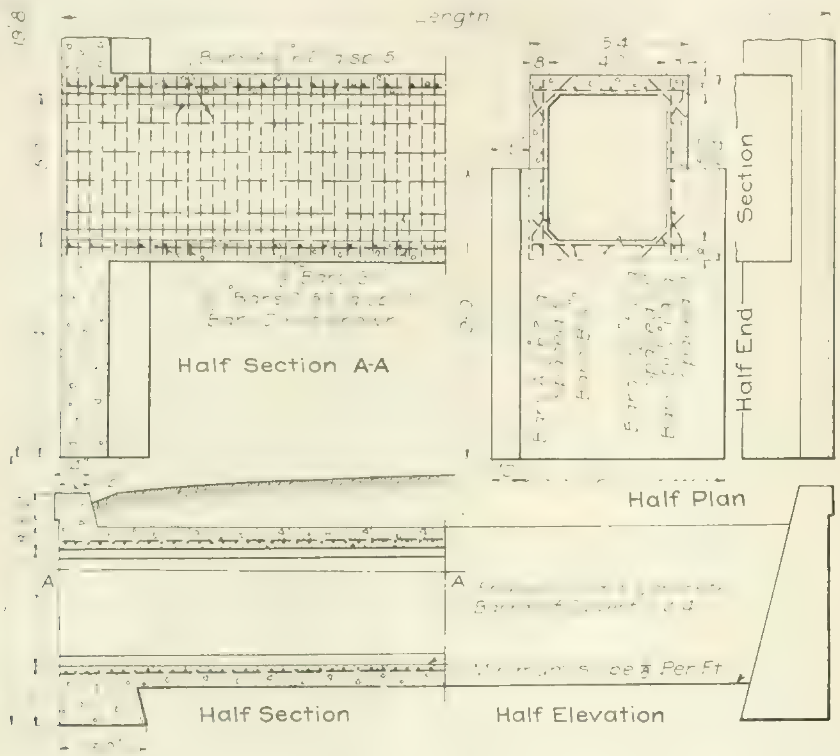


Fig. 4. Type of Concrete Box Culvert Used by the Maine Highway Commission.

shall be measured in excavation and paid for as borrow and not as embankment. All surfaces and slopes shall be left in a neatly trimmed condition.

Overhaul.

If the haul on any material required for embankment exceeds 2,000 ft. it shall be classified as overhaul and payment of ½ cent per cubic yard for each 100 linear feet in excess of 2000 feet shall be made on the material so hauled.

Filling or Embankment.

Embankment shall be formed of suitable material. Stone shall not be used nearer to the bottom course, or surface of shoulders, than 6 ins., except as specified. Where the filling is less than 18 ins. in depth all vegetable matter shall be removed from the original surface. Embankments shall be [illegible] successive horizontal layers not exceeding 12 ins. in thickness. Each layer shall extend across the entire fill and shall be thoroughly rolled or compacted. All surfaces in slopes [illegible]

be left neat and even according to the lines, grades and directions given by the engineer. Such excavated material as may be fit for the purpose and as may be necessary shall be used to fill those parts of the roadway which are below grade or which have become so by the removal of rock or other unsuitable material, in a manner herein provided and the item of earth excavation is to include the proper placing of such excavated material as filling in embankment, and the removal from the work of all such as is not utilized. All surplus excavation or waste material shall be used to widen embankments, flatten side slopes, or be deposited in such places as the engineer may direct.

Preparation of Subgrade.

The subgrade shall be properly shaped, rolled and compacted so that it conforms to the lines, grades and cross-sections shown on plan before any surfacing material is applied. The rolling shall be done with a roller weighing not less than 10 tons. All depressions which develop during the rolling shall be filled with suitable material and the subgrade shall again be rolled until no depressions develop.

Stone Base.

Wherever a stone base is called for by the plans and specifications or during the progress of the work it is considered necessary and is so ordered by the engineer, the contractor shall furnish and place stones, not exceeding 12 ins. in diameter, on the subgrade to a width as shown on plans and in such a manner as to produce a surface parallel with the finished surface of the road, using the larger stones in the center and smaller stones on the sides. All stones shall be roughly placed by hand, the larger chinked with the smaller and the remaining voids filled with coarse gravel. The roller shall then be run over the surface until they are firmly bedded. The thickness when finished shall be as shown on plans.

Wherever a stone base is used, side outlets of stone shall be built three feet wide and of a depth and length which will drain the stone base into a side ditch or culvert. Such outlets shall also be built through the shoulders at intervals of approximately 200 ft. where ordered by the engineer. The average size of stone should be about eight inches. Payment shall be made per cubic yard of stone and gravel furnished and compacted in place in accordance with cross-section plans.

TABLE IV. DIMENSIONS AND QUANTITIES FOR PIPE CULVERT END WALLS.

Diam. pipe, ins.	Dimension. X	Y	Concrete in one end, cu. yds.
12	4'-6"	4'- 3"	0.935
14	5'-3"	4'- 5"	1.127
16	5'-9"	4'- 7"	1.271
18	6'-6"	4'- 9"	1.481
20	7'-0"	4'-11"	1.651
24	8'-3"	5'- 3"	2.043

Stone 'V' Drain.

Wherever a stone 'V' drain shall be called for by the plans and specifications or during the progress of the work it is considered necessary and is so ordered by the engineer, the contractor will prepare the bed, furnish all stone and gravel and build the same according to plans and specifications. Wherever practicable the surface of the old road may first be plowed or otherwise loosened, the loosened material moved toward the shoulders of the road with a grader and if it is necessary in order to obtain the required depth, a second plowing may follow and the grader again be used. Stones not exceeding 12 ins. in diameter shall then be placed on the bottom of the excavation so made, the larger stones under the center of the road with sizes diminishing towards the shoulders. The roller shall be run over the stones forming the 'V' drain until they are thoroughly bedded in place. The top of the 'V' drain shall be finished off with smaller stones or coarse gravel until the surface shall become parallel with the finished grade. Before surface material is applied suitable outlets shall be built through the shoulders at low points and at intervals of approximately 200 ft. where ordered by the engineer. These outlets shall be of stone and gravel three feet wide and of sufficient depth and length to thoroughly drain the bottom of the 'V' drain. Payment shall be made per cubic yard of stone and gravel furnished and compacted in place in accordance with cross-section plans.

TABLE III. SHOWING CULVERT LENGTHS FOR VARIOUS DEPTHS OF FILLS AND SLOPES OF WATER COURSE (23 FT. ROADWAY)

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Fill 4.0' A = 18'-6"	Fill 4.5' A = 19'-6"	Fill 5.0' A = 20'-6"	Fill 5.5' A = 21'-6"	Fill 6.0' A = 22'-6"	Fill 6.5' A = 23'-6"	Fill 7.0' A = 24'-6"	Fill 7.5' A = 25'-6"	Fill 8.0' A = 26'-6"	Fill 8.5' A = 27'-6"	Fill 9.0' A = 28'-6"	Fill 9.5' A = 29'-6"	Fill 10.0' A = 30'-6"
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	34	36	38	40	42	44	46	48	50	52	54	56	58	60	62
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	32	[illegible]	38	40	42	44	46	48	50	52	54	56	60	62	64	66
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	40	[illegible]	44	[illegible]	48	50	52	56	58	60	62	64	66	68
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	46	48	50	54	56	58	60	62	66	68	70	72
1"	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	44	[illegible]	[illegible]	50	54	56	58	60	64	66	68	..	..	.
1¼"	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	50	54	56	58	62	64	68	70	72	..	..	.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	50	[illegible]	58	60	62	66	68	72	74	76		.	..
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	62	64	66	70	72	[illegible]	78	80			
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]					.	.		.	.	.	..	.	
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]						.		.	.		..	..	..
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]											..	.	.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	66													

Note.—For fills from 4' to 8.5' culvert lengths have been figured for slopes from ¼" to 2" per foot for water courses, and for fills from 9' to 10', ¼" per foot to 1" per foot.

Explanation: To use the above table it is necessary to know the difference in elevation between subgrade and top of pipe at inlet, and slope of water course.

The [illegible] of fill may be taken from the table [illegible]

A roadway 23' [illegible] Minimum length of culvert, 36', minimum fill, 1', minimum slope, ¼" per ft.

Gravel "V" Drain.

Wherever stones suitable for building "V" drain are not available a drain may be built of coarse, clean gravel and land tile pipe. In such cases the excavation shall be made the same as for a stone "V" drain excepting that in the bottom of the "V" a suitable bed of gravel 4 ins. deep shall be made upon which to lay 4-in. pipe and after the pipe has been placed and the joints wrapped with burlap 4 ins. wide, gravel having a cross-section as shown on plan shall be placed. No stone whose diameter exceeds one (1) inch shall be allowed within four inches of the pipe. Side outlets 3 ft. wide and of sufficient depth to drain the lowest point in the "V" drain shall be made through the shoulder of the road similar to the "V" drain, at intervals of approximately 200 ft. where ordered by the engineer. Each outlet shall be protected by about one square yard of sod placed as directed by the engineer. The surface of the "V" drain shall be wet and rolled until it has become settled and permanently in place before the surfacing material is applied and shall be parallel with the finished grade. Payments will be made per cubic yard of stone and gravel furnished and compacted in place, including 4-in. land tile in place in accordance with cross-sections shown on plans.

Gravel Base.

Gravel base shall be built to conform with plans, profiles and cross-sections. Gravel shall be furnished by the contractor and shall be of a quality satisfactory to the engineer. In general it shall consist of hard, sound, durable stone of various sizes ranging from pea stone to a maximum size of 4 ins. The gravel shall not contain more than 25 per cent of binding material. The gravel shall be spread in one course, care being taken that the various sizes are assorted and that no pockets of stone or binding material occur. This course of gravel shall be rolled while wet, using a sprinkler if necessary, until it becomes thoroughly compacted and shall then have a thickness of 4 ins. Payments for gravel shall be made per cubic yard in place and compacted on the road. No allowance will be made for gravel driven into the subgrade by rolling.

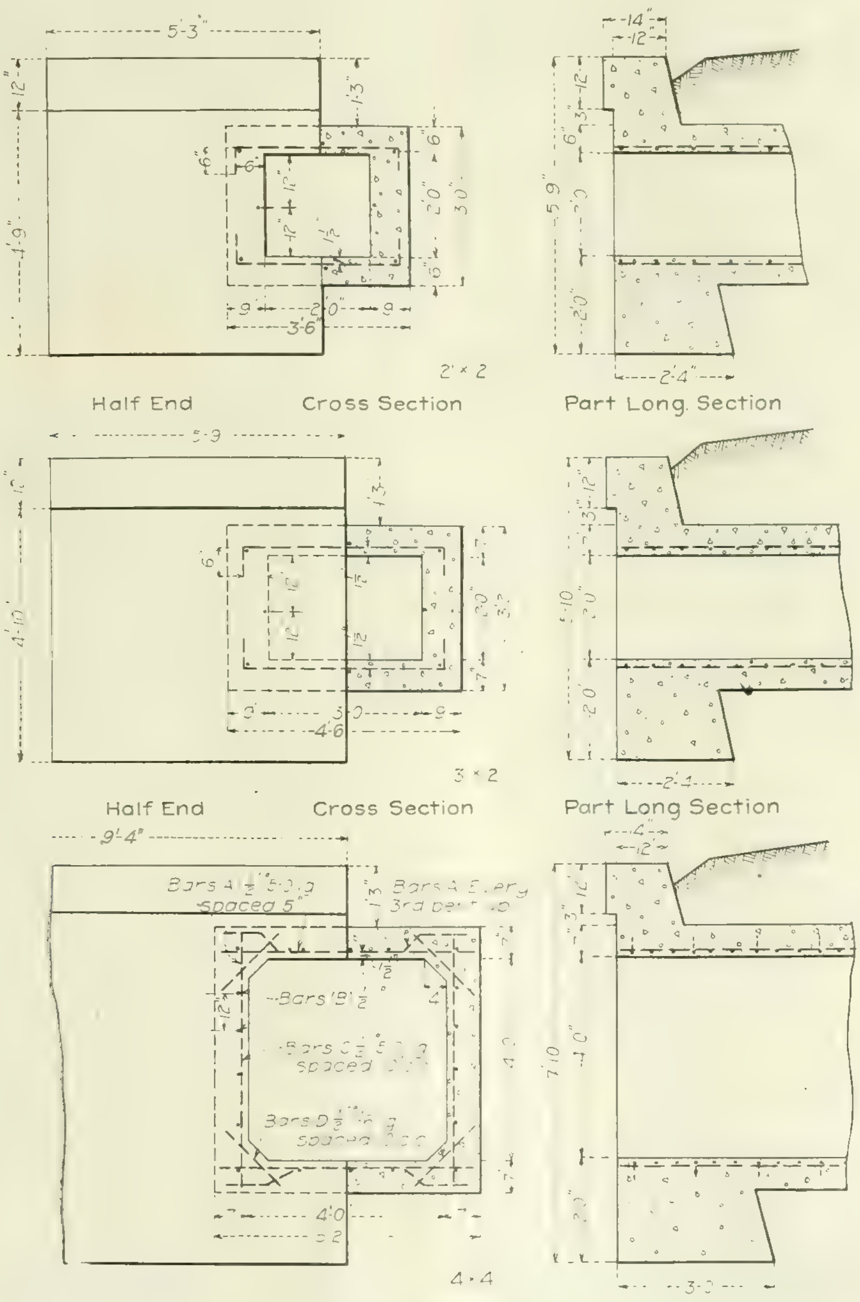


Fig. 5. Details of Standard Box Culverts.

Concrete Masonry.

Cement concrete masonry culverts and end walls for metal culverts shall be built in accordance with plans and specifications for concrete. The price to be paid for cement concrete masonry shall include the excavation, back filling, all necessary materials, centers and forms and all work on the same; no allowance will be made for coffer-dams, pumping or bailing or for any materials or labor necessary on account of water. All concrete shall be measured in accordance with the dimensions shown on plans.

The concrete shall be of the character as indicated on the plans or as provided for in these specifications and shall be built by the contractor in accordance therewith.

Cement.—The cement shall be some standard brand of Portland Cement and must meet the requirements of the standard specifications adopted by the American Society for Testing Materials.

Sand.—The sand shall consist of dry, clean, sharp quartz grains and shall not contain more than 5 per cent of clay, loan, or other foreign materials. The grains shall be well graded and of such size that all will pass a ¼-in. mesh screen, and not more than 20 per cent will pass a No. 50 sieve.

Coarse Aggregate.—The coarse aggregate may consist of either broken stone or gravel. Stone shall be sound, hard and tough and broken to sizes hereinafter specified and when used shall be free from foreign material. No weathered or disintegrated material shall be used. Gravel used for concrete shall consist of clean, hard and sound stones, pebbles and sand, and shall be well graded in size between the limits specified below.

Classes A, B, and C.—Unless otherwise specially provided, there shall be three classes of concrete known as Class A, Class B, and Class C. Class A concrete shall consist (by volume) of 1 part of cement, 2 parts of sand, 4 parts of coarse aggregate, and clean water. The stones of the coarse aggregate may vary in their longest dimensions from ¼ in. to ¾ in. Class B concrete shall consist (by volume) of 1 part of cement, 2½ parts of sand, 5 parts of coarse aggregate and clean water. The stones of the coarse aggregate may vary in their longest dimensions from ¼ in. to 1½ ins. Class C concrete shall consist (by volume) of 1 part of cement, 3 parts of sand, 6 parts of coarse aggregate and clean water. The stones of the coarse aggregate may vary in their longest dimensions from ¼ of an inch to 3½ ins.

Mixing.—The cement and sand shall first be thoroughly mixed dry in the proportions specified, on a proper mixing platform. Sufficient clean water shall then be mixed to produce a pasty mortar. To the mortar thus prepared shall be added the proper proportion of coarse aggregate previously drenched with water, and the whole shall be mixed until every particle of the coarse aggregate is thoroughly coated with mortar. Instead of the above method, a mechanical mixer of approved type may be employed.

Size of Batch.—Concrete shall be mixed in batches of such size that the entire batch may be placed in the forms by the force employed within 45 minutes from the time that the first water is applied. No concrete is to be prepared from mortar which has taken an initial set and would require retempering.

Placing.—All concrete shall be carefully deposited in place and never allowed to fall from a height greater than five feet. Concrete shall never be deposited in running water, and when still water it shall be carefully lowered into place by means of a chute or some other approved method.

As fast as concrete is put into place, it shall be thoroughly tamped in layers not more than 6 ins. thick, and the portion next to the forms shall be troweled by using a spade or by other means to bring the mortar into thorough contact with the forms.

Concrete shall not be deposited when the temperature of any of the materials or of the ahmosphere is below 35° F., and if during the progress of the work freezing temperature threatens or is predicted by the United States Weather Bureau, proper precautions shall be taken to protect from freezing all concrete laid within the four preceding days.

Forms.—Forms shall be so constructed as to continue rigidly in place during and after depositing and tamping the concrete. If during the placing of the concrete the forms show signs of bulging or sagging at any point, that portion of the concrete causing the distortion shall be immediately removed and the forms

TABLE V.—QUANTITIES AND DIMENSIONS FOR VARIOUS SIZES OF CONCRETE BOX CULVERTS.

Size	Length of culvert	Total cu. yds.	No. bbls. cement.	No. cu. yds. sand.	No. cu. yds. stone.	No. bars A — ½" squares, 4' long, spaced 8".	No. bars B — ½" squares, 8 required.	Weight of steel.
2 ft. by 2 ft.	26	12.54	17.98	5.63	11.27	78	25'-9"	440
	28	13.02	18.74	5.84	11.70	84	27'-9"	475
	30	13.50	19.49	6.05	12.12	90	29'-9"	508
	32	13.98	20.24	6.26	12.54	96	31'-9"	542
	34	14.46	20.99	6.47	12.96	102	33'-9"	576
	36	14.94	21.74	6.68	13.38	108	35'-9"	611
	38	15.42	22.49	6.89	13.80	114	37'-9"	644
	40	15.90	23.24	7.10	14.22	120	39'-9"	678
	42	16.38	23.99	7.31	14.64	126	41'-9"	712
	44	16.96	24.74	7.52	15.06	132	43'-9"	746
	46	17.34	25.49	7.73	15.48	138	45'-9"	781
	48	17.82	26.24	7.94	15.90	144	47'-9"	814
	50	18.30	27.99	8.15	16.32	150	49'-9"	848
3 ft. by 2 ft.	26	14.71	21.26	6.61	13.22	*106	†25'-9"	626
	28	15.32	22.22	6.88	13.76	114	27'-9"	673
	30	15.93	23.18	7.15	14.30	122	29'-9"	721
	32	16.54	24.14	7.42	14.84	130	31'-9"	768
	34	17.15	25.10	7.69	15.38	138	33'-9"	816
	36	17.76	26.06	7.96	15.92	146	35'-9"	864
	38	18.37	27.02	8.23	16.46	154	37'-9"	911
	40	18.98	27.98	8.50	17.00	162	39'-9"	939
	42	19.59	28.94	8.77	17.54	170	41'-9"	1,006
	44	20.20	29.90	9.04	18.08	178	43'-9"	1,054
	46	20.81	30.86	9.31	18.62	186	45'-9"	1,102
	48	21.42	31.82	9.58	19.16	194	47'-9"	1,149
	50	22.03	32.78	9.85	19.70	206	49'-9"	1,197

*½ in. sq., 5 ft. long, spaced 6 ins.
†½ in. sq., 8 required.

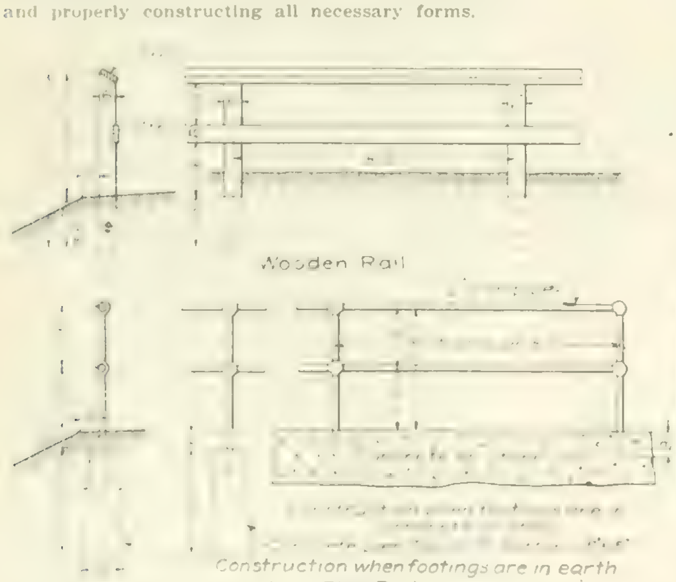
Size	Length.	Total cu. yds.	No. barrels cement.	Cu. yds. sand.	Cu. yds. stone.	No. bars A — ½" squares, 5' long, spaced 5".	No. bars B — ½" squares, 16 required.	No. bars C, ½" squares, 5' long, spaced 10".	No. bars D — ½" squares, 1'-6" long, spaced 10".	Total weight of steel
4 ft. by 4 ft.	26	28.36	39.67	12.83	25.67	126	25'-9"	64	128	1,321
	28	29.16	40.92	13.18	26.37	136	27'-9"	68	136	1,418
	30	29.96	42.17	13.53	27.07	146	29'-9"	74	148	1,528
	32	30.76	43.42	13.88	27.77	156	31'-9"	70	156	1,625
	34	31.56	44.67	14.23	28.47	164	33'-9"	82	164	1,714
	36	32.36	45.92	14.58	29.17	174	35'-9"	88	176	1,824
	38	33.16	47.17	14.93	29.87	184	37'-9"	94	184	1,930
	40	33.96	48.42	15.28	30.57	194	39'-9"	98	196	2,031
	42	34.76	49.67	15.63	31.27	204	41'-9"	104	208	2,142
	44	35.56	50.92	15.98	31.97	212	43'-9"	108	216	2,239
	46	36.36	52.17	16.33	32.67	222	45'-9"	112	224	2,327
	48	37.16	53.42	16.68	33.37	232	47'-9"	116	232	2,424
	50	37.96	54.67	17.03	34.07	242	49'-9"	122	244	2,534
5 ft. by 4 ft.	26	31.57	44.61	14.25	28.50	[1]126	[2]25'-9.	[3]64	[4]128	1,525
	28	32.59	46.21	14.70	29.40	136	27'-9"	68	136	1,650
	30	33.61	47.81	15.15	30.30	146	29'-9"	74	148	1,757
	32	34.63	49.41	15.60	31.20	156	31'-9"	78	156	1,863
	34	35.65	51.01	16.05	32.10	164	33'-9"	82	164	1,979
	36	36.67	52.61	16.50	33.00	174	35'-9"	88	176	2,106
	38	37.69	54.21	16.95	33.90	184	37'-9"	94	184	2,227
	40	38.71	55.81	17.40	34.80	194	39'-9"	98	196	2,346
	42	39.73	57.41	17.85	35.70	204	41'-9"	104	208	2,473
	44	40.75	59.01	18.30	36.60	212	43'-9"	108	216	2,575
	46	41.72	60.61	18.75	37.50	222	45'-9"	112	224	2,688
	48	42.79	62.21	19.20	38.40	232	47'-9"	116	232	2,799
	50	43.81	63.81	19.65	39.30	242	49'-9"	122	244	2,927

[1]6'-2" long, spaced 5 ins.
[2]18 required.
[3]5'-2" long, spaced 10 ins.
[4]1'-9" long, spaced 10 ins.

properly supported before continuing the work. The amount of concrete to be removed shall be determined by the engineer, and the contractor shall receive no extra compensation on account of the extra work thus occasioned. Forms for exposed surfaces shall be constructed of dressed lumber.

All forms shall be left in place not less than 36 hours and all supporting forms not less than 10 days after the concrete has been deposited. These periods may be increased at the discretion of the engineer in charge.

It is understood that all prices for concrete masonry shall include furnishing all materials and properly constructing all necessary forms.



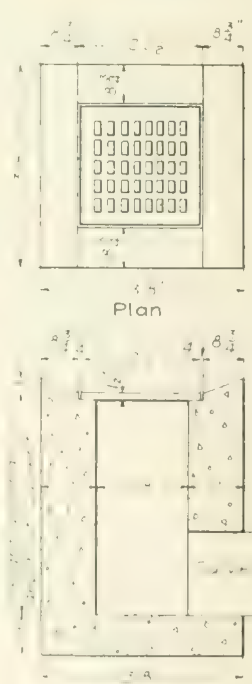


Fig. [illegible] Details of Guard Rail and a Drop Inlet.

Joints.—When the work of placing concrete is to be interrupted for a period greater than 1 hour and there are no reinforcing rods projecting, provision for a joint shall be made in the following manner: Square timbers 8 ins. by 8 ins., or some other suitable size approved by the engineer, shall be bedded in the concrete throughout the length of the course for one-half their thickness and allowed to remain until the concrete has taken its initial set. When the work of laying concrete is resumed, the timbers shall be removed and the surface thoroughly wet. No joints will be permitted in reinforced concrete beams, and in floor slabs the joints shall be vertical and parallel to the main reinforcing bars.

Finish.—Forms covering surfaces of the concrete masonry which are to be exposed shall be removed immediately after the expiration of the period of time necssary for such forms to remain in place, as fixed by the engineer, and all crevices which may appear shall be filled with 1:2 cement mortar. These surfaces shall then be finished with 1:2 cement mortar and a wooden float be used to present a smooth, neat appearance.

Reinforced Concrete.—All reinforced arches, beams, floors, parapets, guard rails, and all concrete masonry measuring less than 9 ins. in thickness shall be made of Class A concrete, unless otherwise specified on the drawings or directed by the engineer in writing.

Abutments and Wing Walls.—Unless otherwise specified on the drawings or in writing by the engineer, Class B concrete shall be used for all abutments and wing walls the thickness of which is not less than 9 ins.

Footings and Cut-Off Walls.—Class C concrete shall be used for all footings and cut-off walls, unless otherwise specified on the plans or directed in writing by the engineer.

Steel for Reinforced Concrete.—Unless otherwise specified on the drawings, all reinforcing steel shall consist of bars which have been deformed in some approved manner. No plain bars will be permitted except as shown on the drawings or directed in writing by the engineer.

The steel bars shall have the net sectional area and be placed in the exact positions indicated on the drawings.

Unless otherwise specified on the drawings or in writing by the engineer, all reinforcing bars shall be of medium steel having an elastic limit of not less than 35,000 pounds per square inch, and shall be sufficiently malleable to withstand bending cold with a radius equal to twice the diameter or thickness of the bar through 180° without fracture.

When placed in the concrete, the reinforcing steel shall be free from grease, dirt, and rust, and it shall be the duty of the contractor to provide means for properly cleaning the steel.

Thorough contact of the concrete with every portion of the surface of the steel shall be obtained.

Splicing Reinforcing Bars.—Unless otherwise specified on the drawings or in writing by the engineer, necessary splices in reinforcing bars shall be effected by overlapping the ends of the bars a distance equal to forty times their thickness or diameter.

Cement Stone Masonry.

Cement stone masonry shall be built in accordance with the plans and specifications. The price to be paid for this item shall include excavation, except rock, back filling, and all work on the same. No allowance will be made for coffer dams, pumping or bailing or for any materials or labor necessary on account of water.

Cement stone masonry shall be built where shown on plans. It shall be laid up in cement mortar composed of three parts of clean, sharp, well-graded sand and one part of Portland cement, the quantities to be determined by measure. No mortar shall be allowed to be used after it has begun to set.

A sufficient number of headers shall be introduced into the construction to firmly and securely tie and bond the entire structure together. Fully one-fourth of the stones shall be headers, and each stone shall be bedded so that it will not rock but will be firm and solid. The stone used for the footing course shall be at least 12 ins. in thickness. All exposed faces shall be raked out to a depth of 2 ins. and neatly pointed with mortar containing equal parts of Portland cement and clean, sharp sand.

Culverts.

Metal pipe and vitrified pipe for culverts will be furnished by the Commission f. o. b. cars at the nearest railroad station. No demurrage will be paid by the Commission. They shall be laid in a bed of gravel or sand and covered with the same quality of material. Such bed and covering is to be not less than 6 ins. in thickness around the pipe and be free from stones exceeding 1 in. in diameter. Back filling at and around all culverts shall be thoroughly tamped and left in good condition. The price to be paid per linear foot for laying culverts shall cover all expense and all work of unloading from cars, delivering on road, all excavation and preparation of bed, setting the culvert in place and back filling the trench, excepting that if rock is encountered in excavation for culverts it shall be paid for under the item for rock excavation.

Shoulders.

A shoulder of selected earth or gravel, free from vegetable matter, extending to the gutters or top of slope, and conforming to the cross-sections shown on plans, shall be made or left on each side of the surface to be paved or metaled. This shoulder shall be rolled according to the directions of the engineer.

Surface Drainage.

The sid editches or gutters shall be built according to the lines, grades and cross-sections shown on plans and shall have a uniform slope with suitable side outlets to carry the water quickly away from the highway wherever necessary. All existing culverts found on the line of the work, crossing intersecting roads, driveways, approaches, or bar ways in front of any property shall be examined, cleaned out, repaired, extended, raised or lowered as each particular case may require, so as to be in harmony with and conform to the grades established for the conduct of this work. Payments for work in connection with surface drainage shall be made in accordance with such items as excavation, laying or building culverts or drains, and in cases where pipe other than metal culverts is required the Commission will furnish vitrified pipe f. o. b. cars at the nearest railroad station and the contractor will be paid in accordance with price bid per linear foot for laying metal culvert pipe.

Cobble Gutters.

Cobble gutters shall be constructed where indicated on the plans. The stones used shall be hard, sound stone set with the longest dimensions vertically with the large end down. The stones must be six inches to nine inches in size. The largest stones shall be selected and set along the edge of the guttr. The cobbles shall be laid in a bed of suitable sand or gravel at least six inches deep and of sufficient height to allow for thorough ramming. After the stones have been sufficiently rammed the surface shall be covered with sand and all joints broomed full.

Guard-Rail.

Guard-rails shall be built according to the exact dimensions shown on detail plans. All materials and labor shall be provided by the contractor. They shall be painted when dry with two coats of white paint of a brand approved by the engineer.

Side-Underdrains.

Side-underdrains will be located by the engineer wherever considered necessary and shall be built according to detail plans. Each joint shall be wrapped with a strip of burlap 4 ins. wide. The contractor shall furnish all materials and labor for the same, and will be paid per linear foot of drain complete.

SPECIFICATIONS FOR GRAVEL ROAD.

Gravel shall be furnished by the contractor from banks approved by the engineer and shall be of a quality satisfactory to the engineer. In general it shall consist of hard, sound, durable stones of various sizes, ranging from pea stone to a maximum size of 3¼ ins. The quality of the binding material shall be determined by the engineer. The amount of binder contained in the gravel shall be not less than 15 per cent nor more than 25 per cent and in case the fine material which occurs in the bank is deficient or is not suitable as a binder the contractor will furnish suitable material and spread a layer of such material on each course, mixing and rolling the same, as directed by the engineer until it is thoroughly bonded. Gravel shall be spread in either two or three courses. The roller shall weigh at least ten tons. Any depressions shall be filled and compacted, at the time they appear, with the same material which is being used.

Two-Course Road.

Whenever the smaller sizes of stone predominate, the bottom course shall have a thickness of 4 ins. after rolling. This course shall be bonded with fine material before the second course is applied. The second, or top course shall be similar to the bottom course and shall have the same thickness. It shall be bonded with fine material until a firm, hard, smooth surface is produced. The rolling of each course shall be done while the gravel is wet, using a sprinkler if so ordered by the engineer.

Three-Course Road.

Whenever the larger sizes of stone predominate the bottom course shall have a thickness of 3 ins. after rolling. The second course shall be of the same kind of material and of the same thickness. The top course shall have a thickness of 2 ins. after rolling and contain stone not larger than 1½ ins. in size. Each course shall be thoroughly bonded with fine material by mixing, rolling and sprinkling until a firm, hard, smooth surface is produced.

Spreading.

Spreading wagons may be used when approved by the engineer. When dump carts are used the gravel shall be dumped upon platforms or upon the ground on the sides and then spread uniformly over the surface to be built. It may be also spread with shovels from the carts. The contractor shall deposit where directed by the engineer, along or near the edge of the road, in piles neatly formed and approximately 500 ft. apart 5 cu. yds. of gravel for use in maintenance of the road. Gravel surface when finished shall conform to the lines and grades given by the engineer in accordance with plans and specifications. No allowance will be made for material which may be driven into the subgrade by rolling.

Payment for gravel road will be made per cubic yard compacted into place in accordance with the thickness specified on plans.

Gravel for maintenance shall be paid for at the price bid per cubic yard, but measured in the wagon at point of delivery on the road.

PERSONNEL.

The personnel of the present commission is as follows: Lyman H. Nelson, chairman, Portland, Me.; Philip J. Deering, Portland. Me.; William M. Ayer, Oakland, Me., commissioners. Paul D. Sargent, chief engineer, Augusta, Me.; Parker L. Hardison, assistant engineer in charge of state aid, Augusta, Me. We are indebted to Paul D. Sargeant, chief engineer, for the data from which this article was prepared.

CONSTRUCTION PLANT

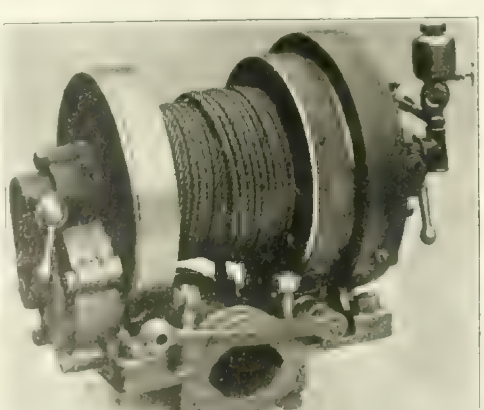
MACHINES — DEVICES — MATERIALS

A New Portable Hoist for Mines, Contract Work and General Light Service.

(Contributed.)

A new type of hoist has been placed upon the market by the Ingersoll-Rand Co., New York City. It is intended for light lifting work, having a capacity up to half a ton and, due to its light weight, which is under 300 lbs. complete, it is particularly suitable for use as a portable hoist for mines, for contract work, for manufacturing plants, for power houses and in railroad shops and ship yards. It is being used in structural work for raising and lowering materials and tools, for placing concrete forms, steel beams and columns, and for shifting light machinery. Contractors also find it suitable for laying heavy water pipe, sewer tile and for the other operations of trench and tunnel work. It is particularly handy for moving dump cars over a limited distance.

The main base of the hoist is arranged so that it can be bolted to a timber and by means of a cap which comes with the hoist it can be clamped to a circular member, as a mine column or arm, shaft bar, or pipe. The adjustment can be made quickly. The dimensions of the hoist are 21¼x16½ ins. and the height is 20⅛ ins. The drum is 6 ins. in diameter with a space of 7 ins. between flanges. This will accommodate a length of 700 ft. of ¼-in. rope or 450 ft. of 5/16-in. rope. The capacity is 1,000 lbs. at a rope speed of 85 ft. per minute and a pressure of 80 lbs. It operates with either compressed air or steam.



Air Operated Hoist for Construction Work.

The motor or engine is of the reversible square piston type, giving four impulses per revolution of the engine. There are no dead centers. The drum is mounted independent of the engine shaft and is operated through the medium of a clutch and gears. Safety is provided for by a powerful worm-operated band brake lined with "Raybestos."

Referring to the illustration showing the front view of the "Little Tugger" hoist, the engine is on the right-hand side, the gear case is on the left-hand side and between the two are the brake and drum. The lever on the left controls the gears and clutch, the one on the right controls the direction of operation and the bottom lever operates the brake. The speed of hoisting is entirely at the will of the operator. When he releases the throttle it returns automatically to central position shutting off the power and stopping the hoist. Oftentimes the hoist will be used for haulage purposes and the release feature enables one man to handle this class of work. He can leave the control lever and carry the rope to the car. Hoists without this feature require two men, inasmuch as the rope has to be released under power.

There are no moving parts exposed except the drum, all gears and shafts being covered. This is an especially desirable feature for underground operation where the light is none too good and where there is constant danger of workmen's clothes or bodies getting caught in machinery.

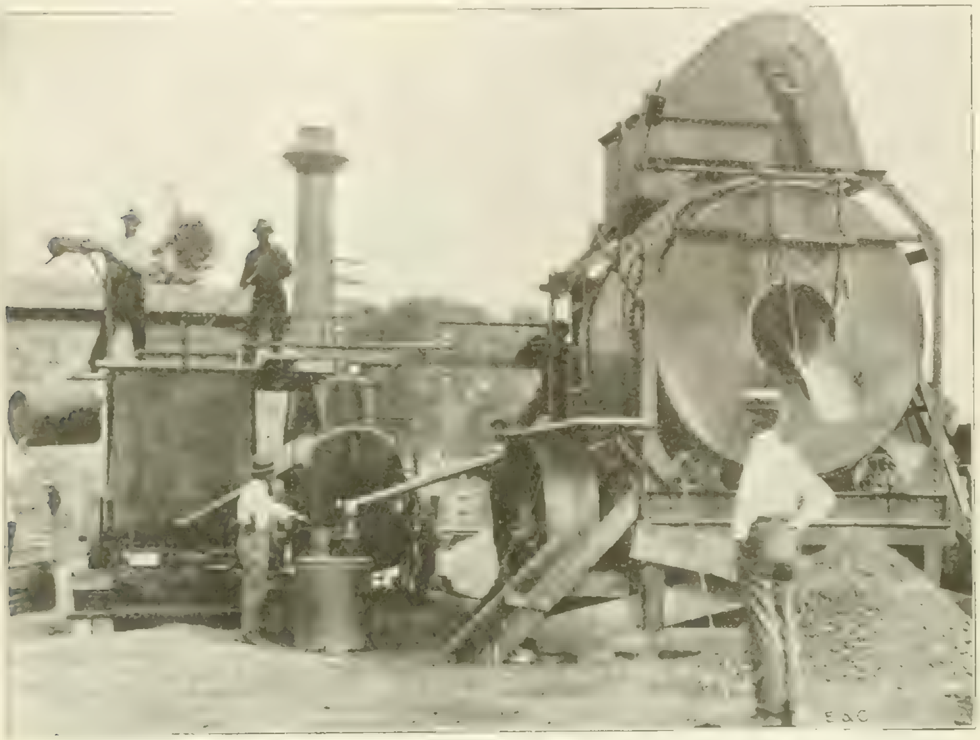
An Improved Manhole Cover.

The manhole cover illustrated herewith is designed to prevent the rattling and accidental removal by the wheels of passing vehicles sometimes noted in the use of manhole covers of the ordinary type. The wedges shown in the accompanying illustration of the cover do the work of tightening the cover. It is claimed that the action of frost does not bind this cover so that its removal is made difficult. This claim is reinforced by the statement that covers of this type have been used for the past two winters in St. Paul without trouble from binding due to frost. They are guaranteed to be easy to take up in frosty weather. Covers of this type are used for sewer manholes, coal hole covers, water caps and similar purposes. These covers are manufactured and sold by the D. & D. Safety Cover Co. of 2423 University Ave., St. Paul, Minn.

The D. & D. Safety Manhole Cover.

The Equitable Asphalt Mixing Plant.

An asphalt mixing plant composed of units easily moved and quickly set up for operation has been developed by the Equitable Asphalt Maintenance Co., Commerce Building, Kansas City, Mo. The plant comprises the following units: A drum asphalt mixer on which is mounted a hopper and bucket elevator and oil burning air heater; an oil tank; a portable steam boiler; a melting kettle; and a measuring tank. The various units of the plant are clearly shown in the illustration.



Equitable Asphalt Plant in Operation.

The mixer proper has an 8-ft. steel drying, melting and mixing drum mounted on a frame made up of steel channels, in turn, mounted on wheels for use in transportation. The drum is supported upon four heavy cast chilled trunnion wheels with steel shafts turning in babbitted bearings, and is rotated by gearing directly connected with a vertical steam engine. The air heater is lined with fire brick and provided with cast iron doors and a steam spraying oil burner. The elevator buckets are of malleable iron and are driven by a sprocket chain and wheel. The hopper is of sufficient size to hold an entire batch and is provided with a suitable valve for charging material into the drying and mixing drum.

The oil tank is of sheet steel and has a capacity of 200 gals. The steam boiler is of the wheel-mounted, locomotive firebox type of 40 HP. A 20-HP. vertical, center crank steam engine is mounted on the mixer. The melting kettle of 800 gals. capacity is portable, mounted on iron wheels and steel axles and has a brick-lined furnace, hinged smokestack and a hoisting crane. The measuring tank is steam jacketed and has sufficient capacity for one entire batch.

In operation the materials comprising the mixture are charged into the drum in a wet or dry state in their proper proportions, then a blast of hot air without flame from an oil burning air heating furnace is passed through the drying and mixing drum, which in revolving causes the materials to be cascaded through the large volume of hot air, until the moisture is driven from the sand or stone, at the same time causing the asphalt to melt and adhere to the mineral aggregate, requiring about 15 minutes to produce a perfect mixture. The rated capacity of the plant is 800 sq. yds. of 2-in. asphaltic surfacing in a day.

It is claimed this plant will successfully heat and remix old asphalt and that no changes are required in the plant in varying the proportions of the asphaltic mixture. The drum is easily cleaned by operating the plant empty for a few minutes.

The weights and dimensions of the plant are as follows: Total weight of 8 ft. drum plant, 32,000 lbs.; length over all, 18 ft.; height over all, 20 ft.; height for moving, 15 ft. 8 ins.; total width, 12 ft. 8 ins.

Welding Malleable Castings with the Oxy-Acetylene Torch.

A method of performing the difficult process of welding malleable castings has been developed by the Vulcan Process Co., Minneapolis, Minn. The process is described as follows: In preparing the work for mending, the fracture is chipped away in the form of a V groove with the pointed bottom just coming to the surface on the opposite side, or, if the casting is thick and the opposite side accessible, two grooves are cut with their pointed bottoms meeting in the center. The part surrounding the fracture is then heated with an oxy-acetylene torch to a bright red, and sprinkled with Vulcan bronze flux followed by a few drops of Tobin bronze melted from the welding rod. If the bronze remains in a little globule the work is not hot enough, but if it spreads and adheres to the surface, the temperature is right, and the groove should be quickly filled. It is not advisable to keep the work hot any longer than is necessary, but to make the mend as quickly and at as low a temperature as possible. The behavior of the bronze affords a guide in regulating the temperature. This process cannot be called autogeneous welding, but a malleable casting mended in this way is practically as good as one piece. It has about the same tensile strength and ductility as the original and the process has the advantage of being very quickly performed.

A Waterproofed Concrete Fountain.

A test of integral waterproofing planned for the Panama-Pacific Exposition is illustrated. The water jets in a spray from the ornament at the top of the dome, spreads over and flows down the dome and is collected in a concealed gutter around its edge, whence it flows down the hollow columns supporting the dome. The entire structure is concrete waterproofed with Ceresit made by the Ceresit Waterproofing Co., 935 Westminster Building, Chicago, Ill.

Waterproofed Concrete Fountain.

Commission Government for Frankfort, Ky., and Wheeling, W. Va.—Two of the latest cities to take steps for the adoption of the Commission form of Municipal Government at Frankfort, Ky., and Wheeling, W. Va. In the first mentioned petitions have been filed for a vote on the question at the November election. At Wheeling a committee is now drafting a charter which will be submitted to the next session of the State Legislature.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLII. CHICAGO, ILL., SEPTEMBER 23, 1914. Number 13.

War in Europe Will Not Seriously Retard Prosecution of American Municipal Improvements.

Municipal improvement work should, from evidence collected by ENGINEERING AND CONTRACTING, be ordinarily active during the year to come. Of 150 cites whose mayors have replied to the inquiry of this journal respecting possible curtailment of city improvements due to war conditions,

- 3% are expecting that construction work will be decreased, but solely because of local conditions.
- 19% are having or expect to have difficulty in disposing of improvement bonds.
- 63% think that European conditions will have no effect upon municipal work.
- 5% have already financed all work proposed for the coming year.
- 10% expect to do even more work than usual.

Taken only at their face value these figures are highly encouraging. We believe that a premium may safely be placed on their face value.

The normal city can, we think, well afford to put forth extra effort to secure at this time means for constructing public works to an amount beyond the normal. One reason is that by increasing public works, employment will be afforded to any who may because of business depression in the industries be otherwise idle—consumers but not producers. A more purely business reason is that in so far as war conditions curtail industrial activities they cheapen labor and materials. The city can do more work for the same money than under normal higher wages and prices. The mayor of an important eastern city states the conditions tersely as follows: "Municipal undertakings are being continued, and others planned for, with the same alacrity as in the time when universal peace existed. Labor can be employed at a decreased wage scale and this, in conjunction with other conditions now existing, makes possible the consummation of municipal undertakings at a cost 15 to 20 per cent less than could be expected during prosperous times." A final reason is that it is the duty of society in its organized units for co-operative action—the city, the state or the nation—to bulwark the individual with confidence by unabated vigor in prosecuting the normal activities of the unit.

About thirty of the cities which report on their planned improvements, doubt their ability to secure means with which to pay for the work and therefore their ability to undertake this work. That this doubt is to a large degree unwarranted in any city whose credit is not for other reasons than European war conditions seriously impaired, is demonstrated by the reports received from the 120 cities that have realized or confidently expect to realize on their credit. Those cities that record failure to market bonds have in most cases gone no further than the bond houses for their market. When conditions are normal this is far enough, but under exceptional conditions of war and interrupted commerce the city that goes no further deserves little credit for its enterprise or courage.

With good credit there can be developed a market for city bonds within the city itself. Five cities reporting, one in Ohio, one in Illinois and three in Kansas, report a certain market for their bonds among their own citizens. There are a score of other cities ranging in size from Chicago, Ill., which within a year has marketed a million or two in bonds by direct sale without special effort, to Independence, Kans., which reports a waiting list of citizen buyers on file at the City Hall, which are successfully selling bonds "over the counter."

There are fewer investments more safe than city bonds for public improvements. Many of them are secured by special assessment, all are secured by wide taxing powers and all ensured by limited powers of the city to borrow against its assessed valuation. Coincidentally there is no more desirable market for a city's bonds than the investing citizenship of that city. The interest paid becomes a part of the local income, the citizen purchaser is given a vital interest in the financial management of the city, the city's financial directors are face to face every day with their neighbors and business associates whose money the city has borrowed or seeks to borrow. All these things are forces toward conservative and honest financing.

The measures to be taken by cities for marketing their bonds locally are two: Bonds of small denomination, as $50, $100 and $500 are required to attract the small investor. Even where a man's savings are enough to purchase a $1,000 bond he prefers to buy with the amount a number of bonds of smaller value because this permits him more easily to cash in a portion or the whole of his securities when an emergency need arises. Second, the city must make known the securities it has to offer and adopt methods to encourage purchasers. Very few cities which attempt the direct sale of bonds seek a market as do the bond houses. For example, the writer receives periodically from bond houses in his home city descriptions of the securities they have for sale with statements of the financial standing of the borrowers. But the writer knows only through an occasional financial news item in the daily papers that the city itself has bonds to sell or of what kind and for what purpose these bonds are. In normal times there is perhaps no need for the city to seek bond purchasers as do the bond houses, but in abnormal times a wisely conducted campaign for customers may be a necessity and no city should feel satisfied, until it has tried and failed so to market its bonds, to discontinue needed improvements.

The Segregation and Analysis of Local Traffic: A Neglected Factor in Traffic Recording on Country Roads.

In the early stages of the development of modern highway economics the accurate determination of the character and extent of the traffic on typical roads of different localities is perhaps all that can be expected. By this means a measure by which the relative wearing qualities of different road surfaces may be determined will be established. And, indeed, even the approximate determination of the amount of traffic different road surfaces will carry economically is a factor well worth establishing at any reasonable cost.

The ordinary method of taking a traffic census consists in stationing observers at intervals along the road and noting carefully the type and number of vehicles passing the stations. From data obtained in this fashion "tonnage per yard wide" and various other units of measure have been derived. The traffic census is well enough as far as it goes, but it does not go far enough.

A classification of traffic generally accepted by students of transportation problems consists of a division into two types: (1) through traffic and (2) local traffic. The former is variable and difficult to estimate, but the latter may be determined with a fair degree of accuracy, especially with reference to heavy loads. Determining one element makes easier the determination of the other. In the past, estimates of the probable amount of local traffic have been neglected, or, in many cases, all traffic has been assumed as local—a condition which is true on very few main roads.

For purposes of economic road design it is necessary to know the amount and type of present traffic and to have some conception of the probable future traffic after the road is improved. The point most worthy of emphasis is that by dividing present traffic into its two parts and determining the *maximum present tonnage of local traffic and its possible fluctuations*, a rational basis for the design of the road surface may be established. At least one element of chance is eliminated.

To secure these data more than a mere census is required. *A traffic analysis with regard to origin* is necessary, the logical subdivision of which into two parts would consist of, (1) traffic from a distance, and (2) traffic originating in territory tributary to the road.

Through traffic is subject to wide seasonal fluctuations both as to the number of vehicles and the character of the loads they contain. A traffic census must, therefore, be so taken that these fluctuations will be considered. Average values are satisfactory for determining wear but maximum values are also quite important in determining width and alignment. These considerations apply not only to pleasure traffic but to goods hauling. An interesting example of the variation in traffic resulting from the hauling of farm produce is found in the marketing of tobacco. This crop is usually sold at auctions held in warehouses located in the various county seats throughout the tobacco growing regions. An increase in the prices paid at a warehouse may result in turning the traffic from several surrounding counties toward that warehouse. As a rule an empty return haul is avoided by the farmer, resulting in increased business for the merchants in the town at which the tobacco was sold. The same rule holds with many other farm crops.

The determination of the maximum possible tonnage of local traffic is a perfectly feasible project within certain limits. In the words of A. M. Wellington, it is possible "to bring reasonably near each other the maximum and minimum probabilities—'the limits of error in either direction, somewhere within which lies the truth and anywhere outside of which lies a certainty of error.'" Studies instituted with this end in view must contemplate the securing of data concerning the type and quantity of farm products raised within the tributary area, the percentage of these hauled, the types and extent of specialized farm industries, i. e., dairying, truck growing, etc., the seasonal distribution of marketing periods for different crops and the rapidity of marketing, the character and tonnage of loads hauled into farming regions from neighboring towns, the ratio of pleasure traffic to heavy hauling, the location of churches, school houses and public gathering places, and the traffic facilities afforded by branch roads leading into main thoroughfares.

The subject of traffic analysis is too broad to discuss fully here, nor do data on which to base such a discussion exist. The possibilities of the subject and unexplored nature of the field should challenge the thoughtful engineer to efforts toward the investigation of its features.

The Engineer's Reading and Study.

It is sometimes charged that the young graduate feels that his studies were completed just prior to his graduation and that he considers himself ready to practice his profession indefinitely without further study. While many engineers proceed as if they held this view, and some doubtless do hold it, observation indicates that the majority of engineers who do not remain students after the collegiate period fully appreciate the importance of further study, but are at a loss to know how to proceed. The engineer has learned to apply himself to study while in college, of course, but on graduation he is confronted with two great obstacles to further study, the one real, the other largely imaginary. In the first place, his outlook has become so broad that he sees many things he would like to study, but finds it difficult to separate the essentials from the nonessentials. He finds such a wealth of books of potential interest that he is quite overwhelmed and does not know where to begin. Without advice, therefore, he dislikes to sacrifice his time in the hunt for really good and useful books and quite often in despair gives up the thought of further reading and study. In the second place, he is very prone to argue and to convince himself that he hasn't the time to read.

The first difficulty named is the greater, but it is by no means insurmountable. Those who do read, and there are many such, must discover the really good and helpful books, and they should make known the results of their exploratory tours into the realm of literature. The engineering specialist who has always been studious should make known the titles of the books which he has found of greatest help, whether such help be direct or indirect. Books of direct utility are more easily selected than those of indirect assistance, in the nature of things, but the latter are of equal importance. To illustrate, certain works on logic, economics, etc., would doubtless be of value to the engineer if he but knew the ones to read. Clearly here is a case in which the old should advise the young. The experienced and successful man should list the books which have done him the most good and should give publicity to his findings in this field. This might be done through the medium of journals, in addresses to students, letters to professors, or by any other equally convenient and efficacious method. It would be a splendid idea for engineering professors to recommend an extended course in reading for the guidance of those industrious ones who desire to continue their studies after graduation.

It is more difficult to suggest a plan to solve the difficulty of the man who lacks the time to read, and it can here be attempted in only general terms. In the first place, "the lack of time" to read is often merely an euphemistic way of expressing a disinclination to read. This type of time lacking is a state of mind. The man who really believes that he hasn't time to read will find time when he becomes convinced of the importance of reading.

The importance of reading can scarcely be overestimated. This refers, of course, to the man who aspires to positions involving duties which are not purely routine. The willingness to read is always an attribute of the successful engineer, and his success rests largely upon this studious habit. We here quote from the recent writings of such an engineer with reference to this subject:

> The engineer must be a close student of the art all of his professional life. So soon as he begins to relax his attention in this direction, so soon does he begin to drop back and be in danger of being superseded. No matter how busy he may be, or with what great responsibilities he may be charged, he cannot afford for one instant to relax his study time. New methods must be investigated where they have promise. The literature of his specialty should all pass under his eye, at least to the extent that he catches the meaning and purport of every important article, paper or book presented during the year.

As far back as in the times of King Solomon some men were taught to divide their time. The man who drives and is not driven by his work must so plan it as to leave sufficient time for reading and study. It has been truly said that the man who declares he has no time to read is unconsciously advertising his slavery to detail and his arrested development.

Electrolysis Mitigation and Engineering Mediation.

The Bureau of Standards has performed a valuable service by its exhaustive field and laboratory studies of the effects of electrolytic action on underground structures, and by its studies of practicable measures of mitigation. We publish in this issue an account of the Bureau's investigations. The article merits study by engineers connected with pipe and cable owning utilities and by electric railway engineers. Many of the specific conclusions recorded are of technical and economic significance. A more important feature of the Bureau's work, to which special attention is here directed, is the building up of a spirit of cooperation between the engineers who represent the utilities whose interests clash due to the destructive effects of electrolysis. This task of mediation could perhaps have been performed so well by no other agency. The fact that it has been successfully performed by the Bureau of Standards in at least two cases may, we trust, establish a precedent that will be frequently followed.

DRAINAGE AND IRRIGATION

The Hydraulics of Irrigation, Drainage, and Other Channels.

Contributed by Louis Schmeer, San Fernando, California.

The fundamentals of our present day science of hydraulics are based on the laws of bodies falling free, first investigated by Galileo (1564-1642) and first mathematically expressed by his disciple, Torricelli (1608-1684). While Torricelli and his contemporaries occupied themselves chiefly with the hydraulics of orifices, later investigators like Mariotte studied the phenomenon of the motion of fluids in conduits and discovered the influence of the frictional resistance of the walls of a channel on the speed of flow. Brahms in Germany and Chezy in France found the frictional resistance to be proportional to the wetted circumference of a channel divided by its cross-section, a factor since called the mean depth or the mean hydraulic radius and denoted by R. This discovery led Chezy to write the formula (1776)

$$V = C\sqrt{RS}$$

in which V = the mean velocity of flow,
S = the slope of the surface of the water in the channel,
C = a coefficient of velocity.

Chezy and his immediate followers assumed this coefficient to be a constant; it is now however known to be very variable and the determination of its variations is the chief object of present day investigations. Since Chezy's time several thousand more or less reliable gagings of flow in channels of all descriptions have been recorded. The most noteworthy work in this line was accomplished by Darcy, chief engineer of roads and bridges in France and by Bazin, his successor. The results of their investigations are embodied in the classical works, "Mouvement de l'eau dans les Tuyaux" and "Recherches Hydrauliques." The former treats of flow in conduits under pressure, the latter is an inexhaustible mine of the most precise information on many phases of flow in semi-circular, rectangular and trapezoidal open conduits. The simple formulas proposed by Darcey and Bazin, based chiefly on a graphical analysis of the data contained in "Recherches Hydrauliques" were formerly much in favor, but have in recent times been superseded by an equation elaborated by two ingenious Swiss engineers, Ganguillet and Kutter.

An analysis of values of the coefficient C reveals the fact that it varies: (1) With the mean hydraulic radius of the conduit, (2) with the degree of roughness of the surface in contact with the flowing water, (3) with the velocity of flow, or, with the head to which the velocity is due.

The earlier investigators, like Weisbach, merely sought to define the variation of the coefficient with the velocity. Darcy and Bazin considered its variation with the velocity the least important and confined themselves to a definition of the others. Ganguillet and Kutter, however, aimed to embody all its variations in their formula. Within certain limits they were fairly successful; that is, their equation expresses the variations of C with a fair degree of precision whenever conditions of flow are similar to those on which the data at their disposal were based.

Besides the data contained in "Recherches Hydrauliques" Ganguillet and Kutter had at their disposal numerous gagings of flow in European rivers and Swiss torrential streams. These data, based on flow in channels of a wide range of values of the mean hydraulic depth, they considered best suited as a base for a general equation. The result is, as intimated above, a formula which gives fairly accurate and trustworthy values when applied to flow in channels in earth, rivers, torrents. It ceases to be reliable when applied to flow in channels of a lesser degree of roughness of the wet perimeter, not only because the variation of C with the velocity is imperfect, but also because its variation with R is defective and entirely amiss for channels of small dimensions, such as pipes ½ to 3 ins. in diameter or feeble depths of flow in open conduits. Nevertheless the formula is widely used to compute the flow in channels having smooth wet perimeters; it is even applied to calculations of flow in conduits under pressure.

The defects of the Kutter formula have been pointed out time and again. Nobody, however, seems to have conceived the idea that some at least of its defects could be eliminated. Yet this possibility exists. To accomplish it, it is only necessary to use the most precise data available and subject these to the very process employed by Ganguillet and Kutter in the elaboration of their formula. The result will be a new equation, like the original, not embodying any general, true law of flow, but expressing the variations of C precisely as they are embodied in the data used. Of the following two examples, selected because the values of n in these happen to be the same as in the original, the first is derived from data relating to flow in the rivers of the La Plata system, South America, the second from data relating to flowing in channels having steeper slopes.

$$(1) \quad C = \frac{52.5 + \dfrac{1.835}{n} + \dfrac{0.0007}{S}}{1 + \left(52.5 + \dfrac{0.0007}{S}\right)\dfrac{n}{\sqrt{R}}}$$

$$(2) \quad C = \frac{30.7 + \dfrac{1.785}{n} + \dfrac{0.007}{S}}{1 + \left(30.7 + \dfrac{0.007}{S}\right)\dfrac{n}{\sqrt{R}}}$$

It will be observed that the first two constants in these equations decrease in value as the third increases; the first gradually vanishing as the third approaches unity. It is plain that in the first formula the third term above the line may be omitted without materially affecting results and the formula will then read

$$V = \frac{1.835 + 52.5n}{(\sqrt[4]{R} + 52.2n)_n} R\sqrt{S}$$

which will be found a good substitute for the original formula. In case data relating to flow in very smooth channels are subjected to the process, the second constant will have the value 2.70 or over, indicating that here the imaginary dividing line between C increasing and decreasing with increasing velocity is given by $R = (2.7)^2 = 7.29$ or over. Such equations, however, no matter how nicely they express the variations of the coefficient C as embodied in the particular set of data from which they are derived, are not general, but special formulas having one defect or another; moreover, they do not lend themselves to the various transformations required in many hydraulic calculations.

This led the contributor to an investigation along different lines. The object aimed at was a simple, if possible as straight line general formula, embodying all the variations of the coefficient C, and the analytical method of investigation was chosen.

Beginning with the variation of the coefficient C with the mean hydraulic radius, the power of this factor to which the velocity of flow is proportional is found by putting

$$X = \frac{\log V_1 - \log V_0}{\log R_1 - \log R_0}$$

From the most reliable experimental data relating to flow in channels of the widest range of mean depths, that is, in small ditches and great rivers, the value of X was found to be equal to ¾, very nearly. Neglecting the variation of C with the velocity, this leads to the equation

$$V = FR^{\frac{3}{4}}\sqrt{S}$$

in which F is a factor varying with the degree of roughness of the conduit. Since $R^{\frac{3}{4}} = \sqrt[4]{R}\ \sqrt{R}$ it is seen that the value of C is in this equation represented by

$$C = F\sqrt[4]{R}$$

But from data relating to flow in channels not in earth other values of X are obtained. The equation is therefore not general; it may, however, serve as a suitable basis for a formula of general application. The variation of the coefficient C with R may be suitably expressed by replacing the variable F by a constant, so that

$$C = (\text{constant} \times \sqrt[4]{R})\ 1 + \frac{m}{\sqrt[4]{R}}$$

in which m is a variable, depending for its value on the degree of roughness of the conduit. The value of the constant is given by

$$\text{Constant} = \frac{C_1 - C_0}{\sqrt[4]{R_1} - \sqrt[4]{R_0}}$$

in which C_1 and C_0 are values of C corresponding to a velocity of 1 ft. per second and $\sqrt[4]{R_1}$, $\sqrt[4]{R_0}$, the corresponding values of $\sqrt[4]{R}$

Selecting the most reliable experimental data corresponding to different depths of flow in a channel of large dimensions the value of the constant is found to be equal, for English measure, to 66, for metric measure, to 50, very near.

From the same data the value of the variable m is found by putting

$$m = \frac{C}{66} - \sqrt[4]{R}$$

The variation of the coefficient C with both the mean hydraulic radius and the degree of roughness of the conduit is consequently given by

$$C = 66\ (\sqrt[4]{R} + m) \text{ or,}$$

$$C = (66\sqrt[4]{R}\ )\ 1 + \frac{m}{\sqrt[4]{R}}$$

$$V = 66\ (\sqrt[4]{R} + m)\ \sqrt{RS}$$

The terms $66\ (\sqrt[4]{R} + m)$ and $66\ (\sqrt[4]{R} + m)\ \sqrt{RS}$, holding good only when the velocity of flow is equal to 1 ft. per second, or else when C does not vary with the velocity, may be called C prime and V prime and denoted by C_1 and V_1.

If the value of C corresponding to any velocity of flow is divided by C prime $= 66\ (\sqrt[4]{R} + m)$, the quotient is a factor which may be called the coefficient of variation of C with the velocity and denoted by (a). The solution of the problem of the variation of C with the velocity consists in finding a mathematical expression for this coefficient a. Putting

$$X = \frac{\log V_1 - \log V_0}{\log(66(\sqrt[4]{R} + m)\sqrt{RS})_1 - \log(66(\sqrt[4]{R} + m)\sqrt{RS})_0}$$

in which V_1 and V_0 are actual measured velocities and $(66\ (\sqrt[4]{R} + m)\ \sqrt{RS})_1$ etc., the corresponding prime velocities, the value of X is found to be a variable power of the prime velocity. The coefficient a is a root of the same velocity and equal to

$$a = (66(\sqrt[4]{R} + m)\ \sqrt{RS})^{x-1} = V_1^{x-1}$$

A comparison of values of m and $x - 1$ reveals the fact that they vary together; when m is positive $x - 1$ is positive, when $m = 0$ $x - 1 = 0$, when m is negative $x - 1$ is negative.

I—EQUATION FOR CIRCULAR CONDUITS RUNNING FULL.

An analysis of experimental data relating to flow in conduits under pressure indicates that $x - 1$ varies chiefly with the roughness of the walls of the conduits, minor variations are due to the dimension of the conduit, to conditions existing at the entrance, to elbows, bends, changes of section, etc. From data relating to flow in pipes of brass, planed staves, new asphalt coated cast and wrought iron, etc., we find

$$X = \frac{9}{8}$$

hence the mean velocity

$$V = (66(\sqrt[4]{R} + m)\ \sqrt{RS})^{\frac{9}{8}}$$

and since $9/8 - 1 = 1/8$

$$a = (66\ (\sqrt[4]{R} + m)\ \sqrt{RS})^{\frac{1}{8}} = V_1^{\frac{1}{8}}$$

From the equation above we have also

$$V^{\frac{8}{9}} = V_1\text{, consequently } V^{\frac{1}{9}} = V_1^{\frac{1}{8}}$$

In the manner indicated above the value of a is found to be equal to

$V^{\frac{1}{12}} = V_1^{\frac{1}{11}}$ for new rivited pipes between 6 and 36 inches in diameter,

$V^{\frac{1}{18}} = V_1^{\frac{1}{17}}$ for new riveted pipes exceeding 36 inches in diameter and also for conduits lined with cement plaster, concrete and brick work,

1.0 for old pipes of all descriptions.

In calculation of flow in conduits under pressure it is frequently necessary to consider losses of head due to the entrance, to eblows, curves, etc. In such cases the equation may be put into the familiar form due to Weisbach:

$$V = \left[\frac{2gH}{1 + Z_0 + Z_1\dfrac{4L}{D} + Z_n}\right]^{\frac{x}{2}}$$

in which

Z_0 = a coefficient denoting the resistance due to the entrance $= 0.505$ for a well rounded entrance not protruding into the reservoir.

Z_1 = the coefficient of friction, denoting the resistance due to the walls of the conduit.

Z_n = a coefficient denoting the resistance due to an elbow, a curve, etc.

Since

$$C_1 = \sqrt{\frac{2g}{Z_1}}$$

we have

$$[66(\sqrt[4]{R} + m)]^2 = \sqrt{\frac{2g}{Z_1}}$$

hence the coefficient of friction

$$Z_1 = \frac{64.4}{[66(\sqrt[4]{R} + m)]^2} = \frac{0.01478}{(\sqrt[4]{R} + m)^2}$$

Inserting this value of Z_1 into the formula above we have for the conditions most favorable to flow, or $x = \frac{9}{8}$

$$V = \left[\frac{2gH}{1.505\ \dfrac{0.01478}{(\sqrt[4]{R} + m)^2} \cdot \dfrac{4L}{D} + Z_n}\right]^{\frac{9}{16}}$$

and similar equations for other values of $\frac{x}{2}$.

These formulas are strictly true, however, only for long conduits; that is, those over 1,000 diameters long, when the term 1.505 may be omitted and when resistances due to elbows, curves, etc., do not enter. This is owing to the fact that losses of head due to the velocity itself, to the resistance at the entrance, to elbows, bends, etc., are always proportional to V^2 while those due to the frictional resistance of the walls of the conduit are proportional to $V^{\frac{16}{9}}$, $V^{\frac{11}{6}}$, $V^{\frac{17}{9}}$, V^2, as the case may be. The relation between the losses proportional to V^2 and those proportional to lesser powers of V determines the power to which the total loss is proportional. On this account, when a conduit is between 300 and 1,000 diameters long, the losses of head for the whole are no longer proportional to $V^{\frac{16}{9}}$, $V^{\frac{11}{6}}$. but to the higher power $V^{\frac{17}{9}}$; in case the conduit is less than 300 diameters long the loss in the whole may be assumed to be proportional to V^2.

The power of the velocity to which the frictional resistance is proportional is somewhat uncertain for conduits of large diameters. Experimental data at present available indicate that $V^{\frac{16}{9}}$, $V^{\frac{11}{6}}$, $V^{\frac{17}{9}}$, hold good for diameters up to and including 9 ft., for greater dimensions considerations of safety at present demand V^2.

For long conduits the equation for the quantity of flow takes the form

$$Q = A\left[\frac{HD^5}{\dfrac{0.01478}{(\sqrt[4]{R} + m)^2}\ 4L}\right]^{\frac{x}{2}}$$

in which for

$$\frac{x}{2} = \frac{9}{16} \quad A = 8.177, \quad D^y = D^{\frac{45}{16}}$$

$$\frac{x}{2} = \frac{6}{11} \quad A = 7.617, \quad D^y = D^{\frac{30}{11}}$$

$$\frac{x}{2} = \frac{9}{17} \quad A = 7.124, \quad D^y = D^{\frac{45}{17}}$$

$$\frac{x}{2} = \frac{1}{2} \quad A = 6.303, \quad D^y = D^{\frac{5}{2}}$$

The equation may be simplified so as to eliminate this multiplicity of values of $\frac{x}{2}$, A and D^y. It may be put into the form

$$Q = 6.303\sqrt{\frac{HD^5}{Z_1 4L}}\ \left[\frac{2gHD}{Z_1 4L}\right]^{\frac{x-1}{2}}$$

in which the second term on the right hand side represents a, the coefficient of variation of $C = V_1^{x-1}$.

This value of V_1^{x-1} may be calculated; in most cases occurring in practice, results sufficiently accurate will be obtained by taking a value corresponding to an assumed prime velocity, thus omitting the second term on the right hand side of the equation. In either case the formula will then read, for all lengths of the conduit

$$Q = 6.303 \sqrt{\frac{HD}{1.505D + \left(\frac{Z_1 4L}{A^2}\right) + Z_n D}}$$

$$H = \left(\frac{Q}{6.303}\right)^2 \frac{1}{D^5}\left[1.505D + \frac{Z_1 4L}{A^2} + Z_n D\right]$$

$$D = 0.4788\left[\frac{Q^2}{H}\left(1.505D + \frac{Z_1 4L}{A^2} + Z_n D\right)\right]^{\frac{1}{5}}$$

The value of D can be found only by successive trials. For a first trial a value of Z_1 corresponding to $D = 4$ ft. will give good results. Values of A and A^2 corresponding to the different prime velocities are found in Table I.

TABLE I.

	$V_1^{1/8}$		$V_1^{1/11}$		$V_1^{1/17}$	
V_1	A	A^2	A	A^2	A	A^2
2	1.093	1.189	1.065	1.134	1.042	1.085
3	1.147	1.316	1.105	1.221	1.067	1.138
4	1.189	1.414	1.134	1.286	1.085	1.177
5	1.223	1.495	1.158	1.340	1.099	1.208
6	1.251	1.565	1.177	1.385	1.111	1.234
8	1.297	1.682	1.208	1.459	1.130	1.277
10	1.333	1.778	1.233	1.520	1.145	1.342

From the equation above it is seen that the variations of the coefficient of friction Z_1 with the velocity is given by

$$\frac{Z_1}{A^2}$$

The practically most useful values of Z_1 are given in Table III.

While the formulas so far given are indispensable in cases where it is necessary to consider losses of head due to the entrance, to elbows, curves, changes of section, etc., or where the discharge of a compound conduit is to be computed, they can in most cases occurring in practice be replaced by far simpler expressions.

For the velocity the original formula reads

$$V = 66 (\sqrt{R} + m) \sqrt{RS}\, V_1^{x-1}$$

Substituting D the diameter for R, the mean hydraulic radius, and I the fall in feet per 1,000 ft. for S the slope, the formula reduces to

$$V = 1.0435 (\sqrt{0.25D} + m) \sqrt{DI}\, V_1^{x-1}$$

and for a prime velocity of 5 ft. per second

$$V = 1.0435 (\sqrt{0.25D} + m) \sqrt{DI}\; 5^{1/8},\ 5^{1/11},\ 5^{1/17}.$$

For the quantity of flow

$$Q = 0.82 (\sqrt{0.25D} + m) \sqrt{D^5 I}\; 5^{1/8},\ 5^{1/11},\ 5^{1/17}.$$

Table II gives velocity and quantity constants $Cv = 1.0435\, V_1^{x-1}$ and $Cq = 0.82\, V_1^{x-1}$ corresponding to a prime velocity of 5 ft. per second and also the practically most useful values of m for circular conduits. Values of $\sqrt{0.25D}$, $D^{\frac{5}{2}}$ and D^5 are found in Table III.

Caution: As intimated above, the experimental data at present available indicate that $A = V_1^{1/8}, V_1^{1/11}, V_1^{1/17}$ hold good for diameters up to and including 9 ft. For greater diameters only two sets of gagings are known to the contributor, one relating to flow in a 10-ft. pipe at Madison River, Montana, the other to flow in a 12-ft. brick conduit at Milwaukee, Wis. These gagings indicate very slight variations of C with the velocity, but the two conduits are not strictly long conduits. In [illegible] should be used and allowance made for the frictional resistance at the entrance.

PRACTICAL APPLICATIONS OF THE FORMULA [illegible]

Weisbach's coefficients for the resistances due to angles, curves, changes of section, valves, etc., are found in every textbook; space does does not permit to give them here. The phenomenon of flow in curves has during the last decade been investigated by Alexander, Saph and Shoder, Williams, Hubbell and Fenkell, Smith and others. The investigations brought to light the fact that the resistance due to a curve does not entirely depend on the relation between the diameter of the conduit and the radius of the bend, but also in a certain degree

TABLE II.

Coeff. of roughness. m.	Velocity constant. Cv.	Quantity constant. Cq.	Description of conduit.
0.95	[illegible]	[illegible]	Straight pipes of brass, tin, lead or exceptionally smooth new asphalt-coated cast or wrought iron. New well-laid pipes of planed staves or polished neat cement.
0.90	1.276	1.00	Pipes of planed staves some time in use; velocity not less than 2 ft. per second.
0.83	1.276	1.00	New asphalt-coated cast iron, wrought iron and lockbar riveted pipes. Other new asphalt-coated riveted pipes up to 6 ins. in diameter. Welded pipes.
0.88	1.208	0.95	New concrete or brick work washed with cement or smoothly dressed with cement-plaster.
0.74	1.208	0.95	Common service pipe, or cast and wrought iron pipe some time in use. Asphalt-coated new riveted pipes up to 3 ft. in diameter.
0.70	1.208	0.95	Pipes of unplaned boards. Pipes of planed staves; velocity less than 2 ft. per second.
0.57	1.147	0.91	Ordinary concrete and cement pipes, sewer pipe, drain tile. Channels lined with brick work.
0.53	1.147	0.91	New asphalt-coated riveted pipes exceeding 3 ft. in diameter.
0.45	1.0435	0.82	Old pipes of all descriptions, fairly clean.
0.30	1.0435	0.82	Incrusted pipes.

TABLE II-A.—LIST OF AUTHORITIES ON FLOW IN CIRCULAR AND SEMI-CIRCULAR CONDUITS.

m.	Description of conduits.	Authority.
0.95	Brass.	Saph and Shoder.
	Tin.	Bossut.
	Neat cement.	Darcy-Bazin.
	Planed staves, new.	Adams, Cochrane.
0.90	Planed staves in use for several years.	Moritz.
0.88	New concrete or brickwork washed with cement or plastered with cement mortar.	Darcy-Bazin, Horton, Lippincott, Newell.
0.83	New asphalt-coated cast and wrought iron pipes.	Darcy, Lampe, Williams-Hubbel and Fenkell.
0.74	Riveted pipes less than 3 feet in diameter.	Saph and Shoder.
0.57	Brick conduits.	Darcy-Bazin, Benzenberg.
	Brick and concrete conduits washed with cement or plastered with cement mortar, in use for several years.	Horton.
0.53	New riveted pipes exceeding 3 ft. in diameter.	Clemens Herschel.
0.45	Old pipes fairly clean.	Darcy.

TABLE III.

D in inches.	D in feet.	$\sqrt{0.25\,D}$	Coeff. of friction = Z_1: m = 0.95.	m = 0.90.	m = 0.74.	m = 0.57.	$D^{5/2}$ in feet.	D^5 in feet.
0.5	0.0416	0.320	00918	00995	01318	01867	.000351	.0000001256
1	0.0833	0.380	00836	00902	01176	01638	.002005	.00000402
1.5	0.125	0.420	00788	00849	01099	01509	.00552	.0000305
2	0.166	0.452	00252	00809	01046	01415	.01134	.0001286
2.5	0.2083	0.473	00730	00784	01005	01359	.01970	.000388
3	0.25	0.500	00703	00754	00961	01291	.03125	.000977
3.5	0.2916	0.520	00684	00733	00931	01244	.04594	.002111
4	0.3333	0.537	00665	00716	00907	01206	.06415	.004115
4 5	0.3750	0.553	00654	00700	00884	01172	.08527	.007416
5	0.4167	0.568	00642	00683	00864	01142	.1121	.01254
6	0.5000	0.594	00620	00662	00831	01091	.1768	.03125
7	0.5833	0.618	00601	00642	00802	01048	.2599	.06754
8	0.6667	0.639	00585	00621	00779	01012	.3629	.1317
9	0.7500	0.658	00572	00609	00256	00980	.4871	.2371
10	0.8333	0.675	00560	00596	00738	00954	.6343	.4018
11	0.9167	0.692	00548	00583	00721	00928	.8045	.6472
12	1.0	0.707	00538	00572	00706	00907	1.0	1.0
13	1 083	0.721	00529	00563	00692	00887	1.222	1.492
14	1.0167	0.735	00521	00553	00680	00868	1.470	2.162
15	1.250	0.748	00513	00544	00668	00851	1 747	3.052
16	1.333	0.760	00506	00537	00657	00836	2.053	4.214
18	1.500	0.782	00493	00523	00638	00809	2.756	7.594
20	1.667	0.803	00481	00510	00621	00784	3.586	12.372
22	1.833	0.823	00470	00499	00606	00762	4.551	20.711
24	2.0	0.841	00461	00488	00591	00743	5.656	32.0
26	2.167	0.859	00452	00478	00599	00725	6.910	47.25
28	2.333	0.874	00444	00470	00567	00709	8.317	69.02
30	2.5	0.889	00437	00462	00557	00694	9.882	97.66
32	2.667	0.904	00430	00454	00547	00681	11.612	143.85
34	2.833	0.917	00424	00448	00538	00669	13.513	184.59
36	3.0	0.931	00418	00441	00529	00656	15.59	243.00
38	3.167	0.944	00412	00435	00521	00645	17.84	318.4
40	3.333	0.955	00407	00429	00514	00635	20.29	411.5
42	3.5	0.967	00402	00424	00507	00626	22.92	524.9
44	3.667	0.978	00398	00419	00501	00617	25.74	662.1
46	3.833	0.999	00393	00414	00494	00607	28.77	827.6
48	4.0	1.0	00388	00409	00488	00600	32.00	1024.0
50	4.167	1.0103	00384	00405	00483	00592	35.44	1255.9
52	4.333	1.0202	00381	00401	00477	00584	39.09	1527.9
54	4.5	1.030	00377	00397	00472	00577	42.96	1845.2
56	4.667	1.039	00374	00393	00467	00571	47.05	2213.2
58	4.833	1.048	00370	00390	00462	00565	51.36	2637.6
60	5.0	1.057	00367	00386	00458	00558	55.88	3125.0
62	5.167	1.066	00363	00882	00453	00552	60.68	3681.7
64	5,333	1.075	00360	00379	00449	00546	65.69	4520.0
66	5.5	1.083	00357	00376	00445	00541	70.95	5033.7
68	5.667	1.091	00354	00373	00441	00536	76.44	5843.0
70	5.833	1.099	00352	00370	00437	00532	82.18	6754.3
72	6.0	1.107	00349	00367	00433	00526	88.18	7776.0
78	6.5	1.129	00342	00359	00423	00512	107.7	11602.9
84	7.0	1.150	00335	00352	00414	00500	129 6	16807.0
90	[illegible]	1.170	00329	00345	00405	00488	151.0	23730.4
[illegible]	8.0	1.189	00323	00338	00397	00498	181.0	32768.0
102	8.5	1.201	00318	00332	00390	00468	210.6	44370.5
108	9.0	1.221	00314	00327	00383	00459	243.0	59049.0
[illegible]	9.5	1.244	00310	00322	00377	00451	278.2	77378.1
[illegible]	10.0	1.257	00303	00318	00371	00443	316.3	100000.0
[illegible]	[illegible]	1.273	00299	00313	00365	00435	358.1	127628
[illegible]	11.0	1.288	00297	00309	00359	00428	401.3	129951
[illegible]	11.5	1.302	00292	00305	00355	00422	418.5	201136
[illegible]	[illegible]	1.316	00288	00301	00350	00416	498.9	248832
[illegible]	[illegible]	1.330	00284	00297	00345	00409	552.4	305176
156	13.0	[illegible]	00281	00294	00341	00404	609.2	371293
[illegible]	13.5	1.355	00278	00291	00337	00399	669.5	448403
168	[illegible]	[illegible]	00275	00289	00333	00393	733.3	537824
174	14.5	1.380	00272	00285	00329	00398	800 6	610973
180	15.0	[illegible]	00270	00281	00325	00384	871.5	759375

on the length of the curve and consequently on the degree of roughness of the conduit. Weisbach did not investigate this phase of the subject and his coefficients for curves are in consequence defective. Alexander's experimental conduits were small wooden pipes, Saph and Shoder used 2-in. pipes of drawn brass, Smith 2-in. pipes of welded wrought-iron and steel with cast-iron fittings, Williams-Hubbell and Fenkell cast-iron mains 12-30 ins. in diameter. Smith's experiments were made with greatest care and his pipes and fittings were those in common use. The results of his experiments are therefore of practical importance.

From the data given by Smith it appears that the loss of head due to a T is equal to

$$h = 0.082\ V^2$$

and that the resistance due to a curve decreases with increasing radius until the radius is equal to 2.5 diameters of the conduit and increases again for curves of greater radius. From an analysis of Smith's data the contributor found that for bends of which the radius is equal to 2.5 diameters and over the frictional resistance is

$$11.81 \left(\frac{D}{R}\right)^{0.65}$$

times the resistance in an equal length of straight pipe and therefore the excess frictional resistance due to a curve

$$11.81 \left(\frac{D}{R}\right)^{0.65} - 1$$

times the resistance in a tangent of equal length. The excess frictional resistance due to a curve vanishes when

$$\frac{R}{D} = 11.81^{\frac{1}{0.65}} = 44.6$$

and is a maximum when

$$\frac{R}{D} = \sqrt{2 \times 44.6} = 9.45$$

The equation for the resistance to a curve of 90 degrees may be written

$$Z_2 = Z_1\, 6.28 \frac{R}{D}\left[\left(11.81\left(\frac{D}{R}\right)^{0.65}\right) - 1.0\right]$$

in which Z_1 = the coefficient of friction $= \frac{0.01478}{(\sqrt[4]{R} + m)^2}$, R and D = radius of curve and diameter of conduit in feet.

Although the experiments of Williams-Hubbell and Fenkell were made on the same class of conduits as Smith's, an analysis of their data leads to quite different results. According to their experiments the value of Z_2 for a curve of 90 degrees is equal to

$$Z_2 = Z_1\, 6.28 \frac{R}{D}\left[\left(6.1\left(\frac{D}{R}\right)^{\frac{3}{13}}\right) - 1.0\right]$$

hence, in this case, the excess frictional resistance due to a curve vanishes only when

$$\frac{R}{D} = 6.1^{13/3} = 2500$$

The results of the experiments of Saph and Shoder may be expressed, for a curve of 90 degrees, by the equation

$$Z_2 = Z_1\, 6.28 \frac{R}{D}\left[\left(2.2\left(\frac{D}{R}\right)^{\frac{3}{13}}\right) - 1.0\right]$$

According to this the excess frictional resistance due to a curve vanishes when

$$\frac{R}{D} = 2.2^{\frac{13}{3}} = 30.5.$$

Table IV contains the factors indicating the excess of the frictional resistance due to a curve in comparison with an equal length of straight pipe. It will be observed that for $\frac{R}{D} = 2.5$ and $\frac{R}{D} = 5$ the values based on the experiments of Smith and those based on the experiments of Williams-Hubbell and Fenkell do not differ greatly; for greater values of $\frac{R}{D}$ the factors based on Smith's experiments are most likely to be correct. The results of Saph and Shoder's experiments may be applied to computations of the excess resistance due to curves in stave pipes and others having no fittings.

TABLE IV.

$\frac{R}{D}$	$11.81\left(\frac{D}{R}\right)^{0.65}-1$	$6.1\left(\frac{D}{R}\right)^{\frac{3}{13}}-1$	$2.2\left(\frac{D}{R}\right)^{\frac{3}{13}}-1$	$\frac{R}{D}$	$11.81\left(\frac{D}{R}\right)^{0.65}-1$	$6.1\left(\frac{D}{R}\right)^{\frac{3}{13}}-1$	$2.2\left(\frac{D}{R}\right)^{\frac{3}{13}}-1$
2.5	4.511	3.928	0.780	25	0.457	1.902	0.047
5	3.149	3.208	0.517	30	0.292	1.782	0.003
8	2.057	2.706	0.361	35	0.171	1.686	.....
10	1.644	2.585	0.293	40	0.074	1.604	.....
12	1.348	2.437	0.240	45	.....	1.537	.....
15	1.008	2.264	0.178	50	.....	1.473	.....
20	0.685	2.055	0.102	100	.....	0.583	.....

TABLE V.

Description of conduits.	$\frac{R}{D}$ = 2.5	4	5	6	10	15	20	25
	Z_2							
1¼-in. wooden tubes without fittings..........	0.049	0.057	0.059	0.059	0.048	0.015	.....	.....
2-in. brass tubes bent into curves................	0.091	0.112	0.121	0.128	0.137	0.147	0.095	0.055
2-in. cast iron pipes with common fittings.......	0.531	0.717	0.743	0.761	0.775	0.712	0.645	0.538
12-in. cast iron mains	0.249	0.344	0.401	0.451	0.634	0.821	0.982	1.123
30-in. cast iron mains............................	0.270	0.371	0.432	0.492	0.699	0.918	1.111	1.300

Table V gives values of Z_2 based on the original experimental data from which the equations given above are derived. It will be observed that the values for the 2-in. cast iron pipe are nearly six times as great as those for the 2-in. brass pipe, hence 5/6 of the resistance in the cast iron pipe is due to the fittings.

One example will make the application of the curve formulas clear.

Given a 6-ft. stave pipe 5 miles long and an available head of 20 ft. The added length of all the curves, which are short but numerous, is 2 miles, the radius of all of them is 120 ft. What is the carrying capacity? In this case, since there are no fittings of any kind, the excess of the frictional resistance due to the curves per unit length of the conduit will be given by

$$Z_2 = Z_1\left[\left(2.2\left(\frac{6}{120}\right)^{\frac{3}{13}}\right) - 1\right]$$

For a 6-ft. stave pipe Table III gives $Z_1 = 0.00367$ ($m = 0.90$). Table IV for $\frac{R}{D} = 20$, the factor 0.102. This gives for $L = 10560$ ft. for the resistance due to the curves

$$Z_2 = \frac{0.00367 \times 0.102 \times 10560 \times 4}{6} = 2.635$$

Inserting all values we have for the prime velocity

$$V = \sqrt{\frac{64.4 \times 20}{1.505 + \left(0.00367\,\frac{26400 \times 4}{6}\right) + 2.635}}$$

$$= 4.33 \text{ ft. per sec.}$$

Hence $a = \sqrt[8]{4.33}$, $a^2 = \sqrt[4]{4.33} = 1.442$

Consequently, the discharge, inserting values

$$Q = 6.303\sqrt{\frac{20 \times 7776}{1.505D + \left(\frac{0.00367 \times 26400 \times 4}{1.442}\right) + 2.635D}} = 145.06 \text{ sec. ft.}$$

Omitting the resistances due to the entrance and the curves the formula gives $Q = 151.65$ sec.-ft. From the formula

$$Q = C_q\,(\sqrt[4]{0.25D} + m)\, D^{\frac{5}{2}} \sqrt{I}$$

$$I = \left[\frac{Q}{C_q\,(\sqrt[4]{0.25D} + m)\, D^{\frac{5}{2}}}\right]^2$$

$$D = \left[\frac{Q}{C_q\,(\sqrt[4]{0.25D} + m)\sqrt{I}}\right]^{\frac{2}{5}}$$

If, in the formula above, H the actual head in feet and L the length of the conduit in feet are substituted for I, the fall in feet per 1,000 ft., the formula will read

$$Q = \frac{C_q\,(\sqrt[4]{0.25D} + m)\, D^{\frac{5}{2}}\sqrt{H}}{\sqrt{\frac{L}{1000}}}$$

$$= C_q\,(\sqrt[4]{0.25D} + m)\, D^{\frac{5}{2}}\sqrt{\frac{1000H}{L}}$$

$$H = \left[\frac{Q}{C_q\,(\sqrt[4]{0.25D} + m)\, D^{\frac{5}{2}}}\right]^2 \frac{L}{1000}$$

This last form of the formula will prove useful in calculations relating to the distribution of water in irrigation systems. Let

L = the length of the section of a conduit from which water is distributed;

q = the quantity distributed;

Q = the quantity carried beyond the points of distribution;

n = the number of points equally distant from each other, at which the total quantity q is distributed;

then, the loss of head in the section L

$$H = \left[\frac{1}{Cq\,(\sqrt[4]{0.25D} + m)\, D^{\frac{5}{2}}}\right]^2 \frac{L}{1000}\left[q^2\left(\frac{1}{3} + \frac{1}{2n} + \frac{1}{6n^2}\right)\right] + 2qQ\left(\frac{1}{2} + \frac{1}{2n}\right) + Q^2$$

In case the number of points of distribution is very great the factors $\frac{1}{2n}$ and $\frac{1}{6n^2}$ may be omitted and the formula reads

$$H = \left[\frac{1}{Cq\,(\sqrt[4]{0.25D} + m)\, D^{\frac{5}{2}}}\right]^2 \frac{L}{1000}\left[\frac{q^2}{3} + qQ + Q^2\right]$$

Is moreover no quantity Q carried beyond

$$H = \left[\frac{1}{Cq\,(\sqrt[4]{0.25D} + m)\, D^{\frac{5}{2}}}\right]^2 \frac{L}{1000}\,\frac{q^2}{3}$$

One example will make the practical application of this formula clear.

Let it be required to distribute 30 sec.-ft. of water at six equally distant points, the section to be 5 miles long, the quantity carried beyond 50 sec.-ft. and the available head 24 ft. What will be the diameter of the conduit supposing it to be (1) of planed staves, (2) of asphalt coated riveted steel? In the first case Cq =

1.0, $m = 0.90$. Inserting these values, assuming for a first trial $D = 4$ ft.

$$D^5 = \frac{5 \times 5280}{24000\,(1 + 0.9)^2}\left[30^2\left[\frac{1}{3} + \frac{1}{12} + \frac{1}{216}\right] + 2 \times 30 \times 50 \times \left[\frac{1}{2} + \frac{1}{12}\right] + 50^2\right]$$

which gives $D^5 = 1410.6$, hence $D = 4.265$ ft. A second trial gives $D = 4.251$ ft. or 51 ins.

In the second case $Cq = 0.91$, $m = 0.53$. Hence $(0.91 \times 1.53)^2$ is substituted for $(1.9)^2$ in the above equation. The first trial gives $D = 4.83$ ft., the second 4.77 ft. or 57.25 ins.

In the first case the diameter of the conduit above the points of distribution will be

$$D = \left[\frac{80}{1.90\,\sqrt{0.909}}\right]^{\frac{2}{5}} - 4.55 \text{ ft.}$$

A second trial gives $D = 4.52$ ft. or 54.25 ins. For the steel conduit $D = 5.073 = 60.88$ ins.

OPEN CONDUITS.

The laws of flow which apply to circular conduits running full or under pressure hold in general also good for open conduits of all descriptions. The value of the coefficient C is in all cases equal to $66\,(\sqrt{R} + m)\,V_1^{x-1}$. For channels in earth however the coefficient m has a negative value and the formula is on this account somewhat defective for small channels of this class. The deficiency may be eliminated by the introduction of a new coefficient K, increasing in value with increasing roughness of the channel, such that

$$K = \frac{2}{1 + m} - 1$$

and the formula will then read, for channels

$$C = 66\,\frac{\sqrt{R} + \sqrt{R}}{\sqrt{R} + K}\,V_1^{x-1}$$

where $^{x-1}$ has a negative value.

The relation between this coefficient K and n, the coefficient of the Kutter formula, is given by

$$K = 100n - 1;\quad n = \frac{1 + K}{100}$$

A straight line formula may, however, be retained by putting, for channels in earth,

$$C = 66\,(\sqrt{R} - (\sqrt{R} + 1)\,0.5m)\,V_1^{x-1}$$

which gives very near the same values as the equation above.

For aqueducts and artifical channels in earth values of m range between $+1.0$ and -0.32, values of a between $V_1^{1/11}$ and $V_1^{-1/10}$.

$$a = V_1^{1/11}$$

holds good for semi-circular channels lined with neat cement, cement plaster, concrete and brickwork washed with cement and for both semi-circular and rectangular channels lined with plaster or rough boards

$a = V_1^{1/7}$ holds good in general for all trapezoidal and rectangular channels including those lined with good ashlar masonry. It holds good also for the channels of any sectional form lined with riveted metal plates. $a = 1.0$ holds good for channels lined with roughly hammered or rubble masonry and for channels in rock-work or cemented gravel. It holds good also for thos enumerated under $a = V_1^{1/7}$ in case the value of R exceeds 3 ft.

$a = V_1^{-1/10}$ holds good in general for all artificial channels in earth as far as practical velocities are concerned. For this class of conduits the value of the coefficient C increases with increasing velocity up to a certain critical velocity and then decreases. This critical velocity is the lower the rougher the conduit; for channels in earth it is below the velocity occurring in practice.

In the same manner as previously for the circular conduits we find from the general equation

$$V = 66\,(\sqrt{R} + m)\,\sqrt{RS}\,V_1^{x-1}$$

for $V_1 = 5$ ft per second the following equations:

$$a = V_1^{1/11},\ V = 2.416\,(\sqrt{R} + m)\,\sqrt{RI}$$

$$a = V_1^{1/7},\ V = 2.294\,(\sqrt{R} + m)\,\sqrt{RI}$$

$$a = 1.0\quad,\ V = 2.087\,(\sqrt{R} + m)\,\sqrt{RI}$$

$$a = V_1^{-1/10},\ V = 1.392\,(\sqrt{R} + m)\,\sqrt{RI}$$

In the latter case, for channels in earth, if M is the fall in feet per mile

$$V = 0.838\,(\sqrt{R} - 0.5m\,(\sqrt{R} + 1))\,\sqrt{RM}$$

The forms of the cross sections of open conduits are so diverse, that it is a great advantage to adopt some simple form as a standard and upon the flow in this base the flow in others. Since in the semi-square of unit depth the mean hydraulic radius is equal to half the depth, the depth equal to half the width and the velocity in feet per second equal to half the quantity of flow in second-feet this form of the section is well suited for such a standard.

Taking velocities and quantities of flow in the semi-square as a basis, velocities and quantities of flow in other rectangles and in trapezoids and semi-circles may be expressed in terms of those in the semi-square. The result will be a set of formulas of the form

$$V = FvC\,(\sqrt{0.5D} + m)\,\sqrt{DI}$$

$$Q = Fq\,2C\,(\sqrt{0.5D} + m)\,D^{\frac{3}{2}}\,\sqrt{I}$$

in which Fv and Fq are factors of relative velocities and quantities, C and $2C$ the constants for the semi-square. In all of these equations the actual value of R does not enter, the value taken is always that of the semi-

TABLE VI.—COEFFICIENTS OF VELOCITIES Cv AND COEFFICIENTS OF QUANTITIES Cq FOR OPEN CHANNELS.

Mean width.	$a = V_1^{1/11}$, Channels lined with clean cement or boards. Cv.	Cq.	$a = V_1^{1/7}$, Channels lined with concrete brickwork, or good ashlar. Cv.	Cq.	a = 1.0. Channels lined with rough ashlar or rubble. Channels brickwork. Cv.	Cq.
1 D......	1.305	1.305	1.239	1.239	1.125	1.125
1 5 D......	1.542	2.313	1.499	2.197	1 333	2.000
2 D......	1,710	3.420	1.622	3.244	1.476	2.952
2.5 D......	1.830	4.572	1.737	4.341	1.580	3.949
3 D......	1.934	5 802	1.836	5.509	1.670	5.011
3.5 D......	1.999	6.997	1.898	6.645	1.727	6.043
4 D......	2.072	8.289	1.968	7.872	1.790	7.058
4.5 D......	2.108	9.488	2.002	9 010	1.821	8.195
5 D......	2 156	10 78	2.047	10.235	1.864	9.311
6 D......	2.223	13.34	2 110	12.665	1.920	11.52
7 D......	2.279	15.95	2.164	15.155	1.969	13.78
8 D......	2.321	18.57	2.205	17.635	2.006	16.04
10 D......	2.387	23.87	2.267	22 670	2 062	20.62
Semi-circle	1.710	2.686	1.622	2.549	1.476	2.319

TABLE VII.—COEFFICIENTS OF VELOCITIES Cv AND QUANTITIES Cq FOR CHANNELS IN EARTH.

	$a = V_1^{-1/10}$ Coefficients for fall in feet per 1,000 feet.		Coefficients for fall in feet per mile.	
Mean width.	Cv.	Cq.	Cv.	Cq.
1 D..........	1.025	1 025	0.441	0.444
1.5 D..........	1.217	1.825	0.528	0.791
2 D..........	1.366	2.732	0.592	1.185
2 5 D..........	1 479	3.693	0.641	1 603
3 D..........	1.551	4 653	0.679	2.036
3 5 D..........	1 637	5.726	0 710	2.483
4 D..........	1.702	6.808	0.738	2.951
4 5 D..........	1 713	7 844	0.756	3.100
5 D..........	1 786	8.930	0 774	3.896
6 D..........	1.854	11.124	0.804	4.825
7 D..........	1 903	13 321	0 8249	5 753
8 D..........	1 941	15 552	0.8427	6.742
10 D..........	2 003	20 030	0 8661	8.687
12 D..........	2 047	24 164	0 8871	10 645
14 D..........	2 079	29 106	0.9011	12 620
16 D..........	2 104	33 664	0 9120	14.592
18 D..........	2 123	38 214	0 9202	16.561
20 D..........	2 140	42 800	0 9274	18.548
30 D..........	2 190	65 70	0 9492	28 476
40 D..........	2 214	88 56	0 9599	38.396

square of equal depth $= 0.5D$.

Tables VI and VII give the values of the constants resulting from the multiplication of the factors Fv and C, Fq and $2C$. For channels in earth for instance, in case the width of the channel is equal to four times the depth, the constants found in the table are 1.702 and 6.808. The corresponding equations are

$$V = 1.702\,(\sqrt{0.5D} - 0.5m\,(\sqrt{0.5D} + 1))\,\sqrt{DI}$$

$$Q = 6.808 \quad \text{''} \quad \sqrt{D^5 I}$$

$$\text{The term } \frac{\sqrt{0.5D} + \sqrt{0.5D}}{\sqrt{0.5D} + K} = \frac{\sqrt{0.5D} + \sqrt{0.5D}}{\sqrt{0.5D} + 100n - 1}$$

may be substituted for the straight line term $\sqrt{0.5D} \pm 0.5m$ for calculations of flow in channels in earth, rockwork, hard cemented gravel and rubble masonry. It gives fairly good results also for the other classes of open conduits.

TABLE VIII. — ROOTS AND POWERS OF DEPTHS OF WATER IN OPEN CONDUITS.

D in ft.	$\sqrt{0.5D}$	$\sqrt[4]{0.5D}$	$D^{5/2}$
I.			
0.05	0.158	0 398	0.000559
0.10	0.224	0.473	0.00316
0.15	0.274	0.523	0.00881
0.20	0.316	0.562	0.01789
0.25	0.354	0.595	0.03125
0.30	0.387	0.622	0.0493
0.35	0.418	0.647	0.0725
0.40	0.447	0.669	0.1012
0.45	0.44	0.689	0.1369
0.50	0.500	0.707	0.1768
0.55	0.524	0.724	0.2243
0.60	0.548	0.740	0.2788
0.65	0.570	0.755	0.3406
0.70	0.592	0.769	0.4100
0.75	0.612	0.783	0.4870
0.80	0.632	0.795	0.5724
0.85	0.652	0.808	0.6661
0.90	0.670	0.819	0.7687
0.95	0.689	0.830	0.8726
1.00	0.707	0.841	1.0
1.05	0.725	0.852	1.130
1.10	0.742	0.862	1.269
1.15	0.758	0.871	1.417
1.20	0.775	0.880	1.577
1.25	0.791	0.889	1.755
II.			
1.30	0.806	0.898	1.927
1.35	0.821	0.906	2.118
1.40	0.837	0.915	2.319
1.45	0.851	0.923	2.533
1.50	0.866	0 931	2.756
1.55	0.880	0.938	2.991
1.60	0.894	0.946	3.238
1.65	0 908	0.953	3.497
1.70	0 922	0.960	3.786
1 75	0.935	0.967	4.052
1.80	0.949	0.974	4.358
1.85	0.962	0.981	4.655
1.90	0.975	0.987	4.976
1.95	0.287	0.994	5.310
2.00	1.000	1.000	5.627
2.05	1.012	1.006	6.017
2 10	1.025	1.012	6.391
2.15	1.037	1.018	6.778
2.20	1.049	1.024	7.179
2 25	1.061	1.030	7 594
2 30	1.072	1.036	8.023
2.35	1.084	1.041	8.466
2.40	1.095	1.047	8.923
2 45	1.107	1.052	9.393
2.50	1.118	1.057	9.881
III.			
2.55	1.129	1.038	10.38
2.60	1.140	1.068	10 90
2.65	1.151	1.073	11.43
2.70	1.162	1.078	11.98
2.75	1.172	1.083	12.54
2.80	1.183	1.088	13 12
2 85	1.194	1.093	13.71
2.90	1.204	1.097	14.32
2.95	1.214	1.102	14.95
3.00	1.225	1.107	15 59
3.05	1.235	1.111	16.25
3.10	1.245	1.116	16.92
3.15	1.255	1.120	17.61
3.20	1.265	1.125	18 32
3.25	1.275	1.129	19.04
3.30	1.285	1.133	19.78
3.35	1.294	1.138	20.54
3.40	1.304	1.142	21 32
3.45	1.314	1.146	22.11
3.50	1.323	1.150	22.92
3.55	1.333	1.154	23.74
3.60	1.342	1.158	24 03
3.65	1.351	1.162	25.45
3.70	1.360	1.166	26.33
3.75	1.369	1.170	27.23
IV.			
3.80	1.378	1.174	28.05
3.85	1.388	1.178	29.08
3.90	1.397	1.182	30.04
3.95	1.405	1.185	31.01
4.00	1.414	1.189	32.00
4.05	1.423	1.193	33.01
4.10	1.432	1.197	34.04
4.15	1.441	1.200	35.08
4.20	1.449	1.204	36.15
4.25	1 458	1.207	37.23
4.30	1.466	1.211	38.44
4.35	1.475	1.214	39.47
4.40	1.483	1.217	40.61
4.45	1.492	1.221	41.77
4.50	1 504	1.225	42.96
4.55	1.508	1.228	44.16
4.60	1.517	1.232	45.39
4.65	1.525	1.235	46.60
4.70	1.533	1.238	47.89
4.75	1 541	1.242	49.17
4.80	1.550	1.245	50.48
4.85	1.557	1.248	51.80
4.90	1.565	1.251	53.14
4.95	1.573	1.254	54.52
5.00	1 581	1.257	55.90

		V.	
5.1	1.597	1.264	58.74
5.2	1.613	1.270	61.66
5.3	1.628	1.276	64.67
5.4	1.643	1.282	67.76
5.5	1.658	1.288	70 94
5.6	1.673	1.293	74.21
5.7	1.688	1.299	77.56
5.8	1.703	1.305	81.02
5.9	1.718	1.311	84.55
6.0	1.732	1.316	88.18
6.1	1.746	1.321	91.90
6.2	1.761	1.327	95.71
6.3	1.775	1.332	99.62
6.4	1.789	1.337	103.6
6.5	1.803	1.343	107.7
6.6	1.816	1.348	111.9
6.7	1.830	1.353	116.2
6.8	1.844	1.358	120.6
6.9	1.857	1.363	125.1
7.0	1.871	1.368	129.6
7.1	1.884	1.373	134 3
7.2	1.897	1.377	139.2
7.3	1.910	1.382	144.0
7.4	1.924	1.387	148.9
7.5	1.936	1.392	154.0
		VI.	
7.6	1.949	1.396	159.2
7.7	1.962	1.401	164.5
7.8	1.975	1.405	169.9
7.9	1.987	1.410	175.4
8.0	2.000	1.414	181.0
8.1	2.008	1.417	186.7
8.2	2.025	1.423	192.5
8.3	2.039	1.427	198.5
8.4	2.050	1.432	204.5
8.5	2.062	1.436	210.6
8.6	2.074	1.440	216.9
8.7	2.086	1.444	223.3
8.8	2.098	1.448	229.7
8.9	2.110	1.452	236.3
9 0	2.121	1.456	243.0
9.1	2.133	1.460	249.8
9.2	2.145	1.465	256.7
9.3	2.156	1.468	263.8
9.4	2.168	1.472	270.9
9.5	2.180	1.476	278.2
9.6	2.191	1.480	285.5
9.7	2.202	1.484	293.0
9.8	2.213	1.488	300.6
9.9	2.225	1.492	308.3
10.0	2.236	1.495	316.3
		VII.	
10.25	2.265	1.505	326.3
10.5	2.292	1.514	358.1
10.75	2.319	1.523	378.9
11.0	2.345	1.531	401.3
11.25	2.371	1.540	425.4
11.5	2.399	1.549	448.5
11.75	2.424	1.557	473.2
12.0	2.450	1.565	428.9
12.25	2.475	1.573	525.1
12.5	2.500	1 581	552.4
12.75	2.525	1.589	580.4
13.0	2.550	1.597	609.2
13.25	2.574	1.604	639.0
13.5	2.598	1.612	669.5
13 75	2.622	1.619	701.1
14.0	2.646	1.626	733.3
14.25	2.669	1.634	766.9
14.5	2.693	1.641	800.6
14.75	2.715	1.648	835.7
15.0	2.739	1 655	981.5
15.25	2.761	1.662	908.4
15.5	2.784	1.669	945.7
15.75	2.806	1.675	984.6
16.0	2.828	1.682	1024.0
16.25	2.850	1.688	1064.5
		VIII.	
16.5	2.872	1.695	1105.8
16.75	2.894	1.701	1148.2
17.0	2.916	1.707	1191.5
17 25	2.937	1.713	1235.8
17.5	2.958	1.720	1281.1
17.75	2.979	1.726	1321.3
18.0	3.000	1.732	1374.6
18.25	3.021	1.738	1422.8
18.5	3.041	1.744	1472.1
18.75	3.062	1.750	1522.3
19.0	3.082	1.756	1573.5
19.25	3.102	1.761	1625.8
19.5	3.123	1.767	1679.1
19.75	3.142	1.773	1733.5
20.0	3.163	1.779	1788.8
20.5	3.200	1.989	1902.8
21.0	3.240	1.800	2020.9
21.5	3.279	1.811	2143.3
22.0	3.317	1 821	2270.1
22.5	3.354	1.831	2401.3
23.0	3.391	1.841	2537.0
23.5	3.428	1.851	2677.1
24.0	3.464	1.861	2821.8
24 5	3.500	1.871	2971.1
25.0	3.535	1.880	3125.0

Values of $\sqrt{0.5D}$, $\sqrt[4]{0.5D}$ and $D^{\frac{5}{2}}$ are found in Table VIII. In channels in earth the side walls are more or less inclined and the mean width, that is the width of the channel at half the depth of the water is taken as a basis for calculations of quantities of flow. The area, bottom and top width of the section are given by

Area = mean width × depth;
Bottom width = mean width — D tan a;
Top width = mean width + D tan a, in which

tan a = the tan of the angle between sloping side and the vertical = the horizontal projection per foot of rise.

Velocities and quantities of flow in the rectangular section are either slightly less or slightly greater than those in the trapezoids of equal mean width. Table IX gives factors of relative flow corresponding to different side slopes and different mean widths. For a mean width = $6D$ and a side slope of 2:1 the table gives the factor 0.958. Hence for this side slope the constant corresponding to M.W. = $6D$ found in Table VII is to be multiplied by 0.958.

$$H=\left[\frac{1.257\ +\ 1.5}{1.027 \times 6.808\ (1.257+1.581)\ 55.90}\right]^{2}\frac{52800}{1000}\left[10000\left(\frac{1}{3}+\frac{1}{10}+\frac{1}{150}\right)+2\times 100\times 150\left(\frac{1}{2}+\frac{1}{10}\right)22500\right]$$

One example will make the application of the formulas clear. Let it be required to find the slope for a channel in earth of which the bottom width is to be 48 ft., the depth 12 ft., the inclination of the sides 2:1, the desired velocity 2.7 ft. per second and the value of K assumed = 1.5 ($n = 0.025$, $m = -0.20$). In this case the mean width is $8 + 12 \times 2 = 72$ ft., which is six times the depth.

For a relative mean width = $6D$, Table VIII gives the velocity constant 1.854, Table IX for a side slope of 2:1 the factor 0.958. For D = 12 ft., Table VIII gives $\sqrt[4]{0.5D} = 1{,}565$, $\sqrt{0.5D} = 2.45$. Inserting these values

$$I = \left[\frac{2.7\ \ (1.564\ +\ 1.5)}{(1.854\ \times\ 0.958)\ \ (1.564\ +\ 2.45)}\right]^{2}\frac{1}{12}$$

which gives $I = 0.1122$ ft. per 1,000, = 0.594 ft. per mile.

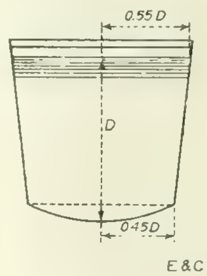


Fig. 1.

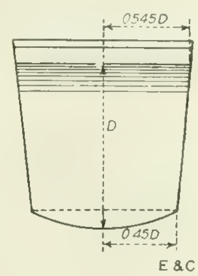


Fig. 2.

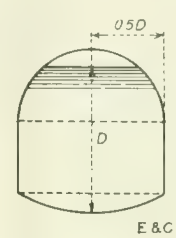


Fig. 3.

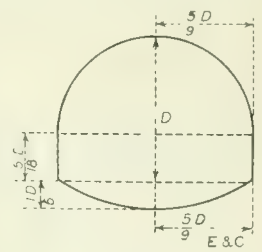


Fig. 4.

The equations for the distribution of water apply to channels in earth. Let it be required to find the slope for a channel in earth 10 miles long which is to carry 250 sec.-ft., of which 100 are to be distributed at five points equally distant from each other while the remaining 150 are to be carried beyond. The desired velocity is 2.5 ft. per second, the mean width of the channel about four times its depth, the side slopes 1:1 and the value of K is assumed = 1.50 ($m = -0.20$).

The area of the section will be equal to $\frac{250}{2.5} = 100$ sq. ft. Hence the depth of the channel, for a mean width = $4D$.

$$D = \sqrt{\frac{100}{4}} = 5 \text{ ft.}$$

For a mean width = $4D$ the quantity constant is 6.808 and the factor of relative flow for a side slope of 1:1 is 1.027. $D^{\frac{5}{2}} = 55.90$. Inserting these values which gives

$H = 13.8$ ft. or 1.38 ft. per mile.

In case no water is diverted $H = 22.8$ ft. or 2.28 ft. per mile.

III.—COVERED AQUEDUCTS AND TUNNELS.

For wooden flumes of moderate dimensions the most suitable forms of the section are the semi-square and the semi-circle. Only for those of great dimensions a section of greater relative width is indicated, as also for great bridge-aqueducts of masonry.

For covered masonry conduits considerations of economy demand the most compact forms, the circle, the square, or modifications and combination of these. The walls of concrete lined aqueducts in excavation are usually given a side slope of 1:10 to 1:12. The cover, for conduits not intended to be under pressure, is a plane slab, 0.5 to 1.0 ft. above the water line. The bottoms are inverted flat arches and the forms in use differ only in the relative radii given to those arches. In all forms the width of the section at the springing of the inverted arch is $0.9D$, at the water line it varies between 1.09 and 1.10 D, according to the side slope adopted, D being the depth of the water above the center of the inverted arch.

For the form Fig. 1 the depth of the arch at the center is equal to $D/19$, the mean hydraulic radius to $0.3447D$, the cross-section up to the water line to $0.9679\ D^2$.

For the form Fig. 2 the depth of the arch at the center is equal to $D/8.5$, the mean hydraulic radius to $0.3165D$, the cross-section up to the water line to $0.9204\ D^2$.

The sides of the tunnel Fig. 3 are vertical between the top and bottom arches and their height equal to $D/2$, the distance of the water line from the crown and the depth of the lower arch are equal to $D/8.5$. The radius of the upper arch is equal to $D/2$, the mean hydraulic radius to $0.3165D$ and the cross-section up to the water line to $0.9204\ D^2$.

The section Fig. 4 is usually chosen for conduits which occasionally run full or under a light pressure. In it the width of the vertical section is equal to $\frac{10}{9}D$ and its height to $\frac{5}{18}D$. The radius of the upper circle is equal to $\frac{5}{9}D$, the depth of the inverted arch to $\frac{1}{6}D$, the area of the section to $0.9086\ D^2$.

TABLE IX.—FACTORS OF RELATIVE VELOCITIES AND QUANTITIES OF FLOW.

Side slope.	Mean width 1 D	2 D	3 D	4 D	5 D	6 D	8 D	10 D
½:1	1.1482	1.0547	1.0465	1.036	1.029	1.0262	1.0208	1.017
1 :1	1.0925	1.033	1.029	1.027	1.019	1.0162	1.0130	1.012
1⅓:1	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
1½:1	......	0.978	0.984	0.986	0.990	0.991	0.992	0.994
2 :1	......	0.918	0.935	0.943	0.953	0.958	0.966	0.972
2½:1	......	......	0.857	0.900	0.915	0.924	0.938	0.948
3 :1	......	......	0.836	0.856	0.879	0.892	0.911	0.923

For the form Fig. 1

$$V = 1.347\ (0.7664\ \sqrt[4]{D} + m)\ \sqrt{DI}.$$

$$Q = 1.3036\ (0.7664\ \sqrt[4]{D} + m)\ \sqrt{D^5 I}$$

For the form Fig. 2 and the tunnel, Fig. 3,

$$V = 1.29\ (0.75\ \sqrt[4]{D} + m)\ \sqrt{DI}$$

$$Q = 1.188\ (0.75\ \sqrt[4]{D} + m)\ \sqrt{D^5 I}$$

For the form Fig. 4

$$V = 1.168\ (0.714\ \sqrt[4]{D} + m)\ \sqrt{DI}$$

$$Q = 1.061\ (0.714\ \sqrt[4]{D} + m)\ \sqrt{D^5 I}$$

For these conduits washed with cement $m = 0.88$ ($n = 0.0106$), plain concrete $m = 0.57$ ($n = 0.13$). For Los Angeles aqueduct $n = 0.14$, $m = 0.45$ was taken.

IV.—EGG-SHAPED CONDUITS.

In order to keep the velocity of flow for all quantities of water carried within certain chosen limits sewers over 2 ft. in diameter are usually given an egg-shaped section.

Three forms of such conduits are in use. In all of these the vertical diameter is equal to 1.5 times the diameter of the upper circle.

In the so-called old form the diameter of the lower circle is equal to ½, in the other forms equal to ¼ of that of the upper one. In the so-called peg-top form the horizontal diameters of the upper and lower circles are united by straight lines.

For the three forms mentioned the areas of the sections and the mean hydraulic radii are given by

Old form. Area $= 1.1[illegible] D^2$, $R = 0.2896\,D$
New form....Area $= 1.11503\,D^2$, $R = 0.2844\,D$
Peg top form. Area $= 1.03354\,D^2$, $R = 0.2680\,D$
D being the diameter of the upper circle.

The corresponding equations are as follows:

For the old form.. $V = 1.235\,(\sqrt{R} + m)\sqrt{DI}$

For the new form.. $V = 1.228\,(\sqrt{R} + m)\sqrt{DI}$

For the peg-top form.. $V = 1.188\,(\sqrt{R} + m)\sqrt{DI}$

For the old form $Q = 1.417\,(\sqrt{R} + m)\sqrt{D^5 I}$

For the new form. $Q = 1.365\,(\sqrt{R} + m)\sqrt{D^5 I}$

For the peg-top form... $Q = 1.228\,(\sqrt{R} + m)\sqrt{D^5 I}$

For these conduits.. $m = 0.57$

TABLE X.—VALUES OF THE COEFFICIENTS m and a FOR [illegible] CONDUITS

m.,	a.	Description of the conduit.
1.0	$V_1^{1/17}$	New semi-circular channels lined with polished cement.
0.95	$V_1^{1/17}$	New semi-circular or trapezoidal channels lined with neat cement or planed boards.
0.90	$V_1^{1/17}$	New rectangular channels lined with neat cement or planed boards.
0.85	$V_1^{1/17}$	New semi-circular or trapezoidal channels lined with brick or concrete masonry dressed with cement mortar or washed with cement.
0.80	$V_1^{1/17}$	New rectangular channels lined with brick or concrete masonry dressed with cement mortar or washed with cement.
0.70	$V_1^{1/17}$	Semi-circular channels lined with unplaned boards.
0.70	$V_1^{1/17}$	Rectangular channels lined with fairly good brickwork or fairly smooth concrete
0.60	$V_1^{1/17}$	Rectangular channels lined with unplaned boards.
0.57	$V_1^{1/17}$	Channels lined with common brickwork, concrete or smoothly dressed ashlar masonry. Sewers of all descriptions some time in use.
0.45	$V_1^{1/17}$	Channels lined with rough brickwork, rough concrete or fairly good ashlar masonry. Channels in earth roughly dressed with cement plaster.
[illegible]	1.0	Channels lined with common ashlar or good rubble masonry. Channels in earth roughly dressed with cement plaster, some time in use.
0.15	1.0	Channels lined with roughly hammered masonry.
0.0	1.0	Channels lined with common rubble masonry.

VALUES OF THE COEFFICIENTS m, K AND a FOR CHANNELS IN EARTH.

m	K	a	Description of channel.
0.57	0.27	$V_1^{1/17}$	Channels of very regular cross-section in stiff clay or clayey loam.
0.15	0.74	1.0	Channels of fairly regular cross-section in fine cemented gravel.
0.0	1.0	1.0	Channels of fairly regular cross-section in coarse cemented gravel or in rock-work.
—0.10	1.2	$V_1^{1/10}$	Fairly regular channels in sand or in sand with gravel imbedded. Channels as left by ditching machines.
—0.20	1.5	$V_1^{1/10}$	Fairly regular channels in loose soil, tolerably free from stones or plants.
—0.33	2.0	$V_1^{1/10}$	Ordinary channels in poor condition. Channels with stones, mud, vegetation or other impediments to flow. Channels as left by dredging machines. Somewhat irregular natural channels, creeks.
—0.43	2.5	$V_1^{1/10}$	Irregular channels in loose rivers, etc. cobble stones. Channels overgrown with weeds.

The governor of Arizona has suggested that convict labor on the highways of Cochise and Greenlee counties be stopped in order to give employment to some of the men who are out of work owing to the shutting down of many of the mines in that district.

WATER WORKS

A Method Successfully Employed in Putting Down Into Quicksand a Dug Well 10 Ft. in Diameter and 30 Ft. Deep.

[illegible] Editors: Mr. Dabney H. Maury's article entitled, Some Water Works Engineering Mistakes, published in ENGINEER[illegible] of Sept. 9, 1914, is very instructive and calls to my mind some similar experiences.

While constructing a water works system in New Jersey a sub-contract was made with a builder to erect the pumping station and to sink a suction well, and he was to be responsible for the entire construction and completion of the work. As he was a man who [illegible] rigid inspection was necessary.

The [illegible] well was 10 ft. [illegible] and [illegible] to a depth of 30 ft. from the surface of the ground, or about 25 ft. below the base of the pumping engines, and was to be supplied by syphoning from five artesian wells some 100 ft. deep.

It was suggested to the contractor that he sink the suction well first, but he preferred to do it the other way and no positive protest was made. The well was sunk in the usual manner with a wooden curb on which the circular brick wall was laid and the material from the inside excavated.

After the contractor had reached a depth [illegible] caused great trouble; and after three days' work, in which no progress was made and the caving became so bad that there was some [illegible] pumping station, he appealed to the writer who made a personal examination and found conditions serious.

Various plans were thought of: One being to sink a series of driven wells around the [illegible] ground water down, but the sand was so fine that this would have been difficult. Another plan was to force cement grout into these wells and let it set for some time before resuming the suction well work. Finally a very simple plan was devised, which proved to be eminently successful.

The top of the well was loaded down with pig lead, so as to take up small space, and a sand centrifugal pump was provided. The water in the well was maintained about a foot above the surface of the ground, while the pumping continued, keeping the open suction close to the bottom by means of a rope, and the discharge from the pump, which contained about 6 per cent of sand, was delivered into a box which overflowed back into the well and the sand was removed as fast as it accumulated. In order to keep the bottom from packing, a hose stream was played around the suction of the pump. In this way the sinking of the well progressed rapidly. Of course, under these conditions no caving could take place. Finally a boulder some 3 ft. in diameter was encountered, but this was readily kept within the well until it had reached full depth.

Finally it was necessary to place a bottom in the well. For this purpose 10-in. maple planks were cut to the right dimensions, and a diver was employed to place them. His first duty was to grapple with hooks the large boulder, which was raised by a block and fall, and he succeeded in putting in the planks within a few hours, working from each side toward the center. The well was then pumped out and everything was found to be in first-class condition.

This plan would have been applicable even if the well had gone to a much greater depth, but it might be impracticable in that case to have a diver put in the bottom. This, however, could be done by using a rich mixture of concrete and depositing it under water by means of a chute; 2 or 3 ft. ought to be sufficient for the purpose.

Since that time the writer has sunk a considerable number of iron wells 4 to 6 ft. in diameter by the same method.

Mr. Maury's experience with terra cotta pipe is common and the writer agrees with his statements and would suggest that if the pres[illegible]t great (say 30 or 40 ft.), and a [illegible] is required, the best way is to make the joints tight by caulking with tar hemp and testing not more than 1,000 ft. at a time under the maximum pressure before any cement is put in the joints. Then the joints should be cemented carefully, paying particular attention to the bottom, where leaks invariably take place. When each section is finished this way, the trench should be immediately back-filled.

Very truly yours,

J. W. LEDOUX, Chief Engineer.

American Pipe and Construction Co.

Philadelphia, Pa., Sept. 12, 1914.

(The foregoing letter, like Mr. Maury's article to which our correspondent refers, is a contribution to water works literature of the highest type of usefulness. We shall be glad to receive for publication similar statements of experiences from other water works engineers.—Editors.)

Caulking Lead Joints With Compressed Air at Waltham, Mass.

During the year 1913 the city of Waltham, Mass., laid about 10,000 ft. of 10-in. water pipe. The portable air compressor for caulking the joints appealed to the superintendent because of its portability, its light weight and its sufficient power. The complete air plant cost somewhat less than $450 and practically paid for itself on this one job. So successfully did this machine do its work that the superintendent of water works, Mr. Daniel J. Higgins, is convinced that the air compressor for caulking lead joints and for rock drilling has passed the experimental stage in water works construction. The following description of the application of this method at Waltham is from Mr. Higgins' paper before the annual convention of the New England Water Works Association:

The machine purchased was directly connected with a single cylinder, four cycle gasoline engine to the air compressor, pumping directly into a vertical tank placed on the same channel iron bed. On top of the tank was placed a safety valve and also an air valve. Beyond the air valve there was a slip joint and a lock coupling for the air hose. We had 50 ft. of hose and after placing the machine on the street side of the ditch, opposite the side where the gravel was to be thrown out, we found, by placing the machine at any point 50 ft. from where we started, that we could caulk 100 ft. without moving the machine. That is, we ran 50 ft. back of the machine and then 50 ft. ahead of it. This put in practically eight joints. In one 5,000-ft. section we ran across about 1,000 ft. of ledge and rather than call in a contractor, or drilling the ledge out by hand, I decided to buy an air drill. This cost about $68 with a set of drills, and we removed the 1,000 ft. of ledge. We found this was a very good investment, as we have used the machine many times since then for removing ledge and have also found it profitable to transport it to various points in the city to remove large boulders when encountered in the trenches.

In caulking we used the best Omaha lead. Instead of a straight ell we used a beveled one, because it was necessary to have more lead protruding slightly beyond the bell end when caulking with the machine than when caulking by hand. This would seem to indicate that the lead was more firmly forced into the joint by the machine, and we have never split a bell. The length of time necessary to caulk a joint was nominally about three minutes to the joint, as compared with 15 minutes by hand tools. Our first caulking tools were made a trifle long, and after experimenting with them we decided that a short tool was more practicable.

At first our caulking gang was a bit timid in using the machine. They were under the impression that it was going to cheat them out of a job, but after a few demonstrations, they realized the machine was a powerful aid to them, and later realized the wonderful power behind the blow of the air hammer. We found that under-side caulking was made much easier by the use of the machine on account of the hard position and reaching that caulkers had to assume in order to do satisfactory work by hand methods. The machine works just as well underneath the joint as on the sides and top where access is much easier.

In my first crew a fairly intelligent man was delegated to run the machine and also to lend a hand in setting the pipe. I found at first that this man was addicted to the very serious habit of using a screw driver and monkey wrench on the machine, and would not do as he was told. This lasted several days and we finally had to let him go. A high school boy, formerly water-boy for the gang, was given the job of running the machine with the same instructions given the former man. He proved a complete success and followed instructions to the letter, and we then had much better results than at first.

The portable air compressor differs from all other plants of this kind. The engine and the compressor are combined in one machine. The air piston is connected on the same crank shaft as the engine piston, making what is known as a double throw method which gives absolutely the same speed and power to the compressor as the engine. Another improvement is the piston discharge valve instead of the old style stem-valve, which makes it possible to reduce the valve space behind the air piston to a minimum. This valve also increases the efficiency about 15 per cent and is practically indestructible. The compressor is also equipped with an unloader which automatically relieves the compressor at any desired pressure up to 125 lbs. The engine is equipped with a magneto which makes the use of batteries unnecessary. The gasoline supply is retained in the base of the engine.

The requirements of a properly caulked joint involve a rather tedious and slow operation when the work is performed by hand. In addition it is expensive and lacks uniformity and reliability. This is most noticeable on the under side of the joint due to its inaccessibility. The pneumatic hammer gives an absolutely uniform joint on top and bottom.

The specifications of the compressor used are as follows:

Engine, 5 HP., hopper water cooled.
Compressor, 4½x6 ins. air cooled.
Capacity, 23 cu. ft. free air per minute.
Size of air tank, 20x60 ins. or 30x60 ins.
Total weight, with 20-in. tank, 1,650 lbs.; with 30-in. tank, 1,800 lbs.
Total length, 6 ft.
Total width, 34 ins.
Pneumatic tool capacity, with 20x60-in. tank, 1 pneumatic caulking tool or 1 pneumatic rock drill; with 30x60-in. tank, 2 pneumatic caulking tools or 1 rock drill.

The cost of the outfit is $436.60 complete with a caulking hammer for caulking pipe, cutting or chipping bricks or concrete and a set of 6 steels and air hose in 50-ft. lengths. We bought an Imperial air hammer for rock drill work for $60 additional.

Specifications Governing Purchase of Water Meters of Disc Type at San Diego, Cal.

The department of water of San Diego, Cal., has recently printed a 50 page pamphlet containing its standard plans and specifications. These plans and specifications relate to the materials used in the distribution system and to all the appurtenances of the distribution system. The specifications also cover workmanship and construction methods. We here quote the specifications governing the purchase of water meters of the disc type:

The meter shall consist of four units, as follows: 1, measuring chamber; 2, gear chamber; 3, registering device; 4, meter body.

Measuring Chamber.—The measuring chamber shall be made in two parts, and be so arranged in the meter that the meter may be disassembled completely without taking the body from the service line upon which it is attached. In testing the meters they shall be guaranteed to register not over 100 per cent on the full flow, or the 1-32 opening, and they shall be guaranteed not to fall below 98½ per cent on the full flow and not less than 96 per cent accuracy on the 1-32 opening at a pressure of 38 pounds per square inch.

Disc.—The disc shall be made of the best class of hard rubber with a specific gravity as near as possible to that of water, and shall either have some means of relieving the pressure or shall be well reinforced by means of an aluminum or otherwise approved reinforcement through the center of the disc. It shall have a disc not less than 2 15-16 ins. in diameter for the ⅝-in. by ¾-in. size, and shall be guaranteed to give a capacity of not less than 20 gals. per minute with a full opening and with a pressure of 38 lbs.

The disc spindle shall be made of phosphor bronze and the disc chamber shall be made of the following mixture, unles otherwise ordered: Copper, 87.12 per cent; zinc, 5.44 per cent; tin, 5.44 per cent; lead, 2.00 per cent.

Strainer.—The strainer shall be made with an opening equivalent to not less than two times the diameter of the inlet pipe, and shall be made of tinned copper and be so located that it may be easily removed.

Gear Trains.—The gear trains shall be strong, simple and durable and so constructed that they are easily removable, and when so ordered shall be made of the class of hard rubber adapted to this class of work.

The spindles shall be made of either phosphor bronze or some material meeting with the approval of the hydraulic engineer. A consideration in awarding the contract for the meters will be given to such items of construction as rubber bearings, jeweled bearings in the meters, gears having double hung bearings, simplicity of gearing and wide faced gearing easily operated.

Where it is necessary to bolt one bearing plate upon another the screw shall either be made of phosphor bronze or some metal meeting the approval of the engineer. The bearing cam shall be screwed and fastened in an approved manner. The gearing and bearings, unless otherwise approved, shall be constructed of bronze mixed from new metal of the following composition: Copper, 87.12 per cent; zinc, 5.44 per cent; tin, 5.44 per cent; lead, 2.00 per cent.

Registering Device. — The registering device shall be separated from the gear train chamber by means of an adequate stuffing box, guaranteed to be water tight under ordinary working pressures. The teeth of the gear train must be made of some non-corrosive metal, and preference will be given to such type of trains as work in rubber bushing.

There shall be dials gaged for reading the capacity of meters in cubic feet, and the sum of the dials must give 100,000 cu. ft.

The top and bottom of the registering device shall be a brass plate not less than 1-16-in. in diameter. The registering dial shall consist of a copper plate enameled in a first-class white enamel. Straight reading dials of approved design may be substituted at option of the hydraulic engineer.

Meter Body.—The meter body shall be made of bronze, and the maker shall submit the proportions of metal he proposes to put into the same. It shall be made in two or more parts: the lower part, holding the measuring chamber, shall be bolted to the upper part, or gear chamber, by four or more bronze studs having a diameter of not less than 5-16-in. and with hexagonal heads, or may be bolted by through bolts having a square head. These bolts shall be cinched by means of hexagonal bronze nuts operated from the top of the bolt.

The casing will be subjected to a pressure of 250 lbs. water pressure per square inch, and if found to leak it will be condemned. The inlet opening and outlet opening through the meter shall be ⅝-in. in diameter, and threaded so as to take a ¾-in. bronze hexagonal coupling, threaded for a ¾-in. pipe. These couplings are to be included in the bid for meters and are to be made of the same composition as the meter body.

There shall be an adequate bronze lid on top of the meter to protect the reading dial, so designed, unless otherwise ordered, that when opened it cannot go beyond 3 degrees past a vertical position. The serial number of the meter shall be designated in good bold figures on the surface of the lid.

All meters shall be drilled through the top portion of the gear in some adequate manner for sealing purposes.

In submitting bids for the various makes of meters the makers will also submit bids for the following spare parts: Measuring chambers, meter discs, gear trains and registration devices. The city reserves the right to make from time to time chemical analysis of the metals entering into the various parts of the meters and should same vary from the specifications the complete lot of meters are subject to rejection without any cost to the city of San Diego, and the city may annul the contract for same.

Mitigation of Destructive Effects of Electrolysis on Reinforced Concrete, Underground Pipes and Cable Sheaths.

The present article gives an account of the work of the Bureau of Standards on the subject of the destructive effects of electric current on reinforced concrete, underground pipes, and cable sheaths, and of the mitigation of these effects. The investigations of the Bureau of Standards on the general subject of the damage to underground pipes, cable sheaths and reinforced concrete by electric railway current and soil corrosion have been carried on during the past four years.

The attention of the Bureau of Standards was first called to the importance of a study of the effects of electric current upon reinforced concrete by letters from engineers and companies seeking information as to the extent of the damage produced and the most practicable means of preventing it. Laboratory experiments had shown that under certain circumstances an electric current flowing from the reinforcing material into the concrete not only caused serious corrosion of the reinforcing metal, but sometimes also split open considerable blocks of solid concrete. These experiments had been repeated and confirmed by numerous experimenters, but theories to account for the phenomenon were conflicting and opinions as to the extent of the danger in practice were very diverse.

The corrosion of gas and water pipes by electric currents flowing from street railway tracks through the earth back to the powerhouses was another phenomenon of electrolysis that for many years had been causing serious concern to the gas and water companies, and in more recent years at least to railway engineers also. It was recognized at the Bureau that a thorough investigation of these questions, for the purpose, first, of understanding the phenomena better and, second, learning the best methods of overcoming the trouble, would be of great practical value, and it was believed that such a problem requiring years of consecutive observation and research, could be undertaken to advantage by a national institution, such as the Bureau of Standards. Accordingly, a special appropriation was asked for and granted by Congress, and work was begun in the summer of 1910. A number of related investigations have been carried out during the past four years, and some are still in progress.

The lessons drawn, to date, from these investigations are here recorded as given by Edward B. Rosa, Chief Physicist of the Bureau of Standards, in his paper before the annual convention of the New England Water Works.

CORROSION OF REINFORCED CONCRETE.

The investigation of the effects of electric current upon reinforced concrete was one of the first to be taken up. At the outset a number of experiments described by previous investigators were repeated in order to verify their results or to decide between conflicting conclusions. Numerous modifications of these experiments, and many entirely new ones were instituted, and a very careful record was kept of all details of the work, so that conditions could be accurately reproduced. The conditions under which an electric current can split open a solid block of reinforced concrete were investigated and the cause of this important and somewhat startling phenomenon was determined. It was also found that when an electric current flows through reinforced concrete toward the core that there is an important cathode effect, such that the bond between the concrete and the reinforcing metal may be destroyed and the concrete cease to be reinforced. In addition to ascertaining the chemical and physical changes which occur and clearing up certain hitherto unexplained phenomena, some practical conclusionss of great economic value resulted from the Bureau's work. I cannot take time even to mention the principal results obtained, but can merely refer anyone who is interested to our technological publication, No. 18, an illustrated paper of 136 pages, which will be sent free on request.

ELECTROLYTIC CORROSION OF IRON IN SOILS.

A second investigation was on electrolytic corrosion of iron in soils, a subject of great practical importance because of the enormous quantity of iron gas and water pipe buried in the ground. For convenience we use the term electrolytic corrosion to indicate the corrosion caused by the discharge of an electric current which enters the metal from external sources. Other forms of corrosion, in which the currents originate in the corroding system from whatever cause, are designated as self-corrosion. These two forms of corrosion are not independent of one another, since the presence of either kind of corrosion generally affects the extent of the other. This mutual influence is of great importance, and must be carefully taken account of in electrolysis surveys. It is partly because of this effect that corrosion efficiencies greater than 100 per cent, sometimes as much as 150 per cent, are encountered; that is, the amount of the corrosion due to a given electric current may be as much as one and a half times as great as the amount calculated from Faraday's law, after deducting the natural corrosion that would have occurred if there had been no external current flowing. These cases occur with low current densities. With high current densities the efficiency of corrosion is relatively low, the iron tending to become passive, and showing corrosion efficiencies sometimes as low as 20 per cent. This is in marked contrast to the effect in concrete, where at high current densities the destructive effects increase more rapidly than the current increases.

Moisture has a marked effect, the efficiency of corrosion increasing with the moisture content of the soil up to saturation.

Temperature changes within ordinary ranges have no important effect upon the efficiency of corrosion. Temperature does not exert an important influence in the total corrosion in practice by affecting the resistance of the soil.

The amount of oxygen present has a marked effect upon the end products of corrosion. If the corrosion is rapid and the supply of oxygen is limited, there will be a preponderance of magnetic oxid; if the rate of corrosion is slow and the supply of oxygen is plentiful, ferric oxid will predominate. Thus the nature of the oxide is an indication whether the corrosion is due to external electric currents, as rapid self-corrosion is unusual unless in the presence of cinders, coke or chemicals.

Careful experiments were made to ascertain whether the efficiency of corrosion is different in different kinds of iron. Eighty specimens were employed, 20 each of ingot iron, machinery steel, wrought iron, and cast iron, half of the latter being used as they came from the mold and half machined to a clean surface. The results showed almost exactly the same corrosion due to the current, although the self-corrosion of the cast iron was greater than for the others, being about 10 per cent of the electrical loss. (The self-corrosion of the ingot iron was 3 per cent; of the machinery steel 4 per cent, of the wrought iron 5 per cent.)

The superiority of cast iron respecting electrolytic and self-corrosion is due to higher resistance of the joints in the pipe lines, higher electrical resistivity of the iron, heavier walls of the pipe, and a tendency to corrode more uniformly.

Other subjects investigated were the causes of the efficiency of corrosion exceeding 100 per cent; the corrosion efficiency at very low voltage, and the effect of moisture content, temperature and pressure on earth resistance. It was found that the corrosion efficiency does not vary with voltage, the current being the same. Impressed voltages as low as 0.1 volt gave the same values as 1.0 volt. Corrosion tests on a large number of different kinds of soil from widely different sources with average moisture content and moderate current densities showed that corrosion efficiencies between 50 and 110 per cent may usually be expected under most practical conditions.

ELECTRICAL RESISTANCE OF SOILS.

The resistance of soils varies widely according to the moisture content, the resistance of comparatively dry soil being several hundred times the resistance of the same soil at saturation. Hence voltage surveys should not be made at times when the earth is exceptionally wet or exceptionally dry.

The resistance of the soil varies greatly with the temperature. At —18°C., the resistance of soil was found to be over 200 times as great as at +18°C. At freezing temperature the resistance is many times the value at 18°C. This has an important bearing on the magnitude of electrolysis troubles, and shows that the damage from electrolysis due to street railway currents may be less in winter than in summer, even though the railway currents are then greatest. The rail resistance is least in winter when the soil resistance is greatest, both tending to lessen the current in the pipes. Electrolysis surveys therefore would better not be made in severely cold weather.

These results have an important bearing on electrolysis mitigation, and the fixing of limiting values to the current density in rails or voltage gradients in the earth, as well as an important bearing on the methods employed in electrolysis surveys.

Nearly a hundred samples of soil were collected from various cities, including Philadelphia, Pittsburgh, Erie, St. Louis and Washington, and their resistances measured. The soil samples of each city were from widely different locations, being taken from the depth at which pipes were buried, and generally near gas or water pipes. The resistances varied through a wide range, according to the character of the sample, but the averages showed some interesting results. The values were much more uniform and lower in the St. Louis samples than in those from other cities, the smallest being 400 ohms per cubic centimeter, and the largest 1800 ohms. The average resistance of 35 samples of St. Louis soil was five times less than of the 20 samples of Philadelphia soil.

These results also have an important bearing on the question of damage from electrolysis and the precautions necessary to protect gas and water pipes and cable sheaths from destruction by street railway currents. The Bureau has made electrolysis surveys in Philadelphia and St. Louis, and we had wondered why the former city was relatively so free from trouble due to electrolysis. There were differences in conditions in favor of Philadelphia in several particulars, but not until we found the great difference in soil resistance did we understand the great difference in electrolysis troubles.

The proposal to fix by state law a general requirement as to voltage drop or current density in the rails is seen not to be very logical, in view of the great variations in the damage which can result under different conditions with the same voltage drops. The full results of this investigation on electrolytic corrosion of iron in soils are contained in our Technological paper No. 25. The methods employed in measuring the resistance of soils and the results obtained will be published shortly in Technological paper No. 26.

SURFACE INSULATION OF PIPES.

The fourth investigation in this series undertaken by the Bureau was on the surface insulation of pipes as a means of preventing electrolysis.

One of the first methods resorted to as a means of preventing damage to buried gas and water pipes by electrolysis and soil corrosion, and still used to a considerable extent, consists in covering the surface of the pipes with a coating intended to insulate them from the surrounding earth. In many cases pipes have been so protected from soil corrosion; but it is doubtful whether there is any instance on record where damage by electrolysis has been effectually prevented in this way if the voltage conditions were at all severe. On the contrary, there have been cases where such coatings have done actual harm, as they tend to fail in spots, and so permit the current to escape from the pipes at such places with a greater current density than otherwise, and so aggravate the local damage.

To obtain reliable information as to the practicability of protecting pipes from electrolytic corrosion by means of paints or wrappings the Bureau undertook an extensive investigation of all the materials on the market for the purpose. The materials available may be divided into three classes as follows:

(1) Paints or compounds applied at ordinary temperatures.

(2) Dips or compounds to be applied hot, such as asphalts, coal tar pitches, etc.

(3) Wrappings, consisting of alternate layers of compound and fabrics.

(In addition, experiments are in progress on fiber conduits used with or without pitch filling and made up with screw joints, and pipes with enamel coatings.)

Time does not permit me to describe the experiments carried out through a period of several years, partly in the laboratory and partly on specimens buried in the earth, but I can give briefly the conclusion, namely, that the paints, dips and wrappings tested were found to be of little or no value in protecting pipes from electrolysis when applied to the positive areas near power stations where the current is tending to leave the pipes. Repeated experiments have shown that all the materials tested increase in weight when immersed in water. Hence they absorb moisture in greater or less degree when buried in the soil, and so become slightly conducting. Experiments show that before breaking down they begin to transmit current. This small current generates more or less gas, which soon develops pressure enough to rupture the coating. In the anode specimens, rust spots appear at the places where failure occurs, and pits then rapidly develop. In cathode specimens gas breaks through the insulating films, although no rust occurs.

These coatings would be of greater value in negative areas, where the current is flowing toward the pipes, but in practice it is not possible to foresee what are to be positive and what negative areas, for changes may at any time be made as the railway distribution system is enlarged or altered. Where there are no appreciable earth currents, and protection against self-corrosion is desired, these coatings and wrappings may be used.

After very thorough and careful study of the problem with the co-operation of many manufacturers of the materials tested, the Bureau has come to the conclusion that money can be expended more profitably in other directions than in an attempt to insulate gas and water pipes from the earth. The full results of this investigation are contained in our recent technological paper No. 15.

INSULATING JOINTS.

A fifth investigation in this series is on insulating joints and their effectiveness in reducing trouble due to electrolysis. There are great numbers of insulating joints of various kinds in use in gas pipe lines, the most common being cement joints. This was one of the first investigations made at the Bureau, and consisted in testing the mechanical strength of joints, measuring the resistance between adjacent lengths of pipes connected by such joints, determining the corrosion around such joints when a given difference of potential is maintained between adjacent lengths of pipes, or a given voltage applied to a line containing many such joints; finding the amount by which the current is reduced when such joints are used as compared with the same pipe in the same soil with lead joints, and making field studies where cement joints and other kinds of insulating joints are used, including the methods of making joints in practice.

I cannot discuss the subject farther than to say that we have found insulating joints of very great value in reducing the current in pipe lines, and so reducing the corrosion caused by such current. Where the potential gradients are high the joints must, of course, not be placed too far apart, as in that case the potential difference at each joint would be sufficient to cause considerable joint electrolysis. There is still some difference of opinion among engineers as to the practical value of insulating joints, but our experience is that while they should not be depended upon as the sole means of protection to pipes, they are economically and mechanically practicable, and are a very important auxiliary means of protection, particularly in the case of gas pipes. They are less often used in water pipe lines.

METHODS OF MAKING ELECTROLYSIS SURVEYS.

Another subject to which we have given much attention, and which will shortly be discussed in one of our Bureau publications, is the methods of making electrolysis surveys. One of the objects of such surveys is to determine the overall potential differences between different parts of a city, due usually to the street railway system where a grounded return is used, as in the overhead single trolley system. When the negative bus bars of the power supply stations and substations are grounded, and negative cables are run out to the grounded tracks, there is established a region of low potential around the station, and large potential differences are often produced in the systems of gas and water pipes, lead cable sheaths, reinforced concrete and any other conducting structures embedded in the earth. In consequence, electric currents of considerable magnitude flow through the earth from tracks to pipes at points more or less distant from the stations and from pipes back to tracks or negative cables in the region near the power stations. It is in the regions near these stations where currents sometimes of hundreds of amperes are leaving the pipes and cables that electrolysis is most severe. It is the purpose of these surveys, therefore, to determine not only the overall potential differences, but also the differences of potentials between tracks and pipes, the current density and potential gradients in the rails of the tracks, whether the tracks are well bonded, and the location of defective joints, the approximate current flowing in the pipes wherever that can be conveniently done, and especially where the damage from the electrolysis is most serious. The engineers of the Bureau have acquired considerable experience in such work. They employ methods which can be used by any engineer.

METHODS OF ELECTROLYSIS MITIGATION—TRACK DRAINAGE AND PIPE DRAINAGE.

Having made an electrolysis survey in any given place and learned the condition of the railway and pipe systems, the next question usually is how to remedy the trouble. As already indicated, we do not believe it practicable to insulate the pipe systems from the earth, nor to prevent serious corrosion to the pipes if heavy electric currents are allowed to flow into and out of them again. There remains, therefore, two possibilities. First, to bring the current back to the stations by metallic conductors provided for the purpose in such a way as to keep the current out of the pipes almost entirely; or, second, to allow the current to get into the pipes and cable sheaths but provide metallic conductors for getting out again without serious corrosion to the pipes. In other words, drain the current out of the tracks by insulated negative cables, keeping the tracks as nearly at the normal earth potential as possible, and insulate the negative bus bars at the stations, so they may be kept at any desired potential difference below the earth; or, drain the current out of the pipes by means of copper cables tapped into the pipes, the bus bars being grounded. These two systems may be briefly designated as the Track Drainage System and the Pipe Drainage System, respectively.

No one will question that the first of these methods is to be preferred, if it can be carried out economically. By it we correct the evil at its source, and it is undoubtedly simpler in theory and on many accounts better in practice. It has, however, been thought by many to be prohibitive in cost, and has been very little used in American cities. The pipe drainage system, on the contrary, has been strongly advocated and in one form or another much used. The investigations of the Bureau of Standards, however, which have been extended over several years, and have included a considerable number of large and small cities, have convinced us that the pipe drainage system as a principal method of protection is not only complicated and expensive to install and maintain, but is in the long run not an adequate remedy for electrolysis troubles. On the other hand, the track drainage system when properly designed and installed is found to be simpler, and more effective and far less expensive than has been supposed.

The difference between the track drainage system with insulated negative feeders and the ordinary method of reinforcing the conductance of the rails by means of bare copper wires is very fundamental. In the latter, the copper and tracks must have a potential gradient sufficient to cause the current to flow back to the station, and if this is to be small enough to obviate electrolysis troubles (say 0.3 or 0.4 volt per 1,000 ft. as a 24-hour average, equivalent to about 1 volt per 1,000 ft. at peak load), the joint conductance of track and copper cables must be very high; that is, a very large quantity of copper must be used and this generally makes the cost prohibitive. On the other hand, if the negative cables and negative bus are insulated (except at the end connected to the track) the drop of potential over the cables can be such as to give maximum economy. That is, the quantity of copper can be so adjusted that the loss of power per annum in the negative return and the annual charges on the cables shall be approximately equal; in other words, the sum of the loss of power in the negative return per annum and the annual charges on the negative cables may be a minimum. This may require a drop three or four times as much on the insulated cables as would be permissible on the tracks, and the negative bus bar may be 10 or 20 volts, or whatever is necessary, below earth potentials. Resistance taps would be inserted in the shorter cables, so that the drop in potential in the various feeders shall be nearly equal. For the sake of economy it is usual to have a smaller drop in the shorter feeders than in the longer, and so have a potential gradient on the tracks toward the power house. But this should be kept small so as to keep down the tendency for the current to leave the tracks and return by way of the pipes.

I should like to describe more in detail, if time were available, the results the Bureau of Standards has obtained in applying insulated negative feeders to electric railway systems for the purpose of reducing electrolysis troubles. Our work has included both large and small systems, urban and interurban, and the results obtained as to economies that accompany and more or less offset the cost of electrolysis mitigation have sometimes surprised railway officials.

So far I have said but little concerning the pipe drainage system, which has been used in this country more than any other system. Connecting the pipes in the positive areas to the tracks by heavy cables lessens the damage, but it greatly increases the current in the pipes, and this is a serious objection. Where insulating joints are in use, and wherever defective joints occur, it tends to increase the joint electrolysis. In repairing gas pipes it is necessary to put a jumper cable around a proposed break in the pipe in order to avoid the possibility of causing an explosion by the spark or arc when the current in the pipe, which may be heavy, is broken. Such drainage cables tend to lower the potential of the pipes drained, and so attracts current to them from other pipes and cables, thereby making them a source of danger to all other pipes not drained. To completely drain all the pipes in the complex network of a city is almost impossible, and to keep track of the changes in the railway system and readjust these drainage cables when necessary, would be an immense undertaking. The system has rarely been carried into effect on a large scale, but has been used to a greater or less degree in many places. We have reason to believe that in many places where pipe drainage has been practiced that serious damage from electrolysis is becoming evident.

SURVEYS MADE BY THE BUREAU OF STANDARDS.

The Bureau of Standards has made electrolysis surveys for St. Louis, Springfield, Ohio, and Springfield, Mass., and several other places, and has found in a number of cases that the economies of power effected in connection with the improvement in the roadway, the shortening of feeding distances and provision of a proper return for the current has been sufficient to yield a fair income on the cost of the changes and new construction. In St. Louis the United Railways Co. equipped a new station in accordance with plans worked out by the Bureau of Standards and found that the results attained with respect to cost and electrolysis protection were in accordance with the advance calculations made by the Bureau. The Ann Avenue station afforded an excellent opportunity to test the theory of the insulated return system and make a comparison with the pipe drainage system. In this station the negative cables were used insulated with the negative bus bar insulated from earth, and readings were taken on the potential drops and also on the current in 21 gas and water pipes. Then the bus bar was grounded and the cables used as though they were bare copper, and again the potential differences were measured, as well as the current in the pipes. It was found that with the insulated feeders the total current in the pipes was 65 amperes, whereas with the cables uninsulated and so a steeper slope of potential in the earth, the total was 273 amperes, more than four times as much.

While no attempt was made in this investigation [illegible] a complete pipe drainage system, some of the pipes near the station were temporarily tied to the tracks in order to determine the effect of so doing on the current in the pipes.

The current in the 21 pipes was increased by so doing from 65 to 85 amperes when the negative cables were insulated. When they were uninsulated, the current was increased by tying in the pipes from 273 to 844 amperes. Thus it is seen that with pipe drainage the current is ten times as great in the pipes when the negative feeders are uninsulated as when they are insulated (844 instead of 85). These measurements show what is to be expected, namely, that the current flow in pipes is greatly augmented when the pipes are connected to the tracks or bus bar near the station.

Measurements of differences of potentials between pipes and rails also show the great advantage of the insulated feeder system. The full result obtained on this experimental installation has been published in Technological Paper Number 32.

In Springfield, Ohio, an insulated negative feeder system was installed at the expense of the railway companies in accordance with the recommendations of the bureau, and measurements were then made to determine the degree of improvement effected in overall potential differences in the earth, and in the differences between rails and pipes. A joint report on the system and its results, addressed to the city manager of Springfield, was prepared by representatives of the Bureau of Standards and representatives of the gas, water and telephone companies and the railway companies. The report recommended that the system of insulated negative feeders installed by the railway companies be continued and maintained and that a system of pressure wires be installed to facilitate measuring potential differences between each station and various points at a distance therefrom. That the railway companies agreed to this report voluntarily shows that they were satisfied that the system installed was to their interest as well as to the interest of the pipe owning companies. The American Railways Co., with commendable progressiveness, is adopting this system on other railways under its management.

The Bureau of Standards has been co-operating with the city of Chicago and the Board of Supervising Engineers in charge of the surface railways of Chicago and we hope to see one station fully equipped shortly as a demonstration of what can be done at a moderate expense toward mitigating the damage due to electrolysis in Chicago.

The City of Springfield, Mass., and the various utility companies affected recently asked the bureau to make a complete electrolysis survey of that city and render a report as to the best methods of improving conditions with respect to electrolysis. This work was done in June, but the final report has not yet been completed, owing to other urgent work.

[illegible]

One of the most important reasons why electrolysis conditions have not been remedied oftener by the railway companies is that the engineers of the railways and of the gas and water companies have not got together to study the question. As an illustration of what can be done by co-operation, I may cite a recent experience of our own. A water company in California employed an engineer to [illegible] prepare a case against one of the street railway companies of that city, since the company refused to remedy conditions. The case was to be taken before the California State Railway Commission, and the Bureau of Standards was asked to send one of its engineers to look over the matter and see if the claims and proposals made by the water company were reasonable. The bureau complied with the request with the understanding that our engineer was not serving the water company merely, but would be impartial as between the railway and water companies. When he had examined the case he saw that the system which the state commission was to be asked to order the railway company to install was not the most economical possible, and that since two different railway companies were involved, it would be better to consider the whole problem at once, instead of trying to make a case against one at a time. He then proposed to these two railway companies that instead of fighting the water company before the state commission that they unite with the water company and the representative of the Bureau of Standards in studying the problem, and finding in what manner and at what cost the railways could take care of their return current so as to eliminate further damage to the water pipes. When the railway companies saw that it was a clean proposition for an engineering study, not committing them in advance to the results obtained, they accepted the invitation, and after a most thorough and detailed study including the cost of construction and increased operating costs, as well as economies effected, the four engineers (one representative each of the two railway companies, the water company and the bureau) agreed on the plan to be recommended, with the intention of asking the state commission to order its installation, instead of making a fight before the commission which the latter would have to decide.

In another case, a city asked the bureau to recommend a form of enforcible ordinance which the city could pass that would compel the electric railway company to take care of its current so as to stop the damage to the city's water pipes. The bureau advised the city that before passing an ordinance an engineering investigation should be made to determine exactly what should be done to remedy electrolysis conditions and ascertain what it would cost. Such an investigation was made and a report submitted by the bureau to the city and to the utility companies concerned. The investigation showed that the trouble could be corrected by the installation of a moderate quantity of insulated negative copper, building one new substation and enlarging another. The cost was quite appreciable, but the saving in power thereby effected was large enough to cover the extra operating expense, the interest and depreciation on that part of the new construction not otherwise needed and leave a small profit, the correction of serious electrolysis trouble being accomplished without any real net expense, and the service on the railway as regards car speeds and car lighting being improved. The railway company was entirely satisfied with the report, including the estimates of cost of new construction and operation and savings of power and improvement of service, and indicated their intention to carry out the work promptly. I am sorry to say, however, that to date nothing has been done, although several months have elapsed. The relation between electrolysis mitigation and improved electric car service just alluded to is a matter of much importance.

[illegible] COMMITTEE.

In order to get some reliable engineering results as to current practice in this country and abroad, and as to results achieved by different methods, a joint electrolysis committee was established about six months ago at the instance of members of the American Institute of Electrical Engineers. This committee has three members each of the American Institute of Electrical Engineers, American Gas Institute, Natural Gas Association, American Electrical Railway Association, American Railway Association, the American Telephone and Telegraph Co., and one member from the Bureau of Standards. The chairman is Mr. B. J. Arnold of Chicago, past president of the Institute of Electrical Engineers, and the committee contains many men prominent as railway, gas and electrical engineers. There can be no doubt that the committee will gather valuable engineering data, and do a good service in getting together representatives of the various interests concerned and having them study electrolysis mitigation on an engineering question. The committee agreed at the start that legal considerations and all questions of responsibility for damage should be excluded, and attention given to the scientific and engineering phases, and to determining what had been accomplished in different countries. It is to be regretted that there are no representatives of the water works associations on this committee.

DIFFERENT CONDITIONS REQUIRE DIFFERENT TREATMENT.

Enough has been said to show that conditions with respect to electrolysis are very different in different places. Precautions that are sufficient in one place are insufficient in another; and changes or new construction that can easily be afforded by one company might be a hardship to another. Hence a uniform requirement on all railway companies, passed by a state legislature or by city councils, is not to be encouraged. On the other hand, a general statute or ordinance holding railway companies responsible for the destructive effects of their return electric currents and requiring them to take suitable precautions to minimize such effects would be reasonable. If the managers of utility companies will treat the question in an enlightened and progressive manner as an engineering and business proposition, and keep out of the courts as completely as possible, city councils and state legislatures will no doubt be patient and give them a chance to solve the problem in an economical and engineering manner. But if these managers assume a reactionary attitude, and decline to do anything until forced to do so, they may be compelled to do in many cases far more than is necessary or reasonable.

HOW THE BUREAU'S WORK IS DONE.

In this work the Bureau of Standards is not acting as consulting engineers to serve a single company or interest, as against other companies or interests. On the contrary, all this work is being done as a part of our general electrolysis investigation, and many of the results obtained are published for the benefit of cities, utility companies, consulting engineers and all others interested.

The records and information obtained by the bureau are available to any one who visits the bureau even before publication, and we have frequent calls from consulting and company engineers interested in these questions. If as we expect the practicability and advantage of the insulated negative feeder system is soon completely demonstrated, it will not be long before its use will become more or less general and the Bureau of Standards will be able to turn its attention to other subjects. But so long as the practicability of this system is not generally admitted, and nothing better is devised in is place, and electrolysis conditions continue to get worse every year, as they are now doing in most places where the overhead trolley system is in operation, we expect to continue our work of investigation and education.

At the beginning the bureau bore the greater portion of the expense incurred in its surveys and reports, and utility companies that co-operated with the bureau generally did so as a favor to us, which we fully appreciated. Now that the practical value of the results achieved are better understood, the expense is borne in large proportion by the utility companies, but in every case the bureau works as an impartial agency, just as mindful of the interests of the railways as of the pipe and cable owning companies.

CONCLUSION.

One peculiar feature of the electrolysis problem is that so many different interests are affected. The gas, water, telephone, electric light, telegraph and signalling companies and often many private owners have pipes or cables affected by the electric current from the street railways. Heretofore the railways have generally claimed, and usually I think sincerely, that it was beyond their power to prevent the damage and hence they could not be held responsible for it, at least to the degree that they could if it was practicable to prevent it. Naturally the railways have favored the use of pipe coverings and pipe drainage, as a means of reducing the damage that put much of the responsibility on the pipe owning companies. The cable owning companies have spent large sums of money in protecting their cables by drainage. This is more practicable for cables than for pipes, for the sheaths are usually continuous electrically, but it is far more difficult and more expensive when the potential gradient is large than when it is small. If the use of uninsulated negative feeders was the only method available for taking care of the return current the railway companies could be excused for not maintaining a low potential gradient in the earth. But the use of insulated feeders is so much more economical, that generally speaking there is no longer any excuse for permitting bad conditions. Just as soon as it is recognized to be practicable it will be deemed as necessary to spend money for negative conductors as for the positive feeders.

The method has been much used in Europe and to some extent in this country, but so little in this country that few engineers knew that it was used and until very recently there were relative few who believed it practicable. I know of no published account giving full particulars of the actual results obtained by such a system previous to the recent articles describing the work done by the Bureau of Standards.

I have laid a good deal of emphasis on the duty of the railway companies to take care of their return current, and of the need of applying the same kind of engineering talent and experience to this phase of plant construction and operation as to other phases. I wish also to emphasize the duty of the owners and managers of water works to co-operate with the gas and telephone companies and railways in studying the problem of electrolysis mitigation in an open-minded and progressive manner. Taking the matter into the courts is to be deplored. It is better to employ engineers to work together to solve the problem in a thorough and efficient manner than to employ experts to testify in court as to who is responsible or what are the damages. As the damage is cumulative from year to year and the rate increases as the traffic increases, there will be no better time than the present to take up the matter.

City of Los Angeles Enjoined from Delivering Impure Water.—Suit has been instituted in the Superior Court to enjoin the city of Los Angeles, Cal., from turning into the public mains the water supply of the Owens River aqueduct. The suit was filed by Henry A. Hart, a former member of the Aqueduct Investigating Board. He alleges that the water is unfit for use, and is supported by Dr. Ethel Leonard, bacteriologist. The aqueduct has cost the city of Los Angeles about $30,000,000. City officials claim that the suit is the work of certain interests that have fought the aqueduct project from its inception.

BRIDGES

Design Features of the Substructure and Approaches of the Bloomfield Bridge, Pittsburgh, Pa.

(Staff Article.)

II.

The Bloomfield Bridge, which spans a ravine and connects Grant Boulevard at Ridgeway St. with Liberty Ave. near Main St., Pittsburgh, Pa., is a highway structure having a total length of about 1,740 ft. and an extreme width of 49 ft. 9 ins. It consists of a combination of cantilever, simple-truss and girder spans on concrete pedestals, piers and abutments. An illustrated article describing the design features of the superstructure was published in our Sept. 9, 1914, issue. In this issue we shall describe the principal design features of the substructure and approaches of this bridge.

Figure 1 shows a profile of the site of the bridge, an elevation of the pedestals, piers and approaches, and gives the spacing of the abutments, pedestals and piers. The structure is on a 3.907 per cent grade. Some of the piers and pedestals are on pile foundations, while others have spread footings. Table I gives the type of bridge and the length of span, center to center of bearings, for each span. These data, together with the statement that the spans, in general, are carried on steel bents, will facilitate an understanding of the substructure details.

CONCRETE PROPORTIONS AND CONCRETE PILES.

Concrete Proportions.—The concrete in the abutment footings, below the top of the cantilevered base, and in that part of the piers, pilasters and pedestals having vertical sides is a 1:2½:5 mix. For that part of the abutment walls, including the coping, above the cantilever base, and in the battered portions of piers, pilasters and pedestals and their copings the mix is 1:2:4. A 1:2:4 mix is also used in the concrete piles, the gravel used in this concrete being that retained on a ¼-in. mesh screen and passing a 1-in. mesh screen.

Concrete Piles.—The specifications require that the concrete piles used shall meet the following test: A load of 30 tons is to be placed on the test pile and allowed to remain for 36 hours; then the load is to be increased to 40 tons, 50 tons, and then to 60 tons, the latter load remaining for 36 hours; the test load is then to be reduced to 30 tons, and then removed. The piles to be acceptable must not show a settlement exceeding ¼ in. after the 30-ton load has remained for 36 hours, nor more than ½ in. after the 60-ton load has remained for 36 hours, nor shall there be a variation exceeding ¾ in. between the reading taken

Fig. 1. Profile of Site of Bloomfield Bridge, Pittsburgh, Pa., and Elevation of Piers, Pedestals and Abutments.

TABLE I.—TYPE OF BRIDGE AND LENGTH OF EACH SPAN.

Location of span between—	Type of bridge	[illegible]
South abutment and pedestal No. 1	Deck plate girder	[illegible]
Pedestals Nos. 1 and 2	Deck plate girder	[illegible]
Pedestal No. 2 and anchor pier No. 4	Riveted deck truss	[illegible]
Pier No. 4 and pier No. 5-6	Pin-connected deck anchor arm	[illegible]
Pier No. 5-6 and pier No. 7-8	2 pin-connected deck cantilever arms 1 riveted deck suspended truss	140, each 120
Pier No. 7-8 and pier No. 9	Pin-connected deck anchor arm	120
Pier No. 9 and pedestal No. 1[illegible]	Riveted deck truss	120
Pedestals Nos. 11 and 12	Riveted deck truss	[illegible]
Pedestal No. 13 and north abutment	8 deck plate girder spans	60, each

before the test load is applied and immediately after the 60-ton load has been removed. In case any pile tested shall fail to meet the above requirements the safe capacity of the entire group in which such test pile is located shall be taken at one-half the load giving a ½-in. settlement; and sufficient additional piles shall be driven at the contractor's expense to make up the difference. Three satisfactory tests are required to be made at the contractor's expense. If straight piles are used they are to have a minimum diameter of 16 ins. If octagonal tapered piles are used the minimum diameter allowed is 9 ins. at the point, with a taper of 1 in. in 10 ft.; while if round tapered piles are used the minimum diameter allowed is 10 ins. at the point, with a taper of 1 in. in 10 ft. Premoulded piles are required to be fitted with a cast-iron or steel pile driving point. They must have thoroughly reinforced driving heads, and must be reinforced with longitudinal rods and with satisfactory hooping or spiral winding so as to resist, without damage, the blows from driving.

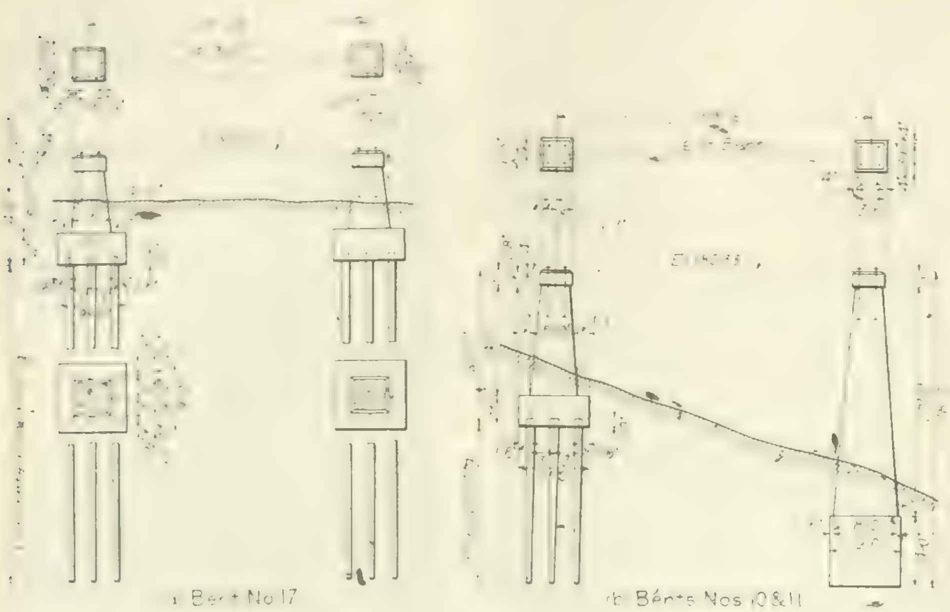


Fig. 2. Detail Drawings of Concrete Pedestals for Bents Nos. 10, 11 and 17 of Bloomfield Bridge.

PEDESTAL DETAILS.

The concrete pedestals for bents Nos. 1, 2 and 3 and for one pedestal each of bents Nos. 10, 11, 12 and 13 (see Fig. 1) have spread footings; the remaining pedestals have concrete pile foundations. The heights of these pedestals and the specified lengths of the concrete piles vary considerably. The distance between the pedestals for the various bents which support the steel spans also varies considerably.

Figure 2 (a) shows details of the pedestals for bents No. 17, these pedestals resting on 50-ft. concrete piles. The anchor bolts for these pedestals (which are typical of those of all pedestals of this structure) are 1½ ins. in diameter and 6 ft. 3½ ins. long. They have square heads and hexagonal nuts, and each bolt has a plate washer 6x¾x6 ins. The tops of the bolts are set 3½ ins. above the tops of the pedestals. Figure 2 (b) shows details of the pedestals for bents Nos. 10 and 11. It will be noted that the east pedestals for these bents have pile foundations, while the west ones rest directly on the subsoil. The sloping character of the ground at these bents is indicated by the drawing. Table II gives essential data on the size, spacing and type of foundation for each pedestal of the bridge.

Main Piers.—Piers 5, 6, 7 and 8 (see Fig. 1), which support the towers of the main cantilever system, have spread foundations and are constructed of plain concrete. The piers supporting bents 5 and 6 are spaced 37 ft. 8⅜ ins., center to center, while those for bents 7 and 8 are spaced 38 ft. 8⅛ ins., center to center.

Piers 5 and 6 have footings 18 ft. 8 ins. x38 ft. 8 ins. x 6 ft. high; pier shafts 15 ft. 8 ins. x 38 ft. 8 ins. at the base, 7 ft. x 27 ft. at the top, and 52 ft. high; and a coping 7 ft. 8 ins. x 27 ft. 8 ins. x 2 ft. thick. Each pier has embedded in its coping an I-beam grillage consisting of two 20-in. 65-lb. I-beams, 26 ft. long, and four 20-in. 65-lb. I-beams, 6 ft. long (two at each end of each pier). The four lines of I-beams in each pier are spaced 10 ins. on centers, the 26-ft. beams being placed on the outside. These grillage beams are connected by diaphragms. There is embedded in each pier two sets of four anchor bolts each for anchoring the tower legs. These bolts are 2 ins. in diameter, 13 ft. 7½ ins. long, and they project 1 ft. 7½ ins. above the top of the piers.

Piers 7 and 8 have footings 17 ft. x 37 ft. x 6 ft. high; and pier shafts 14 ft. x 34 ft. at the bottom, 7 ft. x 27 ft. at the top, and 42 ft. high. These piers are spaced 38 ft. 8⅛ ins. on centers. The dimensions of the copings and all other dimensions and details not given are the same as for piers 5 and 6.

Anchorage Piers.—To resist the upward reactions at the ends of the anchor arms of the cantilever system (at bents Nos. 4 and 9, see Fig. 1) heavy anchor piers are required. These piers support heavy bents (see Part I, in our July, 1914, issue), each of which in turn connect to an anchor arm and an adjacent 120-ft. truss span. These piers are practically buried in the ground. Figure 3 shows side and end elevations of one of these concrete piers and of the embedded anchorage. The four eye-bars, which project above the pier, shown in the drawing, are connected to an adjustable shoe of the steel bents by means of a 6½-in. pin.

ABUTMENTS AND RETAINING WALL APPROACHES.

At each end of the bridge there is a reinforced concrete abutment and an earth approach between reinforced concrete retaining walls. Each abutment supports one end of a 60-ft. girder span and acts as an end retaining wall.

Figure 4 (a), p. 297, is a half plan of the south abutment and retaining wall approach; Fig. 4 (b) is an elevation of the west wing wall and a section through the center line of the abutment; and Fig. 4 (c) is a cross-section of the retaining walls and roadway. These drawings show the type of construction and details of the abutment and retaining walls. It will be noted that the inside of the walls is lined with a 12-in. layer of stone and that drains are provided at the top of the footings to facilitate drainage. The abutment is separated from the retaining walls by expansion joints, as shown in Fig. 4 (b). An expansion joint is also provided at each

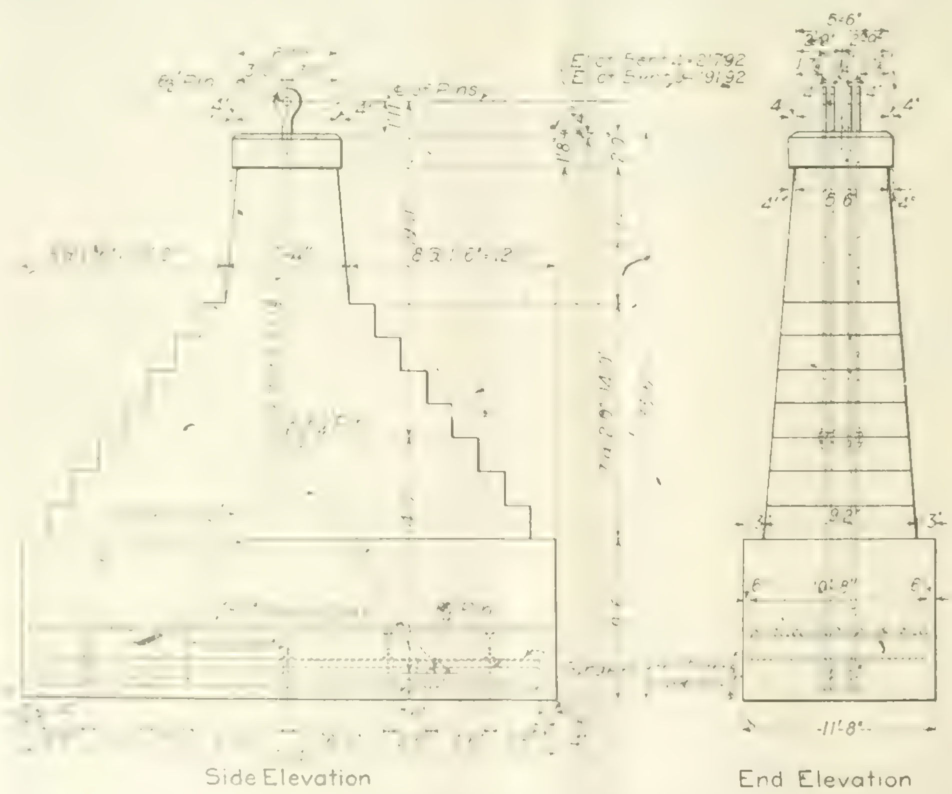


Fig. 3. Side and End Elevations of Anchor Piers of Bents Nos. 4 and 9 of Bloomfield Bridge. Note Embedded I-Beams and Grillage.

step in the retaining walls. It will be noted that the east and west retaining walls differ somewhat as to their dimensions, as the ground surface is lower on the east side.

TABLE II.—DATA ON SIZE, SPACING AND FOUNDATIONS OF PEDESTALS.

Bent No.	Type of foundation. East.	West.	Height of pedestal, ft. East.	West.	Dimensions of footing, ft. East.	West.	Dimensions of coping, ft.	Distance between pedestals, ft., ins.
1	Spread	Spread	8	16.5	7×7×2	8×8×2	4-2×4-2×2	35- [illegible]
2	Spread	Spread	10	10	7×7×2	7×7×2	4-2×4-2×2	34- 1¼
3	Spread	Spread	10	10	7×7×2	7×7×2	4-2×4-2×2	36- 1¼
10 and 11	9, 20-ft. piles	Spread	20	18	9×9×4	9×9×9	4-2×4-2×2	39- 1¼
12 and 13	9, 30-ft. piles	Spread	16	41	9×9×4	9×9×6	4-2×4-2×2	35- 5½
14	9, 40-ft. piles	9,40-ft. piles	14	14	9×9×4	9×9×4	4-2×4-2×2	37- 0¾
15	6, 50-ft. piles	6, 50-ft. piles	14	14	6×9×4	6×9×4	3-5×3-5×2	36- 1½
16	6, 50-ft. piles	6, 50-ft. piles	14	14	6×9×4	6×9×4	3-5×3-5×2	35-11¼
17	9, 50-ft. piles	9, 50-ft. piles	14	14	9×9×4	9×9×4	4-2×4-2×2	35- 0⅞
18	6, 50-ft. piles	6, 50-ft. piles	14	14	6×9×4	6×9×4	3-5×3-5×2	34- 3⅝
19	6, 50-ft. piles	6, 50-ft. piles	14	14	6×9×4	6×9×4	3-5×3-5×2	34- 1¼
20	9, 50-ft. piles	9, 50-ft. piles	14	14	9×9×4	9×9×4	4-2×4-2×2	33- 5¾
21	9, 50-ft. piles	9, 50-ft. piles	18⅓	18⅓	9×9×4	9×9×4	4-2×4-2×2	32- 0
22	9, 50-ft. piles	9. 50-ft. piles	16	16	9×9×4	9×9×4	4-2×4-2×2	32- 0

The north abutment is somewhat similar to that shown in Fig. 4, but is supported on a concrete pile foundation. This abutment has a width of 50 ft. and a maximum height of 24 ft. The combined length of the abutment and retaining walls is 100 ft.

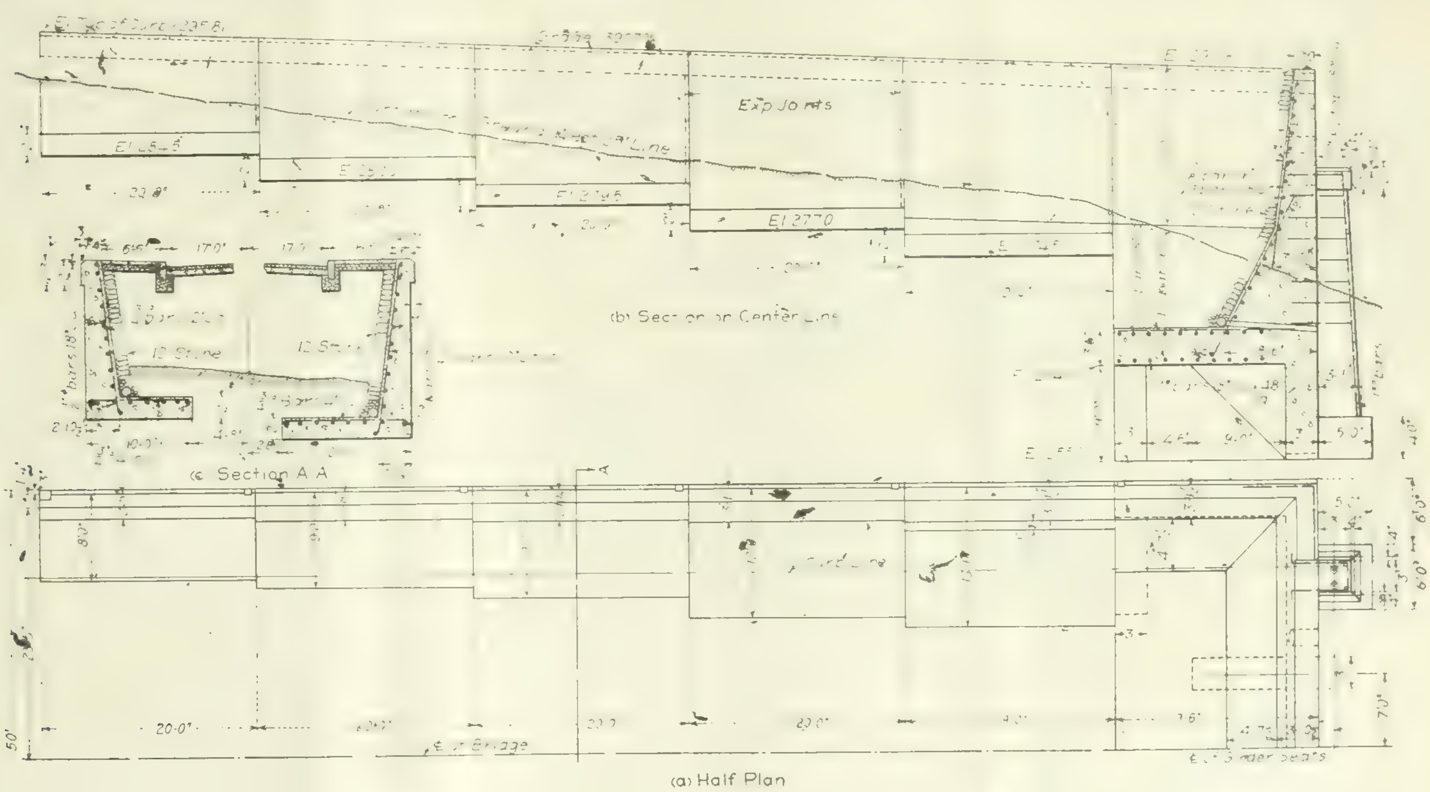


Fig. 4 (a). Plan of South Abutment and Retaining Wall Approach of Bloomfield Bridge; (b) Cross Section of Abutment and Elevation of West Retaining Wall; (c) Cross-Section of Retaining Wall and Roadway.

0 in., not including the projecting supports for the girder span. All except the end 40 ft. of the retaining walls are also supported on pile foundations. Expansion joints, similar to those shown in Fig. 4, are provided.

QUANTITIES OF MATERIALS AND COST.

The estimated quantities of materials in the substructure are as follows:

Excavation, cu. yds	8,280
Fill, cu. yds	3,430
Concrete, cu. yds.	6,100
Reinforcing steel, tons	13.[illegible]
Anchorage and grillage steel, tons	74.6
Concrete piles, lin. ft	8,442

The contract price for the substructure was $82,615.58.

The bidding price for the concrete piles was $1.50 per linear foot; for additions or deductions in quantities of concrete, $7.25 per cubic yard; and for extra steel reinforcement 4 cts. per pound.

PERSONNEL.

The Bloomfield Bridge was designed by the Department of Public Works of Pittsburgh, Mr. Charles A. Finley, acting director, Mr. N. S. Sprague, superintendent, Mr. T. J. Wilkerson, division engineer, and Mr. Emil Swensson, consulting engineer. The substructure was built by the Dravo Contracting Co., of Pittsburgh.

Activities at Dayton, O., Under Commission-Manager Government. — According to a quotation from the city manager of Dayton, made by Mr. F. F. Collins, Superintendent of Water Works at Manistee, Mich., in his paper before the Central States Water Works Association, the following are the principal municipal activities at Dayton since the first of 1914:

Since Jan. 1, 1914, there has been installed an accounting system which provides exact control over public funds and gives a continuous audit, so that the city knows exactly where it stands financially every day. There has also been organized a purchasing department where all city supplies are bought in the open market in competition at reduced prices; established a school for policemen, which gives instruction in ordinance and character of duties and target practice; begun regular fire prevention inspection of every residence and store building, which has greatly reduced the number of fire department runs. The city has also clothed the prisoners in the workhouse in an adequate manner; used the prisoners from the work-house to clean up vacant lots for public gardens and to improve the city parks; and secured a large number of vacant lots for those desiring gardens, and plowed the land free of charge. There are nearly 10,000 vacant lot gardeners within the city limits now because of this move. The city has also appointed five district city physicians for gratuitous service to those unable to pay; has a commission drafting a building code; has added seven patrolmen to police force for use in residence districts; has unified the visiting nurses of the city under the direction of the welfare department; has amalgamated the work of the playgrounds and garden associations with the park and playgrounds activities of the welfare department; has established a free clinic at the office of the division of health, and free baby clinic at two hospitals; and has carried through a school of instruction for playground leaders.

The city administration has taken up important work which will require years to finish. Surveys of the water and garbage situations are being made, a vice investigation is in progress, an examination of the police and fire systems has just been finished, and all this work has been done by experts. The people of Dayton as a whole are behind the city manager, and are just as enthusiastic as ever for the Dayton plan of municipal government.

Manufacturers of Hollow Steel Rods.— The firm of Dunford & Elliot, Sheffield, England, has recently introduced a patented process for manufacturing hollow metal rods. By this process hollow steel bars from ¾ in. to 3 ins. in diameter and 20 ft. long are produced by rolling from a drilled ingot. After the piece has been pierced it is packed with a kind of sand which is capable of resisting high temperatures. The hole is plugged at each end with metal plugs and the billet is then heated and submitted to repeated rollings until a bar of the required dimensions is produced. It appears that the packing material acts as a fluid or as an elastic mandrel, thus maintaining a hole through the bar. After elongation the core is removed by a special process.

Bridge Foundations and the Use of Pneumatic Caissons for Constructing Bridge Foundations in Canada.

The building of large bridges requires the most detailed investigations and careful study to insure the selection of the most efficient type of substructure and the best procedure to follow in constructing the foundations. In difficult foundation work the pneumatic process is often used, and the purpose of this article is to classify bridge foundations and to give some examples of the use of the pneumatic process in Canada during the last few years, together with the reasons which led to its adoption in each case. The article is based on a paper by John W. Doty, in the Proceedings of The Canadian Society of Civil Engineers.

Types of Foundations.

The pneumatic process has been developed to meet treacherous water and soil conditions and to insure the founding of the structure on a suitable and properly prepared bottom in the shortest possible time, and for the minimum cost considering the conditions encountered. It is naturally the most expensive method of construction, but it is possible by means of it to build substructures where all other methods would fail.

To emphasize the importance of this method it will be of interest first to classify foundations in general in the order of their relative cost, assuming that the conditions are favorable to the use of the type selected. The following classification may be made:

(1) Spread footings.
(2) Piles.
(3) Piers constructed by the open method.
(4) Piers constructed by the pneumatic method.
(5) [illegible]

Foundations under Class 1, spread footings, are usually placed directly on the soil a few feet below the ground level. This class of foundation may be subdivided into the following types:

Timber grillages.
Concrete footings.
Reinforced concrete footings.
Steel grillages imbedded in concrete.

Foundations under Class 2, pile structures, are generally used where the material at the bottom of the footing is not capable of supporting the load and it becomes necessary to drive piles to some better stratum of material in order to obtain a safe support. This class of foundation may be subdivided into the following types:

Sand piles.
Wooden piles.
Concrete piles moulded in the ground.
Concrete piles built up, seasoned and then placed.
Steel tubular piles driven and filled with concrete.

The foundations under Class 3 are usually carried a considerable depth below the general ground level to hardpan or rock, and may be subdivided into the following types:

Timber sheet pile constructed cofferdam.
Steel sheet pile constructed cofferdam.
Timber or steel boxes built up and sunk as excavation is made.
Timber lined construction, where the sides of the excavation are lined with timber as the excavation is made.
Caissons built and filled with concrete before sinking.
Monolithic concrete caissons which are molded before sinking.

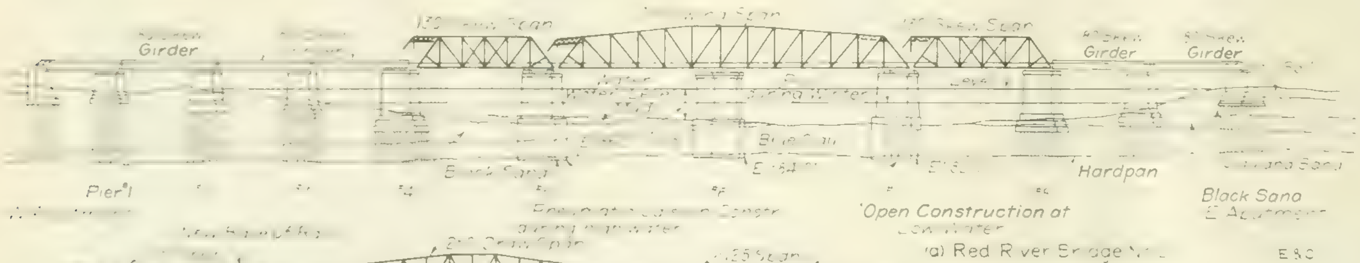
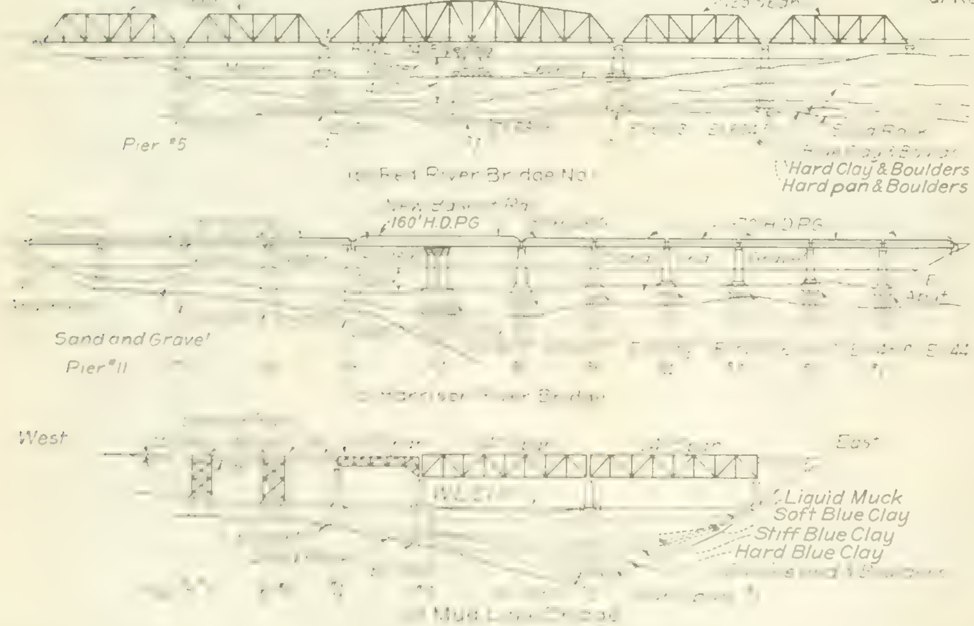



Fig. 1. General Elevations of Four Important Bridges in Canada in Which Pneumatic Caissons Have Been Used in the Construction of Their Foundations.

The foundations under Class 4, piers constructed by pneumatic process, are usually carried a considerable depth below the ground level to hardpan or rock, but the limit of depth to which this method is practical is 100 ft. below water level. This class may be divided into the following types:

Caissons built of timber or steel and filled with concrete before sinking.
Monolithic concrete caissons which are molded before sinking.

As regards Class 5, it is evident that the two last types of Class 3 can be readily commenced by the open method and completed by the pneumatic method.

The principal conditions governing the selection of the type of foundation for any

(1) Magnitude and distribution of the loads to be supported.
(2) Character of the soil at the proposed site.
(3) Depth to rock or to a suitable bearing material below the water level.
(4) Elevation of water level.
(5) Value of the structure to be supported.
(6) Surrounding conditions, such as adjoining structures, etc.
(7) Speed of completion.
(8) Possibility of the disturbance of the material in which the foundation is resting.
(9) Economy of construction.

The type of foundation selected for any particular bridge should be the one that safely fulfills all the above conditions and is the cheapest to construct.

The importance of the foundations for structures makes it imperative that the engineer should take a conservative attitude in selecting the type to be used. The following are some of the causes of failure of the various classes of foundations:

Class 1.—Spread Footings.

Overloading the soil.
Placing footings on a soil overlying one of lesser bearing value.
Designing eccentric footings and not taking into consideration the maximum pressure due to the load not being evenly distributed.
Designing reinforced concrete footings on the assumption that the pressure will act uniformly on the soil, no allowance being made for the lack of uniformity of the soil.
Placing heavy loads on reinforced concrete footings where they are extended below the water level.
Not carrying footings deep enough to prevent frost action.
Not carrying footings deep enough to prevent undermining by scour.
Not carrying the footings deep enough to provide protection against the material under the footing being disturbed by adjoining construction operations.

Class 2.—Piles.

Wooden piles which have not been driven to a firm underlying stratum or to a sufficient penetration.
Wooden piles which are overdriven and broken.
Where a great many piles are driven at close centers it is, in many cases, impossible to drive all the piles to a firm stratum on account of compressing the upper strata. A readjustment of the soil might take place and permit the structure to settle, as all the piles were not driven to a firm underlying stratum.
A steel tubular pile is as a rule overloaded, and no allowances are made for the reduced bearing value when the exposed steel has deteriorated. Moreover, failure may be due to the impracticability of founding the bottom of the pile on a material capable of supporting the usual overload.

Class 3.—Piers Constructed by Open Method.

Failure of this class usually occurs when the foundations have not been founded on a suitable bottom, due to the impossibility of cleaning it off properly on account of the large quantities of water which are encountered.
Failures due, not to the completed foundations themselves, but to the fact that the method of construction would not prevent the flow of material which will probably cause damage to the adjoining property, especially if the water or soil conditions are treacherous.

Class 4.—Piers Constructed by Pneumatic Process.

The impracticability of constructing this type of foundation where large quantities of water are encountered or where the nature of the soil is such that the action of the water on the soil will cause it to flow, making the conditions impossible to reach the assumed bearing elevation.

Class 5.—Combination of Classes 3 and 4.

Failures in this class are due only to poor workmanship or to unavoidable accidents, but these, with the present-day methods, are very improbable.

From the above data it will be noted that the pneumatic process is classified as the most expensive, but it is the one that overcomes the difficulties met in reaching a suitable footing and permits the proper preparation of the bottom on which the structure is founded.

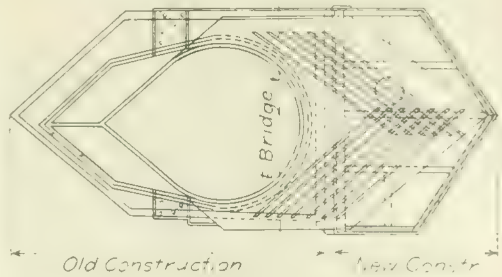
Moreover, this process permits construction below adjoining structures where water and soil conditions are serious, without damage to the adjoining structures.

The pneumatic process has developed during the past fifty or sixty years with the advent of heavy and expensive structures. It might be of interest to note that this method of construction is derived from the principles used in the diving bell. This appliance was conceived by the ancient Greeks, and was developed and used in America for the first time on bridge substructure work about the year 1860, the present method having been worked out during the last fifty years. A pneumatic pier usually consists of three parts, the working chamber, the shaft and the cofferdam. To this structure should be added the equipment necessary to control the air pressure, consisting of the air shaft, the air lock and the air supply pipe line. The working chamber is merely an inverted box with an open bottom, on which is placed a portion of the concrete shaft, while extending above and around the perimeter of this concrete shaft is the cofferdam. The steel air shafting is built into the concrete shaft and connects the working chamber with the air lock. The working chamber, air shaft and air lock are designed to withstand safely pressures of at least 50 lbs. per square inch, which is the practical limit of the pneumatic method. The air lock consists of a shell with two diaphram plates to which are suspended two doors. These doors permit the passing of men and materials from the atmospheric pressure to the increased pressure in the working chamber without allowing any appreciable decrease of the higher air pressure in the working chamber. In addition to equipping each caisson it is necessary to operate an air compressor plant. Although it is not the intention to discuss, in detail, the construction or equipment of pneumatic caissons, the above brief outline gives the main features of the pneumatic caisson and shows the reason for the expense in constructing work by this process.

EXAMPLES OF USE OF PNEUMATIC CAISSONS IN CANADA.

The pneumatic method of constructing bridge substructures has been used during the past few years in Canada for the following bridges:

Two bridges over Red River, at Winnipeg.
Bridge over St. Lawrence River, near Lachine.



Bridge over Harrison River, at Harrison Mills, B. C.
Quebec Bridge.
Bridge over Mud Lake, near Perth, Ontario.

A brief description of the foundation work on each of the above bridges will now be given, together with the conditions which led to the use of the pneumatic process.

Red River Bridge No. 2.—This bridge is located at Winnipeg and consists of six deck plate girder spans, two through truss spans and a swing span. Figure 1 (a) shows a general elevation of the structure, giving the depth of water and the depth to rock, the span lengths, and the character of the material overlying the rock. In general, the water has a depth of approximately 28 ft. while the rock lies about 62 ft. below water level. The soil overlying the rock varies from a stiff clay at the east bank to a fine flowing sand at the west bank, while in some places both of these materials were found in different strata. In general, the soil in and around Winnipeg flows or is displaced by a surcharged load or by vibrations; and even though some clays may be loaded safely with two tons per square, the use of spread footings would have been impracticable for this heavy and expensive structure.

Pile foundations were adopted for the two abutments and for piers Nos. 1, 2, 3, 4, 8 and 10, which support the fixed spans; but on account of the character of the material and the general movement which has been referred to it was deemed inadvisable to use this type of foundation for piers Nos. 5, 6 and 7, which support the 310-ft. draw span. It was therefore decided to carry piers 5, 6 and 7 to hardpan or rock, which borings indicated would be encountered at a depth of 65 ft. below the water level.

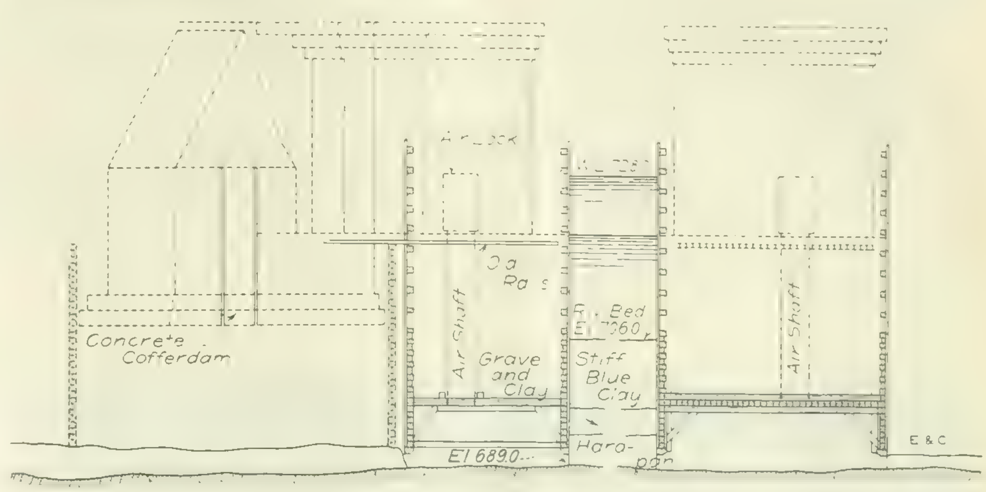


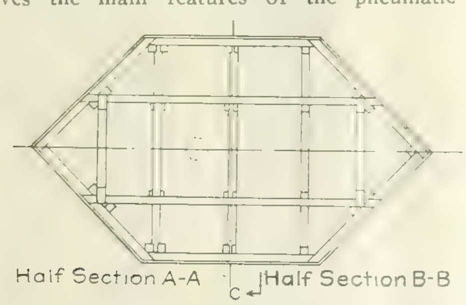
Fig. 3. Type of Caisson Used to Enlarge Pier No. 3 of Red River Bridge No. 1. Note Bonding of New and Old Construction.

Dredging types (Class 3) were considered inadvisable on account of the stiff clay and sand strata which were found at the site of these piers. (Stiff clay is a material which cannot be excavated economically from the inside of a box through any great depth of water.) It was also considered inadvisable to use the open cofferdam type and to excavate in the open, as numerous water-bearing sand strata were encountered at depths which were not economical for unwatering. Moreover, on account of the necessity of founding the piers on a uniformly hard bottom in the hardpan, at such depths below the water level, it was thought impracticable to pump out the cofferdam in order to make the excavation in the open.

Another item which had to be taken into consideration was the time allowed to complete the bridge. The plans were approved by the government Dec. 28, 1912, and it was required that this structure be completed not later than the first of September. The open method, if possible, with its numerous contingencies in this depth of water would have made the date of completion uncertain.

Pier No. 6 was the largest and last pier to be constructed. The working chamber of the caisson for this pier (see Fig. 2) was built on the shore, and was towed to the site June 25, 1913. It was bottomed by Aug. 19, 1913. The caisson had two air shafts, fitted with air locks, and was constructed and braced as shown in the drawing. This pier, which contained 3,192 cu. yds. of concrete, was completed in 55 days from the date the caisson was towed to the site.

Pier No. 5 was constructed by the pneumatic method at the high water period, while pier No. 7 was built by the open method at the low water period. The latter pier required about twice the length of time to construct as the former and was considerably



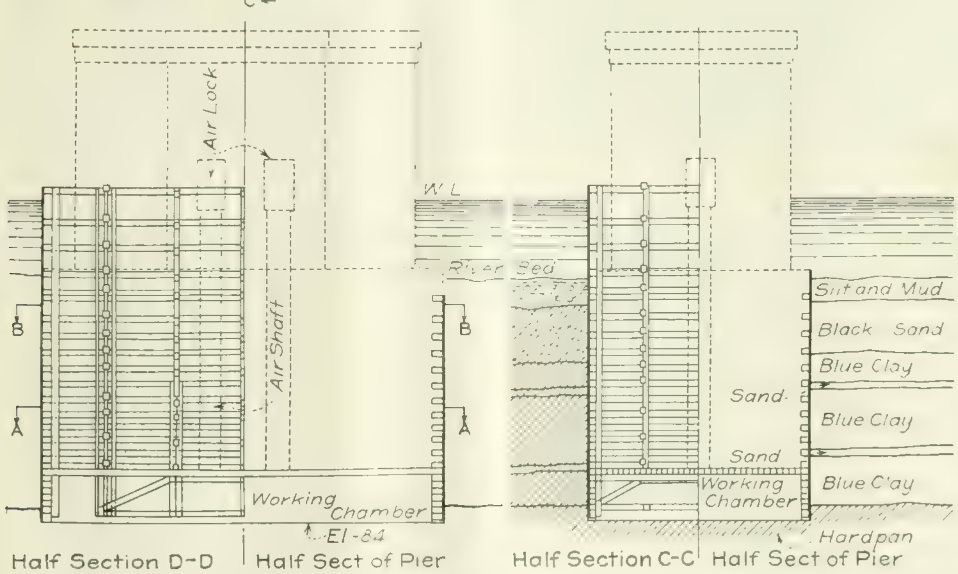


Fig. 2. Details of Caisson Used in Constructing Pier No. 6 of Red River Bridge No. 2. Note Character of Subsoil at Site of Bridge.

more expensive. The entire substructure of this bridge was completed by Aug. 28, 1913.

Red River Bridge No. 1.—This bridge is within three miles of the structure known as Red River Bridge No. 2, and the general conditions are similar, except that the clay does not contain layers of sand as at the site of bridge No. 2. Figure 1 (b) shows a general elevation of the bridge, and gives the span lengths, the depth of water, and the character of the material overlying the rock.

The work consisted of extending the piers of an old bridge for the purpose of double-tracking the structure. The old piers were all carried to hardpan by the open dredging method, except pier No. 5 (see Fig. 1, b) which was founded on piles. The reason for founding this pier on piles was that the material was such that it would flow into a caisson as the excavation was being made in sufficient quantity as to make progress exceedingly difficult.

The extensions to piers Nos. 1 and 2 were built by the wooden sheet pile method (Class 3), which was practicable on account of the stiffness of the clay.

The extension to pier No. 5, which was [illegible] steel sheet pile cofferdam.

The old piers Nos. 3 and 4 were founded on hardpan by the open dredging method, and the concrete was deposited under water. It was probable that some settlement had taken place but that the old piers had come to rest, and the only safe manner of insuring against even a slight settlement of the new portions of these piers was to carry the new portions to rock. This necessitated excavating several feet below the footings of the old piers; and to make this excavation economical, considering both the character of the material and the absolute necessity of leaving undisturbed the hardpan under the old piers, the pneumatic method was used. Figure 3 indicates the method used to extend the center pier of the swing span, No. 3. This drawing shows the type of caisson used and the manner of bonding the new and old construction.

Although the substructure for the old bridge required about two years to complete, that of the new portion was completed in less than seven months.

Harrison Mills Bridge.—This bridge, which is under construction across the Harrison River, at Harrison Mills, B. C., consists of a series of deck plate girder spans and a swing span, which is also of the girder type. Figure 1 (c) shows a general elevation of the bridge, and gives the span lengths, the depth to rock, and the character of the soil overlying the rock.

As the subsoil was principally sand and gravel, with some clay at the east part of the site, it was deemed practicable to found the east half of the bridge on wooden piles, while the west portion was founded on rock, the open caisson method being used, as the distance to rock was not great.

Additional borings which were made at the site after the work had begun showed that the rock near piers Nos. 7 and 8 (see Fig. 1, c) sloped at an angle of from 30° to 40°. At pier No. 8 the depth of water was about 25 ft., the depth to rock at the downstream side of the pier was 30 ft., and the depth to rock at the upstream side, 55 ft. The soil overlying the rock was principally sand and gravel with traces of clay. As it was necessary to carry the footing of the pier into the river bottom at least 8 ft. to protect it against scour, this depth would bring the downstream end to rock, while the upstream end of the footing would be approximately 25 ft. above rock.

It was impracticable to use a spread footing, as one end of the pier would be resting on rock and the other end on sand, gravel and clay, which no doubt would cause it to settle and slip out of line. It also was not practicable to use piles, due to their short length at the downstream end of the pier. Moreover, it was impossible to construct an open dredging caisson as a unit for this pier, as the steep slope of the rock and the expense, which would be incurred by excavating it sufficiently to make the bottom of the caisson level, was prohibitive. For these reasons it was finally decided to found the entire pier on rock, using either an open caisson or a pneumatic caisson for the downstream, or shallow, end and a pneumatic caisson for the upstream end, the rock being stepped in the working chamber to prevent sliding. It was also decided to build the concrete shaft of the pier on two independent caissons, the shaft to extend to a depth approximately 6 ft. below low water.

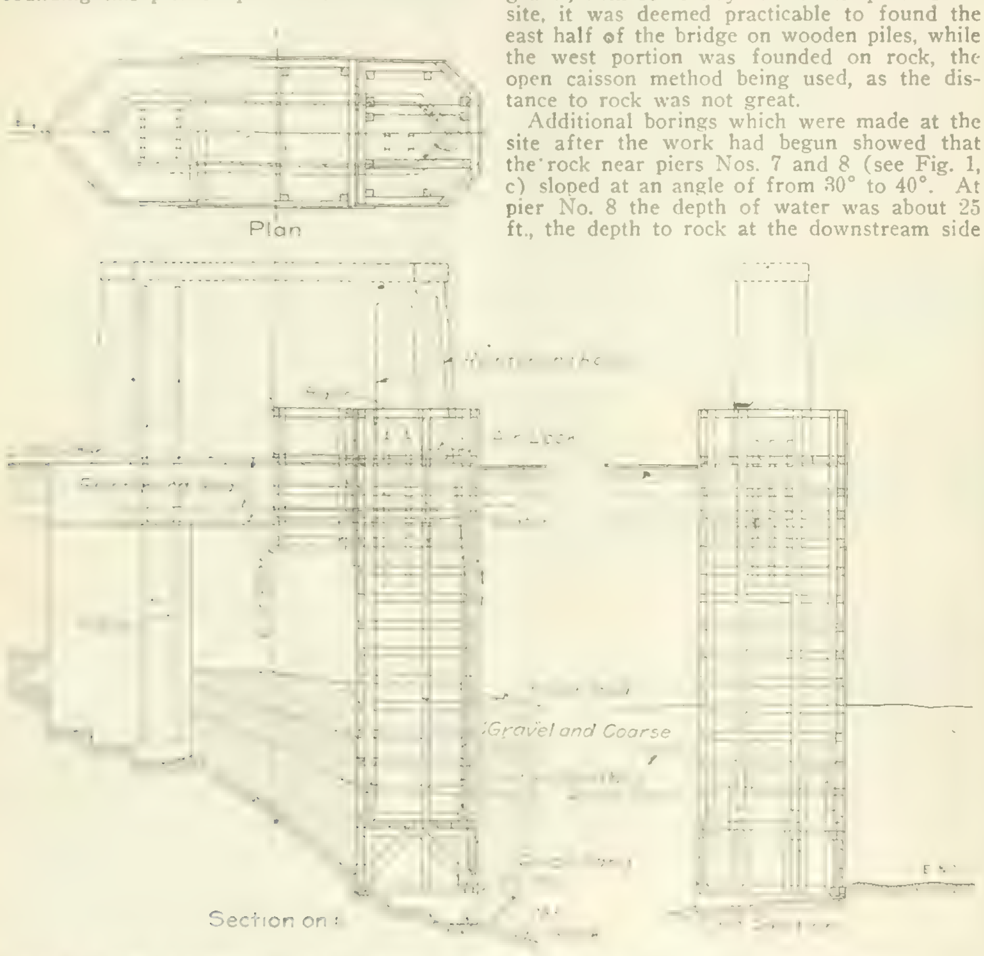


Fig. 4. Type of Caisson Used to Construct Pier No. 8 of Harrison River Bridge. Note Character of Subsoil and Slope of Rock Surface.

Figure 4 shows a plan and elevations of the caissons used for pier No. 8. The two independent caissons were rigidly connected and reinforced, and the pier shaft was reinforced, as indicated in the drawing, to resist the stresses to which this type of pier would naturally be subjected. The same type of construction and methods are also being used for piers Nos. 9 and 10.

Quebec Bridge.—On account of the magnitude of this structure it was necessary to found the piers on a stratum which would absolutely preclude the possibility of settlement, and to carry the footings to such depths as to eliminate any possibility of scour.

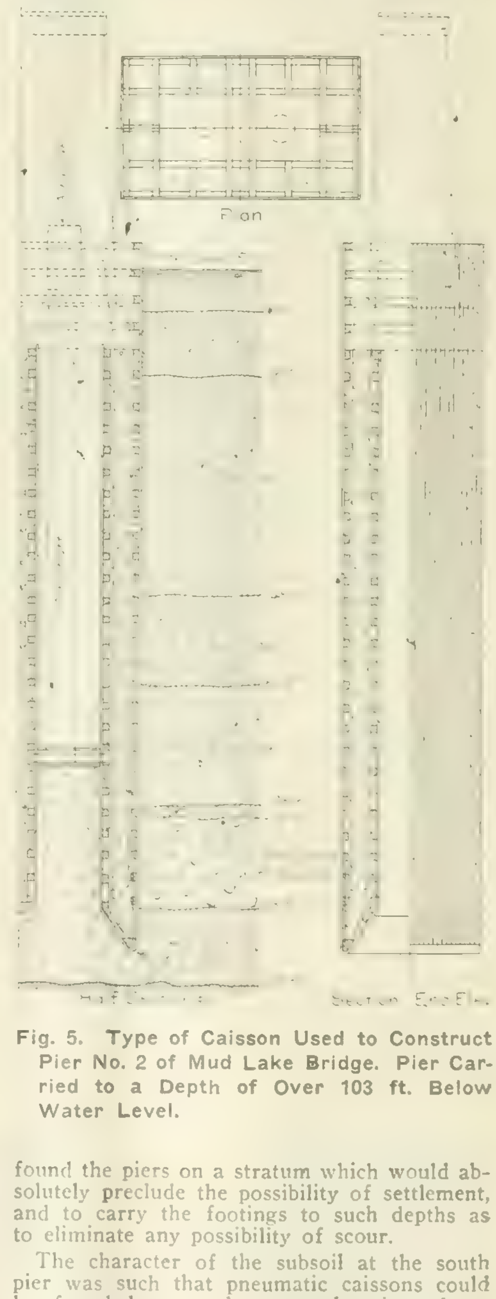


Fig. 5. Type of Caisson Used to Construct Pier No. 2 of Mud Lake Bridge. Pier Carried to a Depth of Over 103 ft. Below Water Level.

The character of the subsoil at the south pier was such that pneumatic caissons could be founded on rock, at an elevation of approximately 85 ft. below the water level. The north pier was founded on hardpan, at a depth of about 60 ft. below water level. For each pier it was necessary to use pneumatic caissons, as the presence of large boulders and the necessity of preparing the rock or hardpan so as to preclude any possibility of settlement, made the use of any other method impracticable.

(A full description of the construction of the Quebec Bridge substructure is given in Engineering and Contracting of July 29, 1914.)

Mud Lake Bridge.—This bridge crosses Mud Lake, near Perth, Ontario, and consists of four deck plate girder spans and three deck truss spans. Figure 1 (d) shows a general elevation of this bridge, and indicates the character of the subsoil.

The conditions at the site of the Mud Lake Bridge are unique in many ways. At pier No. 2 (see Fig. 1, d) the water is only about 8 ft. deep, but for the next 50 ft. the material is a liquid clay. Below this soft material there lies a 12-ft. stratum of stiff clay, under which there is a stratum of boulders, sand and clay, which is very firm and compact. Under the latter structure and overlying the rock to a depth of about 10 ft., there is a layer of cemented sand, the depth to rock at this pier varying from 108 to 115 ft. below the water level.

It is obvious that spread footings would not support the load. A pile foundation would be inadvisable, as the thick layer of liquid mud and the small penetration which the piles would have in the stiffer underlying stratum of clay would not give the piles sufficient lateral stability. Moreover, it is probable that the piles could not be driven beyond the stratum containing boulders. In a liquid mass, such as is here encountered, there is no assurance that the whole mass is not following some progressive movement, which in time might move any structure not fixed to the firm underlying material. In such materials the construction should be such as would permit the materials to flow around the piers without any possibility of displacing them. For this reason it was deemed inadvisable to found the piers on any material not equivalent to hardpan or possibly rock. An open caisson could be carried to the stiff clay, but this type was impracticable, due to the difficulties which would be encountered in dredging and to the fact that the boulder stratum was insufficient to support the piers and was at too great a depth to be dredged.

It was finally decided first to use an open dredging caisson and to carry the excavation to the stiff clay stratum, at which elevation the open caisson was to be converted into a pneumatic caisson and carried down to hardpan or rock. To give greater stability to the pier the bottom was spread a distance of 18 ins. beyond the perimeter of the caisson.

Figure 5 shows details of the caisson used to construct pier No. 2. The excavation for this pier was carried to a depth of 103 ft. 5 ins.—the greatest depth to which any pier has been carried in Canada.

The contract for the construction of the substructure of this bridge was awarded Dec. 28, 1912, with the proviso that the work be completed by July 15, 1913. On account of the delay in obtaining materials, active construction was not started until April 1, yet the substructure was completed Aug. 10, 1913. (The design and construction features of the Mud Lake Bridge were described in detail in our Dec. 24, 1913, issue.)

ROADS AND STREETS

Methods and Cost of Gravel Road Construction in a Road Improvement District in Lowndes County, Mississippi.

(Staff Article.)

Lowndes County is on the eastern border of Mississippi and is included within the belt of fertile farming land which exists throughout the entire length of the eastern portion of that state. The population of the county is about 35,000, Columbus, the county seat, having about 12,000 inhabitants. The county is well watered and contains within its borders the sources of the Tombigbee River.

In September, 1913, contracts were let for the improvement of about 20 miles of road included within Supervisor's District No. 2 of that county; a bond issue of $50,000 having previously been voted for road improvement purposes. The work was to be accomplished under the supervision of a board of highway commissioner consisting of three members, the board employing a road engineer as their agent who was to assume active charge of the work.

DESIGN FEATURES.

Preliminary plans contemplated the use of both gravel and sand-clay surfacing; gravel to be either shipped in by rail or taken from pits located within the county. It was finally decided, however, to use gravel surfacing, and contracts were let for gravel construction, the gravel to be obtained from local pits. A small amount of Tishimingo gravel was used, the quantity being sufficient to surface 1,200 ft. of road.

A 20-ft. graded width was adopted for construction on all roads but one, where a 22-ft. width was used for some distance. Referring to the cross section, Fig. 1, it will be seen that the roadway is widened 2 ft. on each side in cuts to provide for drainage. This method of providing a constant width of traveled way both in cut and on embankment is in accordance with the most modern ideas concerning roadway widths. The slope of 1:1 used for earth on the sides of cuts is sufficient in this latitude; no trouble being experienced from frost action.

Vitrified clay, cast iron and corrugated iron pipe were used for small drains, and provided with plain concrete headwalls where necessary. Reinforced concrete was used for culverts, varying from 2 to 6 ft. in height and 4 to 16 ft. in span. The general type of these culverts is illustrated in Fig. 3. The bottoms of all the culverts were paved with concrete. No culverts were installed having a length between the insides of end walls of less than the graded width of 20 ft. The observance of this rule has an important bearing on making the roads safe to the traveler.

Fig. 2. View Showing Finished Surface. Note Culvert Headwalls the Full Width of the Road Apart.

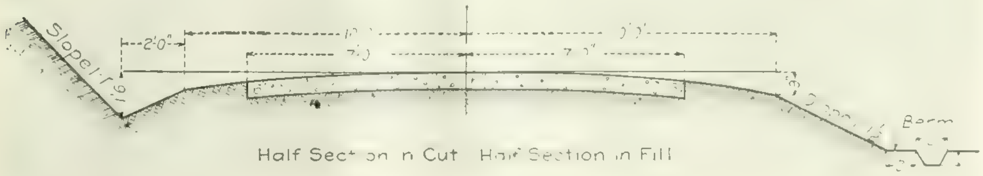


Fig. 1. Typical Cross Sections Used for Gravel Roads in Lowndes County, Mississippi.

CONSTRUCTION FEATURES.

Clearing and Fence Moving.—The moving of telephone poles and clearing of small brush was included in the price paid for grading. However, where occasional fence line trees were to be removed and hedges grubbed such work was paid for at a unit price per station of 100 ft. Where heavy clearing and grubbing through timber occurred a price per square yard was provided. Where fences were to be moved and rebuilt the work was to be paid for by the linear foot as shown in Table I.

Grading.—Earthwork was estimated and paid for in embankment only. Top soil and vegetable matter was stripped where necessary for fills less than 1 ft. in depth. An average free haul of 500 ft. was provided, allowing 1 ct. per cubic yard for each 100 ft. of over-

TABLE I. QUANTITIES AND BID PRICES FOR 20½ MILES OF ROAD CONSTRUCTION IN MISSISSIPPI.

Item.	Quantity.	Bid price.
Moving fences, lin. ft.	1,000	$ 0.04
Clearing and grubbing, per sta.	100	5.00
Clearing and grubbing on new location through timber, per sq. yd.	6,000	0.05
Earth grading, cu. yds.	70,400	0.195
Culvert pipe (laying only) lin. ft., vitrified—		
18-in.	220	0.40
24-in.	21	0.50
Cast iron, 18-in.		0.40
24-in.		0.50
Galvanized iron, 12-in.		0.12
15-in.		0.15
18-in.		0.20
24-in.		0.25
Plain concrete, cu. yd.	225	10.00
Reinforced concrete, cu. yd.	150	12.51
Sub-grade, sq yds.	120,700	0.02
Loading gravel, cu. yds.	25,100	0.125
Hauling, cu. yd—mile	25,100	0.35
Spreading and rolling, cu. yd. compacted	20,100	0.06

haul. The haul was computed from each cut separately and the balancing of a short haul in one cut against a long haul in another was not permitted.

Sub-Grade.—Shoulders were formed of selected material and overlapped at least 1 ft. onto the sub-grade while rolling. After rolling the overlapping material was thrown out with shovels leaving a trench having vertical sides.

The final rolling of the sub-grade was with a 10-ton roller on a portion of the work. Later a heavy horse roller was used which was found to accomplish the same results and to be much more convenient than the heavier power roller on this work.

Gravel Surface.—Gravel for surfacing was obtained from pits opened at points as convenient as possible to the road. The material contained a sufficient amount of clay to bind well. The surfacing was spread in one course 8 ins. thick, the large stones being raked ahead into the sub-grade. The surface was sprinkled and rolled until compacted to a thickness of 6 ins. Figure 2 shows a finished portion of the Military Road surfaced with gravel.

Culverts.—In laying pipe culverts a firm bed was required and in the case of vitrified pipe a pocket scooped out under the bell to prevent excessive bearing at that point. All pipe was purchased by the road commission, the contractor being paid for unloading, hauling and placing.

The work was begun in September, 1913, and completed in August, 1914, requiring about one year for its accomplishment.

COSTS.

Table I gives an itemized statement of the distribution of final costs. It will be noted that engineering amounted to 3¼ per cent and extra expenses about 4 per cent.

The quantities and bid prices for the improvements of 20½ miles of road at a total cost of approximately $50,000 are given in [illegible] representative of gravel road construction in this section of Mississippi.

PERSONNEL.

R. C. Searcy & Co., of Greenville, Ala., was the contractor. C. L. Wood of Columbus, Miss., to whom we are indebted for the information contained herein, was the engineer for the road commission.

TABLE II. [illegible] OF COST OF GRAVEL ROAD CONSTRUCTION IN LOWNDES COUNTY MISSISSIPPI (WORK ACCOMPLISHED BY CONTRACT.)

Item.	Military road.	Wolfe road.	Aberdeen road.	Pickensville road.	Air Line road.	Robinson-Gilmer road.	Totals.	Percentage of total cost.
Length graded, miles.	4.84	1.83	4.62	4.03	.39	4.92	20.63	....
Length surfaced, miles	4.84	1.83	.70	4.03	.39	4.92	16.51	....
Width graded, ft	20	20	20	20	20	{22-½ {20-½	......	....
Width surfaced, ft.	14	12	14	14	14	14	......	....
Surfacing material..	Clay-gravel.	Gravel hauled from pits owned by country.				......[3]	......	....
Grading	$ [illegible]	$1,170.63	$3,78[illegible]	$ 2,934.82	$ 733.76	$ [illegible]	$15,968.15	32.1
Subgrade	789.44	257.33	......	653.33	64.18	808.89	2,573.17	5.1
Loading and hauling gravel	4,885.55	2,246.25	1,021.64	.....	[illegible].67	7,587.93	21,137.61	42.4
Spreading and rolling.	334.72	128.66	......[4]	326.67	17.83	404.44	1,272.32	2.5
Clearing	105.00	250.00	205.00	95.00	45.00	200.35	900.35	1.8
Fencing	36.00	15.00	352.00	92.00	....	48.80	543.80	1.1
Concrete culverts.....	1,177.39	......	464.06	.....	....	1,009.09	2,650.54	5.3
Drain pipe	117.10	88.50	183.95	116.50	....	86.20	592.25	1.2
Miscel. const. work...	350.99	186.81	45.24	132.57	.....	19.50	735.11	1.5
Tot. amt. of contract.	11,544.81	4,319.18	6,058.26	9,408.46	1,2[illegible].44	13,773.15	46,373.30[5]	92.7
Material purchased by commission	170.72	127.53	224.12	312.67	.....	898.50	1,733.52	3.4
Miscel. expense of commission	66.53	25.16	63.23	54.98	5.59	67.63	283.12	.6
Engineering	382.92	146.31	206.23	327.68	40.46	479.03	1,582.63	3.2
Total cost of construction	12,164.98	4,648.18	6,551.84	10,103.77	1,2[illegible].49	15,218.31	49,972.47	99.95
Average cost per mile	2,513.62	2,539.88	{1,081.55[1] {2,548.56[2]	2,507.11	3,294.86	......	......	...
Average haul of surfacing (miles)	1.23	1.91	1.60	1.56	1.00	2.00	......	....

[1] Grading, [2]Gravel surfaced, [3]Tishimingo gravel shipped in by rail for 1,200 ft.; the remainder from county pits. [4]Included in cost of hauling, [5]Engineer's estimate made June, 1913, was $46,255.05.

Methods and Cost of Road and Trail Construction in Alaska.

II.

(Continued from page 234.)

CONSTRUCTION CONDITIONS.

The greatest expense of road building, aside from the high cost of labor, teams and supplies, is the permanently frozen ground heretofore referred to. A layer of moss is often found covering great masses of solid ice, locally called glaciers. Nearly all the bottom land and lower slopes of the valleys is frozen, and a large portion of the higher slopes. The frozen ground varies in composition. In the Tanana Valley it is mostly mica schist; in the Seward Peninsula a peaty muck. Where the soil is [illegible] it [illegible] stripped and graded as usual and, in many other places good roads can be constructed by stripping the moss, allowing the ground to thaw and afterward grading it. Of course, side-hill slopes and ditches slide badly when first exposed to the sun, but this stops after a while. Where the ground is such that it will not form a good road even after grading, corduroy must be used. In this case it is best to leave the protecting blanket of vegetable matter so as to prevent the ground underneath thawing. Fortunately, where the ground is frozen the timber which grows on it has no tap-roots but grows on the surface above the ice, and it can be easily removed. In fact, it often happens that a fire burning out the moss will leave the roots of trees unprotected so that a very light wind will cause them to fall over. Trees should be grubbed out of the road bed and any hummocks of moss leveled off. A layer of poles is placed a foot or more apart, laid lengthwise to the road, the larger trees are used with the stiff branches cut off. Similar poles are then laid transversely to the road. Stiff branches are cut off, but the smaller branches are left. The corduroy is then covered with earth from the ditches to protect it from wear and to afford a smooth roadway. Moss or turf should not be put on the corduroy. As timber decays very slowly, this corduroy will last for a good many years. In the Seward Peninsula there is but little timber suitable for corduroy. Such roads are there very expensive. In some cases gravel is used, but it is often quickly beaten down into the muck underneath. An example of roads constructed under such conditions will be given in the description of a Nome local road.

Ditches, as a rule, must be wide and of gentle slope, otherwise they will wash and cut up badly, particularly in cases where the ground is nearly clear ice. In some instances ditches have cut clear under the corduroy and melting out pieces of solid ice has left considerable holes under the roadway. To prevent this a wide berm should be left. Where a road can be constructed along the bank of a river or small stream good underdrainage is provided; the only difficulty is in case the water should cut into the banks and destroy the road, as often happens. In any case, culverts should be provided not over 300 ft. apart.

Culverts were constructed of logs and poles, except in the treeless parts of the Seward Peninsula and near the coast, where sawed lumber was cheaper. Culverts made of stone were used in a few places.

Fig. [illegible] View Showing a Typical Concrete Culvert. The Bottom is Paved With Concrete.

SLED ROADS.

The long sled roads and trails connecting the different communities with each other and the outside world during the closed season (when the rivers are frozen up) are among the most valuable constructions made, although, of course, very much cheaper than wagon roads. The objections to using the surface of frozen rivers have already been given. The main requirement for a sled road is an easy grade. Protection from wind must be obtained if possible, both on account of the danger to exposure to wind in cold weather and on account of drifting snow. They should usually be put in the timber where possible, but never near the edge nor along the top of a ridge, nor along the side hills or the bottom of slopes on account of drifts. The best location is along a fairly flat timbered valley. Another element which it is practically impossible to foresee is the accumulations of ice, called "glaciers," that are built up during the winter, sometimes to a height of several feet by springs along the road. Sled roads can often be led directly across a pond or lake. In fact, one of the most important sled routes in the North crosses the upper end of Norton Bay and Golofnin Bay at considerable distance from land. However, such places and swampy land should be avoided if possible, as there is always a short period after the close of navigation in the fall before the freeze-up, and again in the spring after thawing begins, before the rivers are open to navigation. The hauling of supplies for parties working on the roads and for the road-houses along them, which are necessary stopping places for travelers, makes it desirable to use good ground if possible.

The requirements for sled trail construction are very similar to those for sled roads, although not so strict. The same may be said of pack trails as compared with wagon roads.

The temporary staking of trails is done as heretofore stated, merely to mark the way to keep people from getting lost. A great deal of this was done, particularly in the Seward Peninsula. Poles or laths were usually stuck in the ground each winter, about 100 ft. apart, with streamers of red cloth. Staking with permanent iron stakes was done for the same reason, but upon trails that were much more used. At first an iron pipe was used with an iron flag riveted or welded to it, but it was found that this flag would break off in a short time, due to flapping in the wind. For this reason, on later work of this kind, the flag was fixed so that it would revolve around the rod or pipe. This method worked very well, although it was somewhat more expensive than the previous one.

Bridges were usually made of native timber hewn into shape near the place of construction, or bought at sawmills, a few of which exist in the territory. Near the coast, bridges were sometimes constructed of wood or steel brought from the "States." It was my experience that the wood found in Alaska was about two-thirds as strong as wood from the same species of tree in the "States."

Surveying encountered no special difficulties, except that due to the extreme difficulty of getting over the ground. This work, as well as the construction of sled roads and trails, was often done in the fall, sometimes necessarily so; after ground was frozen so that it was possible to travel, but before the snow came. This had the disadvantage that there was no forage for the animals, as the very first frost seems to take all nutriment out of the grass in most places. However, it is not often that animals can be subsisted by foraging. In a few places around Cook Inlet and the Tanana and White River valleys horses can live on the vegetation the year round. Where it is possible to travel, surveying was usually done, as was construction work, in the summer. The route from Washburn to Donnelly, part of the Valdez-Fairbanks route, was surveyed in the middle of winter with the thermometer at times as low as 40 degrees below zero.

The wagon roads constructed, with a single exception—the Valdez-Fairbanks road—and the greater part of the sled roads and trails were in the nature of local roads leading from various coast or river towns to the mines in the interior, or from various central supply stations to the mines in the immediate vicinity.

No attempt will be made here even to name the many routes constructed in Alaska or even to describe all of the more important ones. A few among the more important will be briefly described, omitting so far as possible details common to all or common to work in the "States." The general features existing in all parts of the territory have already been given.

The Fairbanks local roads are a system of wagon roads 72 miles in aggregate length, leading from Fairbanks to the mines in the vicinity. They cost an average of about $1,350 per mile in addition to the local road tax, not counting overhead charges.

The Fairbanks-Fort Gibbon sled road, 160 miles in length, was mostly constructed in the fall of the year after the ground had frozen, owing to the great amount of wet ground to be traversed. Clearing was heavy. The cost was $58,440.98, or about $365.00 per mile. The road was cleared to a width of 16 feet and graded where necessary to a width of 8 feet. Bridges and culverts were 10 feet wide.

Sled trails are narrow, and less carefully constructed than sled roads, and the cost is correspondingly cheap, running from a few dollars to $61.00 per mile in the interior and to as high as $373.00 per mile near the Pacific Coast. Pack trails along the coast cost as high as $758.00 per mile in one place. Trails were made from 6 to 8 feet wide, depending on conditions.

In the lowland and swampy sections the cost of construction and maintenance is very high. Corduroy is expensive, as it must be hauled in from the outside, and tundra fires are numerous and apt to burn the corduroy. The soil, being largely peat, will of itself burn in many places, often smouldering for many days, as rains are infrequent. The most important and most expensive of these roads and, for its length, the most important in Alaska, is the Nome-Bessie Road, 3.3 miles long. This road was constructed in 1906. The roadway was built 22 ft. wide between ditches. The ditches were ploughed in the spring when the frost was 2 or 3 ins. from the surface and the material thrown to the center, with Fresno scrapers and road machines. After the sun had thawed the ditches the road was regularly crowned to an elevation of from 1 to 2 feet above the surrounding country. Gravel having been found unsuitable, as it ground into the muck, great quantities of gunny sacks (Nome's supply of coal is brought in gunny sacks) were laid on the ground, forming a sort of mattress and the gravel placed on top of this. A great deal of corrugated iron, the result of a fire in Nome in 1905, was used in the same manner; but the iron worked out from under the gravel in places and became dangerous to traffic. Consequently, most of it was removed. Willows were used in a few places, but did not give as good bearing surface as the sacks and were not as cheap. This road gave excellent service up to 1912, when it was decided to build a gravel road with telford foundation.

SURVEYS.

The cost of surveys is hard to separate from the cost of construction in many cases for, as has been said, the construction crews often followed the surveyors very closely, sometimes in fact belonging to the same party, and it was not possible to keep the cost separate. The location and survey of the Donnelly-Washburn sled road, 55 miles, part of the Valdez-Fairbanks route, cost $4,572.77. The location and survey of the Fairbanks-Hot Springs sled road, 160 miles, cost $3,184.81. These are important routes and the cost of the surveys is almost as great as for wagon roads. On the Pacific Coast costs are very high, for the reasons already given.

The preliminary reconnaissance for the sled trail through Kaltag, Rainy Pass and Knik already described, cost $5,813.74, covering the total expense from Nome to Seward, a distance of about 750 miles. Part of the country traversed was unknown, and, as the work had to be done in winter on account of the impassable nature of the ground, quite an elaborate equipment had to be provided. Transportation was by dog sled.

The most important land route in Alaska is that from Valdez on Prince William Sound, an ice-free port the year round, to Fairbanks, the great mining center of the interior, on the Tanana River. From Fairbanks roads or trails connect with practically every town or mining camp of any importance north and west of the Alaskan Range below Eagle. The ordinary way of reaching Eagle and the Forty Mile mining district, which is connected with it by a sled road, is by way of Dawson; although the Yukon River to Circle makes fairly good traveling and Circle is connected by sled road with Fairbanks.

Nearly all winter travel and mail from the whole interior and Bering Sea and Arctic coasts follows this route, as does nearly all overland travel in summer.

The history of this road is one of gradual evolution. At first nothing but a passable winter trail was attempted. Later this was developed into a sled road, and in 1909 the actual construction of a wagon road was begun. By the summer of 1912 this roadway was in very good shape for wagon traffic throughout its length.

The length of the wagon road from Valdez to Fairbanks is 379.5 miles. The distance by sled road is about 25 miles shorter. The Willow Creek-Chitina branch is 49 miles, and the Donnelly-Washburn cut-off 55 miles long. There are a number of short sled road cut-offs made to shorten the distance or to avoid parts of the road not best suited to winter travel. The Donnelly-Washburn cut-off was constructed in 1907 at a cost of $16,881.74, about $307.00 per mile, including the survey. Maintenance to 1912 was $3,188.11. The Willow Creek-Chitina Branch was started in 1910 to connect with the Copper River Railroad. I give here the general specifications for this road as a fair sample of good sled road construction.

SPECIFICATIONS FOR SLED ROAD CONSTRUCTION.

Clearing and Grubbing.—The roadway for 8 ft. on either side of center line will be cleared of all trees, brush and logs, the same to be cut close to the ground. The roadway will be grubbed for a width of 5 ft. on each side of the center line.

Grading.—Where the ground slopes 1 ft. in 10, or over, the roadway will be graded to a width of 10 ft., which width will be increased on turns.

Staking.—In open flats stakes will be planted at intervals of not to exceed 100 ft. on alternate sides of the trail. The stakes will be substantial poles at least 8 ft. in length firmly planted in the ground.

Bridges and Culverts.—Bridges and culverts will be constructed where necessary to give a smooth and even roadway. Culverts are to be given a width of 12 ft. Width of bridges will be 10 ft.

Five spruce stringers should be used in each span sufficiently strong to bear heavy, four-horse loads. Decking should be not less than 5 ins. in diameter at the small end. For spans over 15-ft., truss bridges should be constructed according to Alaska Road Commission's standard plans.

Corduroy.—Corduroy will be used where necessary on mucky or marshy ground. The width of corduroy to be 10 ft.

Mile-Posts.—Mile-posts to be set at each mile from the railroad station. They should be of squared or sawed timber or squared standing trees, at least 4 ins. square, and will be marked with neat black figures 3 ins. in height.

The road must be completed throughout the present season if possible, and any work required by these specifications [illegible] this year if necessary to the minimum required for double-ender sleds.

Seventeen miles of this road have since been converted into a wagon road by proper ditching and draining. The cost was, for sled road, $670 per mile; for wagon road, $1,132 per mile. The main road follows in general the route of the Abercrombie Trail and the Valdez-Eagle survey as far as Gulkana, although very little of the actual line previously laid out is now used. Prior to 1909, about $230,000 had been spent on this route. A large part of this should be charged against the wagon road, as a good deal of the expense was for work of use for a wagon road, and part of the remainder was in the nature of location and experiments.

TABLE I.—MILEAGE AND AVERAGE COST PER MILE OF ROADS CONSTRUCTED IN ALASKA.

Item.	Mileage.	Cost per mile.
	—Up to 1912—	
Wagon roads	824	$2,[illegible]
Winter sled roads	559	[illegible]
Trail	1,[illegible]52	1[illegible]
	—1913—	
Wagon roads	33	$2,489.68
Winter sled roads	18	275.84
Trail	614	90.44

Detailed Cost of a Recently Completed Concrete Road Near Aurora, Illinois.

A section of concrete road on State Aid Route No. 1 in Kane County, Illinois, begun on April 15 was recently completed. About one-fourth of the grading on this road was completed at no expense to the road, being accomplished with donated labor. The country traveled is somewhat rough, according to the official bulletin of the Illinois Highway Department, from which the information herein is abstracted, and considerable work on the old gravel road, which was replaced by a concrete surface, was necessary in preparing the subgrade. The road was built according to the standard specifications and cross sections for state aid roads, notes of which appeared in the issue of Engineering and Contracting for July 29, 1914.

Expansion joints were spaced 50 ft. and steel corner plates were used for protecting the edges of the joints. In proportioning the concrete a definite graduation of aggregate sizes to reduce the percentage of voids to a

TABLE I.—DETAILED COST OF CONCRETE ROAD NEAR AURORA, ILLINOIS. (CONTRACTOR'S PROFIT AND OVERHEAD CHARGE EXCLUDED.)

Amount of pavement laid..........................1471 ft.; sq. yds., 2942
Width of pavement..........................18 ft. Standard Section No. 6
Thickness of pavement, center, 8 inches; sides, 6 inches.
Length of haul for materials..............¼ mile
Cost of cement per bbl. f. o. b. sidings.....$1.12
Cost of sand per cu. yd. f. o. b. sidings..... 1.20
Cost of stone per cu. yds. f. o. b. sidings.... 1.20
Amount of cement used per sq. yd. of pavement ....0.36 bbl.

Rate of pay for mixers, 37½ cents; finishers, 50 cents; form setters, 55 cents; rough labor, 35 cents; teams, 70 cents per hour.

Item.	Cost per sq. yd.
Superintendence	$0.077
Excavation, shaping road bed and trimming side roads	0.120
Loading sand and stone	0.034
Hauling sand, stone and cement	0.077
Mixing and placing concrete, setting forms and filling joints	0.150
Covering, seasoning and cleaning concrete	0.027
Watchman and miscellaneous labor	0.017
Sand and stone f. o. b. siding, including demurrage	0.392
Cement f. o. b. siding, including demurrage	0.406
Expansion joints f. o. b. siding	0.037
Coal and oil for mixer and miscellaneous supplies	0.005
Forms and other lumber	0.014
Freight on equipment	0.029
Total cost per sq. yd.	$1.385
Cost per sq. yd. excluding excavation	1.265

minimum was required an[illegible] of cement used to the cubic yard of concrete based on 45 per cent voids in the coarse aggregate. Twelve complete turns of the mixer were required before discharging the concrete from the mixing drum.

The costs given in Table I do not include contractor's profit or overhead expenses, such as engineering and inspection. The former is difficult to estimate, the latter ordinarily varies between 5 and 10 per cent.

Comment on Bituminous Terminology.

To the Editors: In your issue of Sept. 2, 1914, you quote the following paragraph from the tentative report on terminology made by the committee on standard tests for road materials of the American Society for Testing Materials:

> Asphalts: Solid or semi-solid native bitumens, solid or semi-solid bitumens obtained by refining petroleum, or solid or semi-solid bitumens which are combinations of the bitumens mentioned with petroleums or derivatives thereof which melt upon the application of heat, and which consist of a mixture of hydrocarbons and their derivatives of complex structure, largely cyclic and bridge compounds.

If anything like this definition is to become the final official declaration of the American Society for Testing Materials time and space can be saved and practical purposes will be equally well served by amending the paragraph so as to read:

> "Asphalts: Anything that is black and sticky."

Asphalt (not asphalts) for at least two generations has meant solid native bitumen. It is only to serve the purpose of the producers of oil asphalts (who are effectively represented on the committee that made the above recommendation) that it is now proposed to place native and manufactured asphalts on the same footing without differentiation as to their origin or characteristics. Only confusion can result from this attempt, for two different things can not be made the same thing simply by the sayso of any committee, however composed. No committee could make cotton and wool the same thing by drawing a definition for "fibers" or "fabrics" which included these two products.

In regard to this matter the German precedent is illuminating in several respects. Dr. E. Holde, Chief of the Imperial Testing Station, Berlin, and author of "Untersuchung der Kohlenwasserstoff Oele und Fette," makes this declaration on page 275 of the 1913 edition of his book:

> In order to arrive at a uniform terminology for pitchy and asphaltic substances it is desirable to reserve the name "asphalt" for such products as are found in nature . . . The substitutes (to be regarded as inferior) which result from the working up of petroleum should be designated "artificial asphalts," or better still, pitches, such as petroleum pitch, grease pitch, brown coal tar pitch, coal tar pitch, etc., according to their origin.

This has the status of official nomenclature in Germany and how definitive it is may be gathered from the fact that proceedings are brought under the law of unfair business competition and misbranding to restrain and punish the designation of oil asphalts as "natural asphalts."

There is no discrimination in this policy. It is well recognized that each asphaltic or bituminous product has its place; the error and the wrong consists in the attempt to put all of them into one class or at least to include all of them in one definition which, like the Mother Hubbard, wrapper, covers everything and touches nothing.

Very truly yours,

Daniel T. Pierce,

Executive Assistant, Barber Asphalt Company.
Philadelphia, Pa., Sept. 3, 1914.

The question of bituminous terminology is yet unsettled and much confusion exists in regard to the correct usage of names for different materials and processes. In the end common sense and convenience will rule, but at the present time the usage of terms is on no such basis. The matter is worthy of discussion.—[Editors.]

Comment on Road Design; Relative Crowns on Grades and on Level.

To the Editors: I have read with considerable interest the article of your issue of Sept. 9, entitled "Some Practical Notes on the Design and Construction of Bituminous Surfaced Roads in England." This paper is very instructive and the closing statements, anent the variety of materials and traffic conditions, contain some sound advice which might well be extended into a condemnation of the practice of standardizing highway construction regardless of materials, traffic, conditions of subgrade, prevailing winds affecting snow, etc. However, there are two points which I wish to mention, the first being the omission of important information.

The advocated use of long radi and superelevation on curves, together with the carrying of roads around towns rather than through them, indicates that the article refers to *speedways* instead of *highways*. And we might well ask whether the roads under discussion are being constructed for utilitarian purposes to supply the needs of the people of the community of whether they are designed as fresh air boulevards for the pleasure of automobile parties. Each type of road has its purposes, but the reader could have a clearer conception of the entire enterprise if he knew what use is to be made of these highways. And along with the suggested information some readers would appreciate a statement as to whether the people who use the improved thoroughfares are the ones who pay the bills. These aspects of highway building are too frequently overlooked in America as well as in other lands.

The second point to which I wish to refer is the relative crowns on grades and level stretches. I have always adhered to the opinion that the greater crown was required on grades, where the slightest beginning of a rut will serve as a guide to escort the rain water down the middle of the road unless there is a relatively heavy transverse slope. And, when the road surface is such as will be affected by running water, this first flow will increase the rut. One objection to heavy crowns is the rutting caused by allowing horses to walk in the center where there is even footing, the tendency to rut depending upon the amount of crown. But this objection is greatly reduced on heavy grades where drivers involuntarily pull their horses out of the tracks while holding back on the down grade. There is less likelihood of bad ruts with motor than with horse-drawn vehicles owing to the human factor in guiding. But, whatever the use to which the highways are put, there is surely more need for crown on the grades than on the level stretches.

Very truly yours,

Leonard C. Jordan.

1502 Union St., Brooklyn, N. Y.
Sept. 14, 1914.

With regard to the laying out of roads passing through villages and small cities and the improvement of by-pass or shuttle roads avoiding business sections, much may be said pro and con. The subject is of small importance in many sections of the country but in the improvement of through roads over which a heavy motor traffic will pass it is, at least, worthy of study, especially since future developments may demand its consideration.

The practice of increasing road crowns on steep grades is quite generally accepted as advisable, certainly where a macadam or gravel surface is used. With other types of surfacing its features have been little discussed, especially with regard to the danger of the creeping of bituminous surfaces.—[Editors.]

Engineering Experiment Station Opened.—Final organization of the Texas engineering experiment station, to be conducted at the Agricultural and Mechanical College, College Station, Texas, by the engineering school, has been completed. Research work along lines of interest to engineering industries will begin with the opening of school on Sept. 22. Especial stress will be laid on highway engineering work, and experiments in cement and concrete, internal combustion engines, coal, oil and gas tests, etc., are planned.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume VLII. CHICAGO, ILL., SEPTEMBER 30, 1914. Number 14.

"How Can Engineers Best Utilize the Technical Journals?"

Space is claimed in this issue by an article bearing the title quoted above. We recall no other article which answers the query so well. In particular the author brings out two broad facts which are of paramount importance to subscribers for engineering journals. The first fact is that the engineering journal is essentially a working tool. The second fact is that unless the engineer works this tool to its highest efficiency he is failing to realize as he might on the capital invested in it.

The article suggests how this highest efficiency may be secured from the engineering journal. Probably many readers have other methods which suit them better. But whatever method may be selected, the main point is not altered; the engineering journal is useful to the subscriber exactly in proportion to the efficiency with which he develops its usefulness. It is useful to subscribers of different ages and degrees of experience in different ways, but its main usefulness to most readers is as a repository of usable data and suggestions. The reader who chooses for his purpose a quick and certain means of getting from this repository the information needed is the one who makes the most efficient use of his engineering journal because to provide the means for quick and certain reference intelligent previous reading is essential.

The Planning of Engineering and Contracting for Easy Reference.

The engineering journal is a reference book of current engineering thought and data. Admitting that it contains thought and data of worth, the value of the engineering journal as a reference book lies in the readiness with which the sbuscriber can locate for preliminary reading the information in which he is most interested and in the ease and precision with which he can record for future consultation portions selected during preliminary reading as being of particular value.

The planning of an engineering journal to meet the requirements just set forth is not entirely simple. Its subscribers have engineering interests of wide variety. The lines of engineering that they practice are, even when broadly classified, quite various. In each line there are practitioners of various ages and various degrees of experience. The necessities of each line of practitioners in the way of engineering information are different, and within each line of practitioners the necessity is different for each individual according to his experience and knowledge. Obviously no engineering journal can be planned to meet individual necessities completely at all times; it can be planned only to meet the necessities of classes of practitioners.

The planning of ENGINEERING AND CONTRACTING has been conducted with special pains to facilitate rapid preliminary reading and quick and precise recording of items for future reference within the limits indicated in the preceding paragraph. All articles are civil engineering articles. This is the first classification. All articles are either (1) technical articles or (2) construction news articles. This is the second classification and two separate sections of approximately equal volume are provided in ENGINEERING AND CONTRACTING for this classification. All technical articles are devoted to economics of civil engineering design and to civil engineering construction and operation methods and costs and all current news articles are confined to works offering opportunities for business or employment. These restrictions make the second classification definitive and precise. All articles, technical and news, are classified under the twelve headings: (1) Editorials, (2) Water Works, (3) Sewers, (4) Roads and Streets, (5) Bridges, (6) Buildings, (7) Drainage and Irrigation, (8) Rivers and Harbors, (9) Railways, (10) Book Reviews, (11) Construction Plant, (12) General. This is the third classification.

The classifications described make the location of articles for preliminary reading a matter of a moment's search. The recording by engineers of selected articles for future reference follows one of two general plans. One plan is to index the selected articles on cards, and bind and file the journal indexed, as successive volumes are completed. The second general plan is to clip the articles selected for future reference and file them in accordance with some system found satisfactory by the subscriber and reference to which is had by index cards or indexed envelopes or folders. To meet the requirements of either plan, each technical article in ENGINEERING AND CONTRACTING has prepared for it and printed at the same time a complete index card. These cards are assembled with ample margins for clipping on the leaf which separates the technical article section from the current news section of each issue. Clipping this leaf does not mutilate either technical or news sections, either or both may be bound in volumes or may be clipped for the purpose of filing selected articles. If volumes are bound the index cards for selected articles are clipped and pasted on standard library index cards which are then filed. If instead of binding, selected articles are clipped and filed in envelopes or folders, the corresponding index cards may be clipped and pasted on the envelopes or folders themselves, or on standard cards, as when the journal is bound in volumes.

An examination of the index cards as printed in this issue will show their adaptability to the formation of a reference index. Each has an index subject heading which enables any office assistant, once the cards desired are check marked, to clip, paste, alphabetise and file. Space is provided for removing this printed index heading, leaving the remainder of the card intact, should the subscriber prefer to select and write headings conforming to the classification adopted to meet his special purposes. For the average engineer who desires simply a ready reference index to certain articles the cards as printed serve all purposes and, once the engineer has read and selected the articles wanted and check marked the corresponding index cards, fifteen minutes by anyone with paste and scissors are sufficient for preparing and filing the index cards for an issue of ENGINEERING AND CONTRACTING.

Granting, as we are justified in doing, that continued subscription for ENGINEERING AND CONTRACTING demonstrates that the subscriber finds value in its articles, the preceding presentation of the way in which he can best utilize that value requires no apology. The best that the editors or the expert contributors of engineering journals can do for their subscribers counts for little unless these subscribers are alert to take advantage of what is done. Aids, such as have been outlined above, to the subscriber in performing his part make it a comparatively easy task.

Data on the Design of Domes.

Although a number of important domes have been constructed in this and foreign countries our engineering literature contains few definite data relative to the design of these domes. On this account an engineer has little information to guide him in the design of this type of construction. In this issue we are publishing an exceptionally complete article describing the design features of the large steel dome of the Palace of Horticulture, San Francisco, Cal., this structure being one of the imposing buildings now under construction for the Panama-Pacific International Exposition. Not only are the methods and assumptions used in designing the steel dome and its substructure given in detail, but the results obtained by various methods applicable to dome design are compared. The computations and the procedure followed in the design are given in sufficient detail to enable an engineer readily to follow the discussion. This dome, which is believed to be the largest in existence, has a diameter of 150 ft. and forms a semi-sphere. It rests on a series of girders which are in turn supported on eight steel piers, each pier having a height of 65 ft. and consisting of four braced columns. The authors of the article and those in charge of the design are to be commended for having made available such important data on the design of steel domes.

Engineers and Other Men.

Occasionally the public utterances of engineers smack somewhat of the pharisaical. Knowing engineers as well as he does it is not strange that the engineer should respect his professional brethren, but he should avoid the narrow view of regarding them as the repositories of all the virtues. He should know other men better. Men of good character and of honest purposes are much alike whether they serve society as engineers or in other capacities. An illustration of the character of statement which we have in mind follows. It is from a presidential address before an important engineering society. To quote:

> What one quality, think you, commends the engineer for positions of trust and responsibility? It is that he is true to the interests of those who employ him, true to his devotion to duty, fair in his treatment of those dependent upon him, and just in the settlement of disputes and differences.

Much fault might be found with this quotation; to begin with the idea of competency is not even suggested. The thought should be ever present, in addresses of the kind from which we quote, that it may be affirmed of all men with equal truth that "it is not sufficient to be good unless one is also good for something." The possession of integrity and the other virtues and the lack of ability may conceivably be more disastrous than the converse in matters of an engineering nature.

It was another thought partially implied, however, which attracted our attention to the quoted paragraph. While it probably was not the intention of the speaker to suggest that only engineers are possessed of the noble attributes enumerated, such an inference might readily be drawn from his unqualified statement. It is true, nevertheless, that many engineers are so impressed, for example, with the honesty of engineers, that they make statements which would lead one to suppose that the profession has a corner on honesty. This is as far from the truth as to say that culture is absent from such utilitarian works as those which occupy the attention of the engineer.

Again, many engineers make very sweeping statements about the dignity of their profession and of their service to mankind. Granting the substantial truth of such statements it is well to bear in mind that others, even those in lowly callings, have their ideals and are actuated by lofty and unselfish impulses.

It is farthest from our intention to decry the simple virtues which date back at least as far as the Sermon on the Mount. Neither is it our purpose to deny that engineers possess these virtues, but we do deplore the narrow thought or careless expression which holds the engineer to be their sole possessor.

The Real Task of American Manufacturers in Developing South American Trade.

From the avalanche of assertion and comment concerning trade opportunities in South America this fact protrudes: The problem of South American trade development by manufacturers in the United States is how to substitute themselves as merchants in place of the temporarily deposed German, French and English manufacturers. The American manufacturer is easily disposed to assume, since he makes for sale merchandise and machinery wholly as good as does the European manufacturer, that the American merchandise and machinery are just as salable. This is not so, and because it is not is one very evident reason why American trade in South America is less, and often very much less, than that of European commercial nations. And unless American manufacturers readjust their viewpoint and reform their practice it will be the reason why Latin America will abandon the traders from the "States" and return to European markets just as soon as the warring nations resume business.

No scheme of merchant shipping and no perfection of banking facilities will make South American merchants buy of firms which do not properly invoice goods in the language of the country of destination in accord with custom house regulations, which do not properly pack for protection and transportation the goods shipped, which do not conform strictly to specifications as to character of goods, and which do not adapt themselves to the business customs and peculiarities of the Latin merchant. These are assertions which cannot be successfully disputed. They come from every consular officer sent to South America; they are repeated by every engineer and business man who has been a resident of South America; they are affirmed by the native merchants of every South American country.

Circumstances have brought about a condition more favorable than has existed for years, perhaps more favorable than ever will exist again, for the development of American trade with Latin America. American manufacturers in plenty make the classes of goods bought by Latin Americans. They do not, however, make these goods in as salable form as does the European manufacturer. They must do this and they must learn to adopt the methods of solicitation, of extending credits, of invoicing shipments and of special manufacture to suit the local demand. They must become as a few American firms have become and as European manufacturers very generally have become, South American merchants. This fact is the most prominent fact brought out in the present wide discussion on South American trade opportunities.

The Relative Positions of the Engineer and the Architect in Designing Commercial Buildings.

It is difficult to define the true functions of the engineer and the architect in designing and constructing modern commercial buildings. Before the development of our high steel frame buildings, and before reinforced concrete had attained prominence as a structural material, the problem of building design was essentially an architectural one. If the engineer's services were required at all it was only in connection with the foundations, and even then he was consulted only where conditions made the design and construction exceedingly difficult. Due to the development of our steel frame and reinforced concrete buildings, however, conditions have changed and the design of these structures is becoming more and more an engineering problem. In such buildings strength and durabilty are of greatest importance, and to insure these qualities in building construction requires the services of engineers. It is only natural, due to the gradual development of commercial buildings, that in present building organizations the architect is supreme, while the engineer is placed in a subordinate position. In the more prominent architectural firms there have been established engineering departments, in charge of engineers who sometimes are given the dignified title of "Chief Engineer." In most cases the building is planned almost entirely by the architect, and the engineer must make his steel or reinforced concrete design conform to the architectural design. In some cases this has resulted in structural monstrosities and in exceedingly expensive structural designs.

If the architect is sufficiently broad-minded, and if his training and experience have been of the right kind, he will have felt the advisability of permitting the engineer to assist materially in planning the building. However, as the final decision lies with the architect the tendency is to over-emphasize the architectural features to the detriment of the engineering design. In making this statement it is realized that the architectural treatment of a building is of great importance, but in most commercial buildings safety and permanence are essential, while low cost is often absolutely necessary if the structure is to be financed. The latter factors give increased emphasis to the engineering design. Most architects have received their initial technical education in our universities, and the present tendency of architectural schools is to give less emphasis than formerly to mathematics and to those subjects which teach fundamental engineering principles—and more emphasis to the aesthetic design.

We believe that the design of many classes of commercial buildings should be in charge of engineers and that the architectural features of such buildings can well be taken care of by architects in the employ of these engineers or by engineers who have gained sufficient knowledge of architectural principles to enable them to design structures in which aesthetics has been duly considered. We also believe that engineers have not been sufficiently alert to their interests in permitting conditions to come about such as now exist in the building field. Furthermore, the engineer's position will not be improved unless he takes more active steps than formerly to protect his interests. Due mainly to the organized activity of architects, laws have been passed in some states which require that all building designs must be in charge of and signed by licensed architects. This has acted further to bar the engineer from activity in the building field. It is, of course, true that engineers can qualify as licensed architects by passing the required examinations. Under existing conditions, however, this is difficult, as the board of examiners is composed of *architects* —men who frame the examination questions from an architect's viewpoint.

One serious phase of the question, from an engineer's standpoint, is that, under existing conditions, contracts for building designs and superintendence naturally come to architectural firms, the business dealings being between *owner* and *architect*, the engineer getting a small share of the business from the architect—not from the owner who furnishes the necessary capital. We believe there is a legitimate field in building work for both the engineer and the architect, but we are convinced that in designing certain classes of commercial buildings the engineer should be supreme. The latter is certainly neglecting a promising field when he does not exercise his right to negotiate directly with the owner in obtaining contracts for building designs and superintendence.

GENERAL

How Can Engineers Best Utilize the Technical Journals?

In [illegible] reading matter comes to a man from every side even the specialist must plan his reading so as to secure the greatest benefit from the literature pertaining directly to his specialty. The man who would cover his field and keep up with the times must devise a plan of reading to suit his interests, his time and his income. The present article, which is the major portion of a paper by John W. Alvord, consulting engineer, Chicago, before the annual convention of the Federation of Trade Press Associations, discusses the general aspects of the reading problem, from the engineer's standpoint, and suggests reading plans for engineers of various ages and interests.

In 1880, when the writer first commenced to take some interest in technical journals, such publications in this country were relatively few. Since 1880 we have added to civilization the electric light, the electric railway, the telephone, the phonograph, wireless telegraphy, the X-ray, high steel buildings, reinforced concrete, the explosion motor, steam turbines, high-duty pumping engines, central power stations, hydro-electric plants, automobiles, flying machines, bacteriology, filtration, modern sewage disposal and scientific sanitation, and the engineering world has been practically made over in about 35 years.

Engineering literature has had to specialize, divide, concentrate and keep pace with this rapid movement. Of necessity it has grown voluminous, and the problem of the engineer in 1880, which was to treasure and index almost every scrap of printed matter on any engineering subject whatever that came his way, is today to sort out, discard and eliminate that which he can no longer use, and limit himself to the inspection and reading of that which bears principally on his selected professional specialty. Many do not succeed even in doing as much as this with the technical journals that flood in upon us in our busy days.

That we cannot keep abreast of the times without reading the engineering journals is obvious. That if we carefully read all the engineering journals in our chosen specialty we would have no time left to earn a living is easily capable of demonstration. What, then, is the proper attitude to adopt toward this ever-increasing flood of information that pours in upon us so relentlessly, week after week, month after month and year after year?

If we look about us to see how our fellow engineers solve this matter we shall find a great variety of attitudes toward the problem. Some engineers simply do not take engineering journals, reading one occasionally here and there as opportunity offers. Others take all they can afford to take and let them pile up around the office, often unopened and unused. Others still limit themselves to a select few, which they carefully bind and shelve. Still others read journals when they can, and throw them away when they move on. As a rule, however, the engineeer prizes his technical paper, and endeavors in some ill-defined and formless sort of fashion to preserve its information for future use. Generally he fails to find any practicable scheme which makes his rapidly accumulating material of much value to him after it has once passed under his eye, and for a large number of engineers technical journals are only professional newspapers with which to idle away an hour or so and to satisfy their curiosity. That their value is something much more than this, or should be more than this, is so apparent as to need no denial.

The problem of the engineer with his technical paper is much affected by his age, station and aim in life. To the man who is in engineering only to get money and more money, the engineering journal is a newspaper, in which he may notice mainly where there are better jobs than his own that may be sought after and perhaps obtained. To the man who is anxious to fit himself every year of his life for something better, it is an opportunity, quite unequaled many years ago, for a great variety of study. To the young engineer, the engineering journal, properly read and noted, is part of a post-graduate course in engineering. To the middle-aged man, it is a mine of data, bearing in all sorts of ways on his work, and to the mature specialist only does it begin to become burdensome by its repetition of experience and its volume of matter on subjects which have already, to him at least, been well digested.

Let us see if we can outline how each of these classes can get more profit out of the matter contained in the engineering journals than do the careless or indifferent, who, after their journal is once looked over, let it go to waste or idleness.

The young engineer and the college graduate need, most of all, practical experience. It is safe to say that engineering literature will never have any proper perspective for him until he has been connected in some capacity with engineering work himself, be it in ever so modest a capacity.

With the actual doing of engineering work, however, should come contemporaneously the reading of technical journals, particularly along the lines in which he is working. Nothing can be more instructive, broadening and enlightening to a man doing a particular kind of work than reading about similar work *at the same time.*

It follows, therefore, that the young engineer should, as early as possible, take at least one good, first-class engineering journal, and own it himself; bind it, if he can afford to, but lay it away in an orderly manner, in any event. If he can afford two journals, so much the better, especially if they are selected so as to widen his outlook.

Many young engineers are omnivorous readers by instinct and curiosity. Some cram on technical literature, largely because their professors at college and others have given them the good advice to read engineering journals as a habit. Others cannot bring themselves to read much "shop talk" out of hours, because they naturally prefer recreation and mental relaxation. There is, however, a happy medium. It is to be doubted if laborious reading of all kinds of engineering articles all the time is advisable for anyone. Mere quantity of reading is mentally detrimental. If one might advise, it would be to suggest enforced systematic reading of all articles particularly bearing on the line of work the reader is immediately engaged upon, and the optional reading only of such other articles as interest him. This ought not to be much of a task. In course of time, as his experience broadens, engineering reading will become less burdensome and more interesting because its relation to practical matters will be more and more appreciated and the discriminating use of engineering literature better understood. Of course all this applies to engineering societies as well, but that is another story.

In the matter of indexing for the young engineer, much must be left to the judgment and taste of the individual. The engineering indexes are very complete and useful in these modern days. The mind itself is a wonderful indexer. It is safe to say the average intelligent man reading an article which impresses him as useful and valuable can, without effort, remember for many years after the name of the journal and the approximate year in which the article appeared.

It is probably not wise for the young engineer to indulge extensively in card indexes, filing systems and the like for topically arranging his available engineering journal articles. Few men know very early in life where fate and interest will land their future attention, and filing systems and special indexes are expensive and time consuming, and when indulged in without definite aim nearly always quickly become too voluminous and thereby useless. Many a young engineer has spent many weary hours filing and indexing, only to abandon his system later on in despair at the quantity of material he early collects and the difficulties of making it quickly available.

If any suggestions are made along this line, it would be to start a loose leaf letter size ($8\frac{1}{2}$-in. $\times$ 11-in. page) notebook, and note in it (separate pages for separate subjects) only what appears to be extremely useful, either in exceedingly brief abstracts from engineering articles or diagrams, costs, etc. These notes will be most useful if they are confined to that kind of work in which the compiler is immediately engaged and has on his mind at the time, or, at the most, work very similar to his own which has perhaps had his personal inspection.

If any such book is started, it is highly desirable that it be of letter size, because that is nowadays the working size to which all sorts of documents, engineering reports and estimate work are approximating. Pocket notebooks, card indexes and odd sizes of notebooks should be avoided, if possible, as likely to be finally abandoned. The letter size fitting the stock office furniture and ordinary typewriter is much more likely to endure with the average man as a permanent system.

The young engineer is tempted to read much about large enterprises—the Panama Canal, big bridges, astonishing tunnels, great dams. This does no harm and probably holds his interest for the time being. Gradually he learns that, for him at least, the chief value of the technical journal does not lie in its dramatic side, necessary as that may be for our general information, interest and pleasure, but its chief value lies in a fund of small things which make up routine work of the ordinary everyday job. These are to be watched for and noted as practically useful to the average man.

We next come to the man in early middle life, actively engaged in his profession, and note at once that his problem with the technical journal is the absence of "time." Absorbed in a multitude of responsibilities, harassed with unexpected difficulties, worn out at night with the long day of strain, how shall he derive any useful good from the multitude of journals which his more ample income can readily afford, but which pile high on his table after every brief absence from the office and constantly aggravate him with their temptation to neglect other duties? Whether or no such an engineer shall make any effort systematically to assimilate, file and study current technical journals depends in part upon the nature of his routine. If he is largely engaged in administrative work, or is a salaried officer in a large enterprise with a comparatively limited range of problems, or a limited call for miscellaneous data, he may generally be content with a cursory examination of the engineering journal such as will keep him qualified on his undertaking, and the preservation of such journals in bound form, with the standard published indexes. If, however, he is entering upon novel work, or work presenting a great variety of problems, overlapping into a great variety of fields, ambition will compel him to do more than this, and some form of special indexing will appeal to him more or less strongly as he feels the need more often for research in up-to-date material.

The average editor can judge of a technical article with only a brief inspection—a sentence here and there, a headline and a moment's reading of the summary and conclusion. Long familiarity with matter of a similar character gives him the assurance that he can detect in this rapid review anything novel, new or original, and can fairly pass judgment upon it in a general way. The working engineer who has had some experience with technical literature can form the same habit and save much time. It is really wonderful how much repetition there is in engineering writing and in the production of engineering papers. Each new generation needs the same drill in its reading as did its predecessors, but it wants the old form in new dress, and each year a vast number of engineers have arrived at that degree of maturity that they will be interested to read matter that suddenly impresses them and which seeems to them new, but which in reality has already been largely well written long ago. It thus happens that we are under the necessity of seeing much the same facts and principles repeatedly published in varying form, for some one is always attracted to really read them, with consequent benefit to himself, under the belief that they are new and novel.

Again, the mature engineer notes that a large amount of engineering literature is of the purely descriptive order, merely giving outlines of work that has been accomplished without going into reasons or principles. All this kind of writing is valuable and useful and has its proper place, but all of this class of literature has its limitations. One of the most severe of its limitations is that it rarely describes mistakes, errors of judgment or failures, and in these lie the most valuable lessons to the seeker after truth. One is obliged to read between the lines or read with reservation, much as one does in reading accounts of battles in the daily press. It is always wise to look back and note the origin of the dispatches in such cases.

Much light is thrown upon engineering literature by personal or general acquaintance with the author. One can more fully appreciate what an author says when he knows fairly well what the author's experience has been. All men have their high, strong ground, their less trodden side slopes and their twilight zone of knowledge, and they should not be blindly accepted as authority in all of the fields in which they sometimes venture an opinion.

A tremendous lot of engineering literature is written which is of little permanent value. Often it represents the writer's struggles to understand a subject. Often it is compiled largely from a desire for publicity. Fortunately, the editors of the technical papers can limit this kind of reading by care in selection.

But amid all these drawbacks a discriminating mind will always find a great deal of wheat amid the chaff, and the wheat that will be gleaned will be of a differing kind and amount, depending upon the type of mind of the reader, his present problem and his desire to systematize his information. What, therefore shall he do with his special selection when once he thinks he has separated it from the flood of raw material?

Several courses are open to him:

First—He may rely on his memory and the published index to his bound volumes. It is safe to say, however, that few engineers really make much practical use of this method. The intervening index and the bother of a search following prove to be discouraging to that degree that a proposed reference search is abandoned in about one-half the suggested attempts. The ideal filing system is the one in which, with the least amount of effort, one can put his hand immediately and accurately on the thing itself, be it a book, a pamphlet or a data sheet.

Second—He may keep a special card index of important data and reference to valuable articles. This at once involves labor and attention which few busy men can give, and which, if done by assistants or librarians, largely loses its personal value to the one who needs it. The same objection as to the discouraging effect of intervening indexes holds good here, too, and it is further safe to say that, of all the contrivances for indexing, the most difficult to readily handle and rapidly examine is the card index system.

Third—He may abstract important data in a limited way on loose leaf transparent paper, standard letter size, and he may remove or detach articles of special value from out his journals, to be filed in the regular office filing system, like correspondence.

The writer has tried all of the above methods at considerable cost in time and patience, and has for many years settled upon the third method above outlined. With all its admitted limitations it seems to be the best for an office which is expected to find out information on a great variety of subjects in a limited time and with the least amount of effort.

Some description of its practical workings may be of interest here.

All the technical papers of the office pass on to the desk of the head of the office and are at least looked over (not read) by him. Articles important to his particular specialty are checked with pencil and articles of especial interest are looked over with care and double-checked. Once in a long while data important enough to go to the data file is noted. This is either especially abstracted by the stenographer, or if a diagram or cost data, perhaps traced in the drafting room—all on transparent paper for copying purposes. Special data of these kind on 8½-in. × 11-in. sheets are filed in the office data file (a separate but common standard correspondence file). From the data file loose leaf working notebooks are made up from blue prints for office or travel purposes. They are altered, refilled, amended and sorted back from time to time as needed to keep them of usable volume and usefully up to date.

The technical journals, with checked articles, go to the office clerk or the stenographer at odd hours, or the librarian if one can be afforded, and the useful articles are removed by tearing them out with a ruler. They are folded, usually once, to standard size, with one edge lap left for binding, and are then filed in a subject index file, like current correspondence. The Dewey Decimal system, especially arranged for the office, is used, but only as a general subject plan. When the file is full portions of its contents, especially that which is most useful, is simply bound in plain pasteboard covers and placed in the library shelves, with titles. Such a book (or many books) would contain all the recent articles thought to be of special value on a given single subject. The remaining portions of the technical paper are thrown away, but in a large office, warranting the expense, duplicate bound copies can be kept as well, with the general published index as their key.

The objections to this system are as follows: (1) It is too expensive for any but the most important offices doing specialized work. (2) Data accumulates almost too fast unless rigidly kept down to a minimum. (3) It requires some personal attention of the head of the office, a competent assistant, or the employment of a regular librarian.

The advantages are: (1) It compels the [illegible] what is being published in current engineering literature, if only by inspection. (2) It removes all intervening indexes between the searcher and the final repository in bound volume. (3) It keeps one's library usefully up to date on all lines in which one should be especially interested. (4) It is economical for final shelf room and binding cost.

Obviously, one should not start so elaborate a system as this unless he is fairly sure of the special line of engineering to which his life will be devoted. Otherwise, waste effort and discouragement will be certain. It is not to be recommended to the young man, but only to the mature man of early middle life when his work clearly indicates the necessity for it. It is, however, the prime requisite of the engineering specialist. To him some such system is invaluable. Of course, modifications can be made in it which will lessen its expense, a d if the amount of material which is filed is rigidly kept down to a minimum the resulting accumulations will not be embarrassing or so expensive.

Not a few consulting engineers use this standardized system interchangeably, particularly the data file, thereby greatly increasing its usefulness to each other as a joint effort.

We come finally to the mature and experienced engineer of advancing years. How can he make engineering and technical literature of use?

It is safe to say that when an engineer has much passed fifty or sixty years of age, and has led an active life, in constant touch with affairs, his need for engineering literature lessens. Out of the mass of detail which seemed to him so overwhelming and endless in his youth and early manhood fundamental principles emerge like peaks out of the clouds, and upon these as foundations all detail classifies itself simply and naturally, and therefore he feels less need for accumulated data or particular description. Probably no one enjoys engineering reading as does the mature engineer, for he can read between the lines and find much to instruct as well as interest, and yet while he is probably the most interested and intelligent reader of engineering literature that the journals have, his ambition as a collector is gone, and filing systems no longer appeal to him.

If his acquaintance is wide, he reads with interest the accomplishments of his friends, and the addresses of engineering society presidents, and articles on the ethics of the profession. Of failures he is the keen student. The personal column appeals to him, and if he is of right-mindedness he is conscious of more pleasure than formerly in the accomplishments of those who have succeeded and succeeded well in dire and burdensome responsibility. More often than the young man, he will turn back for his satisfaction to papers that served him well in times past, and perhaps smile at the lack of improvement that later attempts to deal with their subject often show.

Like aged men who relapse into second childhood, the engineering journal again becomes for him a technical newspaper of great personal interest and deep satisfaction, for no longer is he keen for jobs, or eager for data, but the human, personal, and ethical side of the life work of the engineer are uppermost in his mind, and he realizes that though he may have seemed to others, and even may have seemed to himself to have been striving all these years for emolument, as a matter of fact, the deep and abiding motive of his life work has been the pleasure of being *"needed"* and the joy of being useful.

In conclusion, I would remark that technical papers, along with the technical societies and their proceedings, form the repository of the professions; they are the interchange of experience, the common store upon which we all draw. Without them we would be strangely helpless. We are indebted to every one more or less who records his experience for the common use, and that debt we should endeavor to helpfully repay in kind, but wisely, concisely, and thoughtfully.

Concrete Block Molding for the Panama Canal Buildings.

All permanent canal buildings at Panama are being built of concrete blocks molded at field plants and at a central plant at Corozal. Referring to the block molding work the "Canal Record" announces that several improvements have been made in the methods of operating at the plant, which have made it possible to reduce the force of laborers by about one-half. One is the substitution of a power driven tumbler or rattler for hand labor in removing particles of concrete from the cast iron palettes, which, inserted at the bottoms of the molds, form the bases on which the blocks are handled until they harden, when the palettes are loosened by a hammer. Two laborers operate the tumbler and do work for which 14 men were formerly required.

Another device which has given general satisfaction is a washer for cleaning screenings. The screenings are shoveled from the cars into an inclined trough, about 10 ft. long, at both ends of which are powerful jets which throw the screenings into motion. As they slide down the flume they fall on inclined screens, the particles of rock passing through into a pile, while the finer particles of earth are carried away by the water, through a discharge flume. In addition to cleansing the screenings mechanically, and allowing a reduction of force of 12 laborers, the washer has done away with dust, which was formerly very objectionable. Moreover, as the finer particles are carried away and settle, a quantity of sand can be skimmed off the top. This is used to a great extent in place of Chame sand and effects considerable saving in the cost of material.

Estimated Cost of Short Transmission Lines for Electric Power for Tunneling.

The following estimated costs for 200 hp. of 1-mile, 5-mile and 25-mile transmission lines for electric operation of tunneling plant were prepared by the General Electric Co. for Messrs. D. W. Brunton and J. A. Davis, and are given in Bureau of Mines Bulletin 57 on "Mine Tunneling:"

Cost of Installation for Erection-Transmission Line for Different Voltages and Distances.

(1) 200 hp., 1 mile, 440-volt, direct current:	
Poles, cross-arms, insulators and fittings (poles spaced 100 ft.)	$ 375
33,000 lbs. copper cable, 500,000 circular mils (4 conductors required), at 18¾ cts. per lb.	6,025
Cost of erection	300
Total	$ 6,700
(2) 200 hp., 1 mile, 440-volt, 3-phase, 60-cycle, alternating current:	
Poles, cross-arms, insulators and fittings (poles spaced 100 ft.)	415
34,000 lbs. copper cable, 350,000 circular mils (6 conductors required), at 17¾ cts. per lb.	6,035
Cost of erection	375
Total	$ 6,825
(3) 200 hp., 1 mile, 1,100-volt, 3-phase, 60-cycle, alternating current:	
Poles, cross-arms, insulators and fittings (poles spaced 125 ft.)	385
5,100 lbs. copper cable, B. & S. No. 0 (3 conductors required), at 17¾ cts. per lb.	905
Cost of erection	265
6 transformers, 1,100 to 440 volts, with switches, etc., erected	2,900
Total	$ 4,455
(4) 200 hp., 5 miles, 1,100-volt, 3-phase, 60-cycle, alternating current:	
Poles, cross-arms, insulators and fittings (poles spread 125 ft.)	1,870
122,000 lbs. copper wire, B. & S. No. 000 (9 conductors required), at 17¾ cts. per lb.	21,650
Cost of erection	1,580
6 transformers, 1,100 to 440 volts, with switches, etc., erected	2,900
Total	$28,000
(5) 200 hp., 5 miles, 6,600-volt, 3-phase, 60-cycle, alternating current:	
Poles, cross arms, insulators and fittings (poles spaced 125 ft.)	1,870
6,500 lbs. copper wire, B. & S. No. 6 (3 conductors required), at 17¾ cts. per lb.	1,150
Cost of erection	1,080
6 transformers, 6,600 to 440 volts, with switches, etc., erected	3,700
Total	$ 7,800
(6) 200 hp., 25 miles, 6,600-volt, 3-phase, 60-cycle, alternating current:	
Poles, cross-arms, insulators and fittings (poles spaced 125 ft.)	$ 9,350
103,000 lbs. copper wire, B. & S. No. 1 (3 conductors required), at 17¾ cts. per lb.	18,300
Cost of erection	5,150
6 transformers, 6,600 to 440 volts, with switches, etc., erected	3,700
Total	$36,500
(7) 200 hp., 25 miles, 22,000-volt, 3-phase, 60-cycle, alternating current:	
Poles, cross arms, insulators and fittings (poles spaced 125 ft.)	9,900
33,000 lbs. copper wire, B. & S. No. 6 (3 conductors required), at 17¾ cts. per lb.	5,860
Cost of erection	5,190
6 transformers, 22,000 to 440 volts, with switches, etc., erected	5,200
Total	$26,150

Method and Cost of Constructing a Reinforced Concrete Bulkhead at Hibernia Mine, New Jersey.

To permit flooding of old workings and shut off the water from the new workings at the Hibernia magnitite mine in New Jersey a reinforced concrete bulkhead of the design shown by Fig. 1 was constructed. The design and construction are described by S. L. Wise and Walter Strache in August Transactions American Institute of Mining Engineers as follows:

As the bulkhead was required to withstand a pressure of 200 lbs. per square inch, it presented some difficulties. A careful consideration of the various types of mine dams now in use led to the adoption of a design of the form of a truncated wedge. In this, the pressure side of the dam is of greater area than the back, so that the resultant action is similar to driving the wedge. By cutting generous skewbacks in the walls, roof, and floor, this type in reality becomes an invisible arch. The wedge feature tends to compress the materials in the bulkhead, thereby adding to its imperviousness. Concrete was chosen as the material. In order to lessen the labor and simplify the construction of the forms, straight forms were placed on both the front and back of the dam; the arch in this bulkhead is therefore invisible. To waterproof the bulkhead the entire pressure side was joined with 3 ins. of waterproof cement. Concrete was 1:2:4, using washed and screened gneiss.

In drilling the recesses for this bulkhead care was taken so to point the holes that the excavation would coincide in form and dimensions to the design. At a distance of 25 ft. from the old dam, holes 36 to 39 ins. in length and spaced 1 ft. apart were drilled in the sides, roof, and floor, at right angles to the course of the drift. Stoping and hand drills were used in this work and four men constituted the gang in this as well as the subsequent drilling with column drills. Thirty-five feet from the old bulkhead a series of holes 4 ft. in length and from 1 to 1½ ft. apart were placed slanting to conform approximately with the inclination of the skewbacks. These holes were only burdened with about 1 ft. of ground. Under ordinary conditions longer holes would have been drilled, but the proximity of operating pumps made extraordinary precautions necessary for their protection during the shooting, and so heavy blasting was not attempted. A round of six holes was shot at a time. A third series of holes was drilled slanting to conform with the deeper portions of the recesses. When blasted, this series broke evenly at the line of the 3-ft. holes first drilled and the resultant recess conformed almost exactly to the figure determined upon, and the total excavation agreed with the original estimate of 60 cu. yds. The muck was economically disposed of in a nearby chute.

The materials required for the work were unloaded and stored close to the mouth of the shaft. Due to the lack of space in which to store the materials on the 16th level, the matter of lowering and delivering the required materials without interrupting the work was one of the most troublesome obstacles encountered. Eight to twelve men were employed on the surface in sacking sand and stone while the excavation was in progress on the 16th level. The empty bags produced as the cement was used augmented the 200 old cement bags purchased for the sacking. While enough sand could be stored on the 16th level for this entire bulkhead, there remained insufficient room for the storage of the daily requirements of stone and cement. It was found advantageous, therefore, to employ a small night crew, who lowered much of the material required for the next day's work.

The ten curved rails were bent to a 14-ft. radius over a form in half a day. This was done on the surface.

The forms on the 16th level bulkhead were built of 2-in. undressed lumber and 6 to 10-in. round posts were used for studding and braces. The forms were thoroughly braced and were wired to stiffen them further. The interior faces of the forms were covered with tar paper, and the junction of the forms with the rock was plastered with a 1:1 cement mortar on all sides. The pressure-side forms were carried to the roof of the level at once, but did not extend into the recess.

The recess was thoroughly cleaned of loose rock and washed down, and all the reinforcing material, pipes, and the manway were placed in position before the concreting was started. Furthermore, the floor and sides of the recess were plastered with a 1:1 cement mortar before placing the concrete.

The concrete was made of 1 part cement, 2 parts sand and 4 parts stone, these proportions being determined by actual measurement. A batch of concrete contained ⅔ cu. yd. The sand was first placed on the mixing platform and the heaps flattened down. On this was emptied the cement, and these two materials were thoroughly mixed and flattened out before receiving the stone. This mixing took place about 12 ft. from the front form of the bulkhead. Enough water was used to make a wet mixture. Two men did the first mixing and turned the mass, then passed it on to the next two, who again turned it, passing the finished concrete to the last two men at the mixing board. These men shoveled directly into the form. In this manner, while each two men received a short rest of a few minutes between batches, fresh material was being placed on the starting end of the mixing platform while the men nearest the form were still disposing of the concrete mixture. This also insured a thorough mixing. One man remained in the form to level off each batch. The best day's work consisted in the placing of 12 cu. yds. of concrete.

The waterproofing compound, "Impervite," was carried up as a 3-in. facing, its level being kept the same as that of the concrete. An even thickness of the waterproof layer was maintained by the use of three forms of 3/16-in. plate, 6 ft. long by 6 ins. wide, fitted at the upper corner with 3-in. spreading bolts. These forms, placed across the entire width of the face, were raised 3 to 4 ins. at a time, and enough concrete was then shoveled against them to keep them in place. The almost semi-liquid waterproofing compound was mixed on the level and was carried to the forms in buckets.

Before leaving at night, sharp man-size (about 100-lb.) stones were set at least 6 ins. apart in the concrete mass. This made a strong bond, and before concreting the next day this rough surface was freshly plastered with a thin 1:1 mortar. As the roof was reached, false forms were placed, and the work was finally finished in tightly bonded dovetailed blocks. Throughout the work, the leakage from the old dam, 6 gals. per minute, passed through the 2-in. drain pipe of the bulkhead.

Seven 2-in. grout pipes, four on the pressure side and three on the opposite side, were placed in the concrete as the work neared completion. They were all located near the roof and were directed to such places as were most difficult to fill with concrete. As the work had to be hurried, but a day and a half elapsed after completion of the cement work before grouting was begun. The grout mixture was a mortar consisting of 1½ parts of sand to 1 part of cement made fluid with water-dissolved "Impervite." A mine-made grout "gun" was used, and the grout was forced successively into the several pipes by means of air under the pressure of 85 lbs. per square inch. As the grout was forced through the different pipes the ejection of some of this material through the other pipes indicated that the greater voids were filled. As the "gun" connections were changed those pipes giving the greatest discharge were plugged, and the discharge was finally limited to one pipe. This, too, was filled and plugged. The first day's grouting was allowed to set over night, and the following day all the pipes were again tested. This time there was no communication between the pipes, and as little or no grout could be forced into any one of the pipes, the grouting was considered most satisfactory.

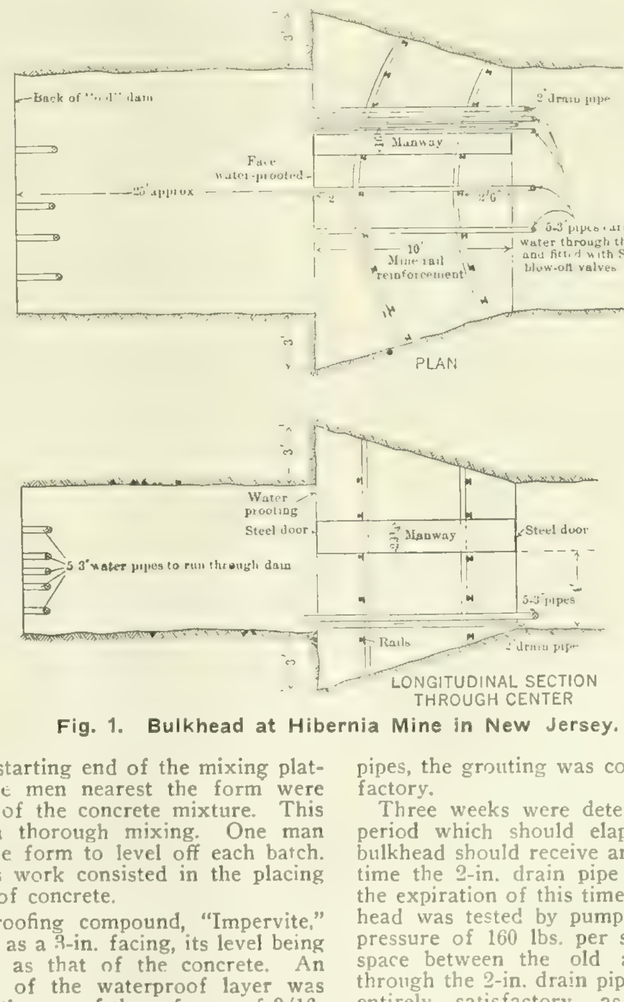


Fig. 1. Bulkhead at Hibernia Mine in New Jersey.

Three weeks were determined upon as the period which should elapse before the new bulkhead should receive any load. During this time the 2-in. drain pipe was left open. At the expiration of this time the completed bulkhead was tested by pumping water up to the pressure of 160 lbs. per square inch into the space between the old and new bulkheads through the 2-in. drain pipe. The results were entirely satisfactory, as the total seepage amounted to only ½ gal. per minute at first. This small leakage subsequently stopped almost completely.

A cheap class of labor was employed exclusively, the men receiving $2 per 10-hour shift. Following are tables showing the cost of the work. The interference caused by the necessity of keeping two large pumps in operation within 50 ft. of the bulkhead was perhaps the greatest cause for the apparent high cost. The labor cost of lowering materials was also very high for the amounts handled, which had to be lowered 1,350 ft. in one skip.

Division.	Costs per cu. yd.
Labor	$13.17
Superintendence	2.17
Transportation	0.84
Materials	8.38
Total	$24.56

WATER WORKS

Data on Consumption and Cost of Water for Uses Difficult to Control if the Supply is Unmetered.

Many minor uses of water, not common in all households, help to swell the total figures on water consumption in American cities with unmetered supplies. Some of these uses in themselves reach surprisingly large proportions. In this article are given numerous data on the amount of water used for some of these purposes which we call minor since they are seldom or never enumerated among the municipal uses of water. These data are based on careful observation and actual measurement and constitute a powerful argument for the metering of all water supplies. The information given is taken from a paper before the recent annual meeting of the New England Water Works Association by Mr. William F. Sullivan, engineer and superintendent of the Pennichuck Water Works of Nashua, New Hampshire. Numerous tables and photographs accompanying the original paper will be published in the Journal of the Association. The figures for discharge given at various points in the article, presumably are for the pressure usually carried at the Nashua water works.

When the water rate for building purposes is a flat one, usually nominal, based on the size or kind of building, there is a waste due to the carelessness, sometimes wilful, of the workmen. Much water drawn for drinking purposes is allowed to run to waste. The workmen take a drink from the butt end of a hose or faucet, usually about a pint, and the end of the hose is then dropped to the ground and left running. The discharge from the butt end of a hose with sillcock open half way is 9 gals. per minute. The water may be allowed to run to cool before taking a drink and thus 7 to 16 gals. per minute are wasted. Observations show that from 10 to 4,000 times as much water is wasted as drunk during the summer months on building construction.

For slacking lime and the tempering of mortar, the mixture of cement and concrete on small jobs the mason's helper or attendant, as a rule, does not shut off the water when he has enough for the needs at hand, but drops the butt end of the hose on the ground or sticks it into a cask already full of water and the overflow runs over the edge in a miniature waterfall. Quite often at quitting time the attendant goes home neglecting to shut off the water for the night.

If the rate is a fixed sum per cask for lime or cement averaging about 6 cts., the above mentioned wastes are as frequent. Those who pay 6 cts. per cask realize that the rate is excessive. Those who do the work know [illegible] per barrel of lime. This quantity of water at a meter rate of 20 cts. per thousand would cost 2½ cts. per barrel.

Equally so it is difficult to control and to account for water used if the rate is per thousand of brick laid, perch of stone laid or yard of plaster spread. Efficient builders know this and are willing to have the water they use for building purposes metered. When the meters are not furnished by the water works, the contractor has a tested meter for such purposes. These men control a useless waste of water and at the same time save money.

Water for building purposes is sometimes taken from adjoining premises before the owner of the new building has made application for water or before the water works has laid the new service. In cases where the neighboring premises are metered, there should be no charge on the part of the water works for the water used. The settlement should be between the owners of the premises. In adjusting the matter and making a proper charge, the owners of adjoining premises often have difficulty in controlling their tempers. The old resident believes the new resident at the beginning has overdone the neighborly act. It requires at times the services of an arbitrator or a police court judge to settle the matter.

When the adjoining premises has an unmetered supply, it sometimes requires on the part of the water works officials explanations to show that they are entitled to compensation for the water used. The man with the service believes that he has a right to furnish water to his next door neighbor, and both at times reason that the water is paid for once and that is sufficient. Both lose in their contention, as the charge is usually collectible to the owner of the new building.

City departments often assume the right to puddle trenches, flush streets and sewers at will. Flushing sewers is often done by plumbers and drain layers. A drain layer borrows several lengths of 2½ in. hose from the fire department, attaches it to a convenient hydrant, places the nozzle into an opening in the drain or sewer and lets the water run. These men seldom know or little care that hydrants are set primarily for fire protection and are not designed to be opened by unskilled hands at random. Such use of a hydrant in one day may be more severe on the working parts of a hydrant than 20 years of legitimate use. Fire stream tables show that with 150 ft. of average 2½ in. hose with 1⅛ in. nozzle and a hydrant pressure while stream is flowing of 50 lbs. per square inch, the discharge is 192 gals. per minute. Sometimes it takes hours and sometimes days to flush a sewer. Assuming an average of two hours, the use would be about 23,000 gals., for which at 20 cts. per thousand the charge should be $4.60 plus something for the wear and tear. If charges were made for this kind of service, the persons doing the work would think it excessive.

In the same line of work as flushing sewers is flushing streets. The city water is called upon to sweep the streets and gutters and then follow the sweepings into the catch basins and often clean out the blocked sewers from these sweepings by flushing. This work should be controlled by a suitable charge and by the placing of suitable flush hydrants to be used in place of fire hydrants. The water used through 150 ft. of 2½ in. flush hose is about 11,000 gals. per hour, which at a 10 ct. rate would cost $1.10 per hour per stream.

Where a water taker keeps a cow or cows and has an unmetered service, it is often convenient to have running water in the troughs for the cattle and to cool the milk. A faucet partially open runs about 10 gals. per minute, and running an hour in the morning and an hour at night uses at the 20 ct. rate 24 cts. worth of water per day.

A worthy special privilege is to allow the baby's night milk bottles to be kept cool and sweet from a running stream. The bottles are usually placed in a receptacle and the water left running over them. Tests on the flow from faucets show that this cooling process will use water to the value of from 10 to 60 cts. per night. Other bottled products, however, are kept cool by the use of a trickling stream, and the cooling often costs the water works more than the value of the contents.

Another instance of waste is where the family goes away on a vacation and the man of the house stays at home. It frequently happens that the water is permitted to run during the whole vacation period. This waste amounts to over 4,000 gals. in eight hours.

Many experiments have been made on the flow from orifices and nozzles and it was thought it might interest some if the ordinary commercial sink faucets or plain bibbs were calibrated as it were, and a few practical experiments made on discharges. Ten so-called ¾ in. sink faucets were selected. They were found to be approximately alike, but different in smoothness of bore and in the number of turns to open. The tests were made on the fractional parts of number of turns to open, that is—a ¾ in. plain bibb with an opening of 1/16 of a turn flowed 2.1 gals. On full opening with 2⅛ turns open the flow was 24 gals. per minute. With a ½ in. plain bibb and an opening of 1/16 of a turn the flow was 0.7 gals. and on 2¼ turns open the flow was 18 gals. per minute. The usual faucet opening that is commonly practiced by people who let the water run for these uncontrolled uses is from ⅛ to ¼ turn. This waste is about 6.5 gals. per minute or 9,360 gals. in 24 hours. At 10 cts. per thousand this equals 93 cts. per day.

Sometimes fishermen who are thrifty enough to go out to the streams and gather thousands of shiners for bait put shiner tanks into their cellars, shed, barn, or under the piazza. They do it in a manner to keep water works officials from knowing about it, for they know that if they pay for the water to keep the shiners alive the industry is not a paying proposition. Knowing that a meter would place a tariff on the business which would kill it, they once in a while by-pass the meter. We were able to put a meter on a shiner tank with a so-called by-pass unknown to the owner and found that he was using water to the value of $1.40 per day.

Sometimes the sewing machine is run by a water motor and the water works furnishes the power. The kitchen faucet and the degree of opening are the only guides as to what water is used. On installations of this kind the only proper governor is a meter.

Nowadays the weekly washing for the family is done with a motorized washtub. A water taker who paid a minimum charge of $6 per year objected to having a meter placed on a service which supplied the power to run a washing machine. This particular taker not only did the family washings but took in washings. She was told that the water works was willing to furnish the water for the washing, but was not willing to do the washing. These machines are capable of using upwards of 1,000 gals. per hour.

The majority of hand hose privileges are not abused. The persons paying for this privilege are reasonable. But there is a minority of takers in all unmetered communities who believe that in order to get value for what they pay on flat rates they must use the hose extravagantly.

If the rule says that hand hose may be used on the premises between fixed hours of the day, the regulation is somewhat easier to enforce, especially where the police cooperate. But where the rule permits the use of hose a certain number of hours without specifying the exact hours, it is difficult in all cases to obtain close control. The right to use hose regardless of time limits is a common belief and some people knowingly take advantage of the inability of water department officials to have a sufficient number of inspectors to keep a daily record of the hours of use. At night advantage is taken of the darkness and lack of supervision to use water on lawns and gardens indiscriminately. This abuse seems to be more prevalent with privately owned works than with municipal plants.

It has often been observed, when the temperature suddenly drops to freezing during the night, that the morning sun may be seen glistening on ice covered lawns, shrubs and plants. It is a common sight to see a fixed hose, nozzle or spray running during a rain storm of many hours duration.

The hose rate based on foot frontage is

equally difficult to govern. In addition it requires the checking up of frontages while no regulation is provided for time. If hand hose only is used, it is reasonable to suppose that this method is not so susceptible to abuse. It is a fact that some people are content to wet their lawns sufficiently to keep them in good condition while others soak them and make them soggy.

In this era of back to the soil, when hundreds of families are practicing intensive farming and chicken raising, these pursuits require much water, and some of us realize that these minor uses of water are considerable and difficult to control.

In the rural districts supplied with water and paying but small interest returns on the cost of construction and a low minimum, means are sometimes taken to circumvent the water works by people who have large grounds or farms. The houses on these places often have the open drain pipe from the kitchen sink conveying water from the outlet through an open ditch, or arrange wooden troughs to carry the sink waste to a head ditch having laterals. During the dry period the sink faucet is kept open and the water allowed to run for long periods. Thus the water works furnishes a first class irrigation system at low cost. One sink faucet can furnish by this means without aid of hose or pipes 25,000 gals. per day worth $5.

Water takers will often tie the hose nozzle to the back of a chair, or fasten it through the handle of a garden fork, or make a hitch to a wheelbarrow, or squeeze it into the fork of a tree, and sometimes the hose is coiled up into helical shape, the nozzle turned under and the stream or spray pointed in any direction desired.

If the consumers' attention is called to these excessive uses, they will argue that the rule which reads a certain number of hours per day for hand hose means and permits a fixed spray or jet. "What difference is there between a fixed jet and a hand hose?" "Does the fixed jet, which is the same as the hand hose, use any more water?" Keeping up a rapid fire of questions, they will ask, "Do you expect me to stand or sit and hold a hand hose four hours per day, or do you want me to hire a boy for that work?" There have been cases where, rather than place a meter on the supply, they pay for the services of a boy or man to sprinkle the lawn and thereby get the full benefit of the allotted hours. The reply to the foregoing questions is usually that because of the abuses of the time limit when fixed nozzles are used, it requires stricter regulations.

In states where public service commissions have been established, privately owned utilities, unlike municipal plants, are not allowed to grant special privileges to cover individual cases. Private companies must not show any discrimination in rates or service. To carry out the orders of a public service commission and control the misuse of water by some, the water department is often obliged to meter fixed sprays and jets and control by this means, the fairest and most equitable way, the use of water.

Instances are many where people permit hose streams to run night and day, week in and week out. In one case, a user paying $4 a year for a hand hose privilege with the right to use it four hours a day, fastened the nozzle to a block of wood and let the water run for weeks. The water department kept a record of the time and when the owner's attention was called to the misuse she emphatically declared we were mistaken, notwithstanding the fact that her neighbors complained that they were annoyed by the water running over the concrete walk and that the sizzling noise of the stream disturbed their sleep. This particular misuse for 24 days wasted an estimated quantity of 691,200 gals.

Experiments were made on the discharge of garden hose to determine how much water a hose privilege of this kind used. Through 50 ft. of good quality ¾ in. rubber hose, at a pressure of 55 lbs., and using a Boston nozzle, so-called, which has a spray attachment and a discharge orifice of 0.22 ins. in diameter, the jet stream discharged 3½ gals. per minute, while the spray attachment with a pressure of 45 lbs. discharged 7 gals. per minute. The same nozzle with 25 lbs. pressure discharged 2.5 gals. for the jet and 5 gals. for the spray.

The Fairy type of nozzle with an opening of 0.18 in. reversed the amount discharged by the jet and the spray. The jet discharged 7.5 gals. and the spray 2.5 gals. at 48 and 60 lbs. pressure, respectively. With a pressure of 25 lbs. the jet discharged 5.5 gals. and the spray 2 gals. per minute.

The straight tip type of nozzle discharged 7.5 gals. per minute at 47 lbs. pressure.

The Boston type of garden hose nozzle with spray attachment discharges about 300 gals. per hour or 7,200 gals. per day, which at a 20 ct. rate per thousand gallons, if metered, would amount to $1.44 per day.

The Fairy type spray nozzle with jet attachment uses 240 gals. per hour or 5,760 gals. per day, or $1.15 worth of water at meter rates.

The ordinary straight tip hose nozzle uses water to the value of $2.16 in 24 hours. The rate of 20 cts. per 1,000 gals. is taken because it approximates the average cost of producing 1,000 gals. of water in the United States when all the factors are figured.

Another abuse of the garden hose is by people who want to defraud the water works by removing the nozzle and letting the open butt run on to a brown or parched spot on the lawn. An open butt of a ¾-in. hose, 50 ft. in length with a sillcock pressure of 55 lbs. will discharge 16.5 gals. per minute or 23,760 gals. per day and should give a return to the water works of $4.75 per day.

In one instance the discharge from such a butt was allowed to run for 96 hours on a brown patch with an area of 1 sq. yd. The original cost of this land was 10 cts. per square foot or 90 cts. per square yard. The lawn cost about 6 cts. per square foot for loaming, and grassing and the total cost per square yard of lawn was $1.44, while the cost of the water used for four days should be $19 for the square yard, or $2.11 per square foot.

Open butts are also surreptitiously allowed to run under hedges and shrubbery. Long and short lengths of perforated pipes are concealed in and under the hedges and a hose or pipe connected. One of these perforated pipes uses about 15 gals. per minute or in a day water to the value of $4.32, which is more than is received for a year's hose privilege.

The use of lawn and garden sprays, portable fountains, etc., has been given a great impetus in recent years in unmetered places. This increased use has come about principally by the low cost of sprinklers. The price of sprinklers range from 25 cts. to $5 and some that are specially designed cost more. Tests of the quantity of water used by sprinklers show that some of these low priced tin factory sprinklers use as much water and are as effective for the purposes designed as the more expensive ones.

In a city where the water company had an abundant supply and had generally permitted the water takers for years to make the city more beautiful by furnishing water to maintain the ever-green appearance of lawns and hedges, and this with little regulation or enforcement of the rule regarding sprays, there came a time when the abuse became excessive. Conservation of resources coupled with the present day sentiment that special privilege and discrimination should cease, brought about a change in policy. This company like others realized that the granting of special privileges for what might be termed a civic betterment or for reasons of expediency, were illogical, and adopted the following rule:

"Premises using water for ornamental fountains, portable fountains, garden or cooling sprays or any automatic whirling or fixed jet or nozzle on garden hose shall be metered." After the publication and approval of this rule by the public service commission, inspections were made and those breaking the rule were notified. The task was to enforce this rule with a minimum of friction and ill will on the part of the water takers. A campaign of education and explanation was carried on. The company fortified itself with many photographs showing the uses and misuses of sprays. There were many humorous phases of the picture taking, persons not knowing the real object of the photographer posed beside the broken hose or running streams.

When it became known what the object of the picture taking was, many takers held the hose instead of using the sprinkler. In some cases water takers had to be confronted with the photographic proof and sprinkler data to be convinced that they were getting something they were not paying for.

Fifteen sprinkling devices, such as are ordinarily for sale at hardware stores, were obtained and the discharges determined. These tests were set so that the experiments would be conducted as nearly like the ordinary use as possible. The table of discharges from sprinklers showed the flow per minute, per hour and per day by these devices, also the value of the water discharged for these periods of time.

If time permitted, there could be shown the waste and uses of leaky fixtures and the deliberately continuous use of the water by both self-closing and non-self-closing water closets for flushing and to prevent freezing. There are many and varied small uses difficult to control.

The remedy for these abuses has been applied in those places which have adopted either universal metering or the metering of takers who abuse the service privilege.

A further remedy which has been suggested to assist water works managers to put their plants on a businesslike and practical basis would be the placing of municipal water works under the same supervision by state public commissions as other public service corporations, for, after all is said, the municipal corporation has the same need of supervision as the private corporation. In the thickly settled portions of the country the end must come to continued extensions of water collecting areas and a broad and far seeing control of wastes will do much to settle this most serious problem with which we are confronted.

Water Department Methods which Limit Per Capita Consumption to 39 Gals. Daily at Milton, Mass.

Milton, Massachusetts, is a town with a population of 8,470, lying between Quincy and Boston, and bordering on Canton and Hyde Park. It is supplied with water by the Metropolitan System with two distinct services. The first, designated as the southern high service, accommodates about 83 per cent of the consumers with a daily consumption of 285,000 gals., and has a pressure range of 30 to 110 lbs. The remaining 17 per cent is supplied by the southern extra high service, with a daily consumption of 39,000 gals. and a pressure range of 50 to 133 lbs.

On hearing these figures the natural question arises: "How does Milton maintain its low consumption of water, using, as it does, only 39 gals. of water per day per capita?" The foregoing question was answered fully by Mr. David A. Heffernan, Superintendent of Water Works at Milton, in his paper before the annual convention of the New England Water Works Association which we here reprint as follows:

To my mind there are three principal reasons for the low per capita water consumption at Milton which I will endeavor to explain from a practical viewpoint. They are: 1. Universal meter system. 2. Our method of construction. 3. Rigorous control of hydrants.

Nothing else contributes quite so much to low consumption as a system universally metered. In Milton this is carried out to the highest degree. Not only are the private services metered, but in every municipal building, every standpipe, and every fountain, the water is being measured. Only hydrants escape this minute inspection.

No meter remains on a service for more than five consecutive years. At the end of that time, perhaps before, it is removed, cleaned, repaired, and tested in our own shop.

Meters are read twice every quarter by the inspectors, who carry aquaphones and are always on the alert for foreign noises. If the reading is larger than the average for that house, or a sound is heard on the service pipe, which might possibly be a leak, a report is made to the office on the inspector's return. Then a department plumber is sent to inspect the premises. Should he find the leak to be on the pressure side of the meter, it is repaired immediately by the department at the expense of the owner. If the leak is discovered on the house side of the meter, the owner is notified and his own plumber makes the necessary repairs. In this manner the water department and the consumer unite in reducing waste water to a minimum.

Another important reason is that the department does all its own work, covering main construction, services and repairs.

Our system consists of 49 miles of cast iron pipe, ranging in size from 4 ins. to 16 ins., the system being controlled by 576 stop-gates; 364 hydrants take care of the fire needs, while watering carts may be supplied from 58 standpipes. A total of 1,678 services furnish the inhabitants with water. Our own employes lay the service pipe at a stated cost per running foot, and it extends to the inside of the cellar wall where the meter is set. The responsibility of the department ceases at the meter, which is supplied up to ¾ in. without cost to the consumer. Should the applicant for water desire to lay the service pipe himself, or otherwise than by the town, the department will make the tap and lay the pipe to a point just inside the property line, build a manhole, set the meter, and let him complete the service. However, this choice is very rarely taken advantage of for the reason that if a leak should occur between the meter and the house, the department would have nothing to do with its repairing. And it has been found that contractors do not use the care in laying the pipe that the department does. Thus, the town, having complete jurisdiction over all the work it does, and over no other, and using only the best materials in this work, is in a better position to prevent unnecessary waste through these channels.

All construction, service and main work is tested by water pressure before back-filling. On all 2-in. work and over, a testing plug, tapped to hold a shut-off, is inserted in the bell end of the pipe. By means of this tap in the plug the trench may be puddled after it is seen that all the joints are tight.

Hydrants used in the town are post hydrants of one type and are uniform throughout, having a 5-in. gate opening and a 7-in. barrel. Gates on all hydrant branches save shutting down an entire section when repairs on one hydrant are needed. Only firemen are allowed to use these hydrants and then only in case of fire. If contractors need water in a place where no means of supply, other than a hydrant is available, the department will send a man to furnish them with water, the contractor paying for the water used and the labor incurred in supplying it. Besides an annual inspection, hydrants are carefully examined after every fire to make certain that there is no leakage.

I have tried to explain as simply and concisely as possible what makes our consumption so low. Many will say, or at least think, that it is because Milton does not contain many factories. This argument, however, migh easily be refuted with figures.

The night consumption also tends to bear out my statement. Here, too, the consumption is the lowest of any in the district supplied by Metropolitan water. An average of only 10 gals. per capita is measured daily between the hours of 1 and 4 a. m. Compare this with the average of 56 gals. for the system and it is seen that our pipe lines are in good condition.

The total amount of water measured by the Metropolitan meters in Milton for the year 1913 was 118,000,000 gals. The registration by house meters was 96,000,000 gals., showing a difference of 22,000,000 gals., or 18 per cent. This is accounted for by flushing dead ends, water used at fires, and the underregistry of house meters. We are fast connecting up our dead ends, and, taking everything into consideration, Milton seems in a fair way to lower its already low consumption, of which its water department is so justly proud.

Cost of Pumping Stations, Pumping Machinery and Distributing Reservoirs in Small Water Works of Massachusetts.

In our issue of Aug. 26, 1914, we published an article on the designing of small water works systems. The article was from a paper by William S. Johnson, Hydraulic and Sanitary Engineer, Boston, Mass., before the New England Water Works Association. The present article gives some cost data pertaining to pumping stations, pumping machinery and distributing reservoirs, and, like the article mentioned, is drawn from Mr. Johnson's paper. These and other data are published in the Journal of the Association for June, 1914.

Perhaps one of the most serious problems which confronts those interested in the con-

TABLE I.—COST OF PUMPING STATIONS IN RECENTLY CONSTRUCTED SMALL WATER WORKS IN MASSACHUSETTS.

Town.	Population (1910).	Pumping station— Material.	Size.	Cost.	Cost per sq. ft.
Ashland	1,682	Cobbles	22x30	$1,935	$2.93
Bedford	1,231	Brick	19x36	1,857	2.71
Deerfield	.....	Cobbles	9x12	350	3.24
Dracut	3,461	Wood and steel shingles over entire surface	.....	1,730	....
Dudley	4,267	Brick	24x26	1,948	3.12
East Brookfield	.....	Brick	24x30	1,628	2.26
East Douglass	2,152	Brick	20x35*	2,500	3.57
Leicester (Cherry Valley and Rochdale)	.....	Brick	30x40	2,368	1.97
Littleton	1,229	Brick	24x34	3,100†	3.80†
Marion	1,460	Brick	.....	.....	....
North Chelmsford	5,010	Brick	33x23	2,000	2.64
Oxford	3,361	Brick	24x24	2,000	3.46
Pepperell	2,953	Brick	28x28	2,852	2.68
So. Hadley (Fire Dist. No. 2)	.....	Brick	25x36	2,700	3.00
Wareham	.....	Brick	25x36	2,133‡	2.37‡
West Groton	.....	Brick	16x16	500‡	1.95‡
Wrentham	1,743	Brick	25x36	1,647‡	1.83‡

*Two stories. †Includes some grading. ‡Without pumping machinery foundations.

TABLE II.—COST OF PUMPING MACHINERY IN RECENTLY CONSTRUCTED SMALL WATER WORKS OF MASSACHUSETTS.

Town.	Pumping machinery— Pumps.	Engines.	Cost.	Cost per horsepower.
Ashland	2-7 x8	2-18 hp. oil	$4,358	$121.00
Bedford	2-8½x10	2-25 hp. gasoline	4,000	80.00
Deerfield	1-4 x6	1-7½ hp. motor	475	63.30
Dracut	1-8 x10	1-20 hp. gasoline	1,783	89.15
Dudley	2-8 x10	2-25 hp. motors	2,500	50.00
East Brookfield	2-5½x8	2- 8 hp. oil	3,100	193.75
East Douglass	1-10 x10 1-7¼x10	1-35 hp. motor 1-15 hp. motor	2,455	49.10
Leicester (Cherry Valley and Rochdale)	2-8 x10	2-18 hp. oil	4,414	122.50
Littleton	1-7½x10	1-25 hp. oil	3,960	158.50
Marion	2-7½x8*	2-40 hp. oil	8,157	102.00
North Chelmsford	2-7¼x10	2-25 hp. motors	3,500	70,00
Pepperell	2-8 x10	2-25 hp. oil	6,200	124.00
South Hadley (Fire Dist. No. 2)	2-8 x10	2-35 hp. oil	6,875	98.25
Wareham	2-8 x10	2-25 hp oil	5,642	112.80
West Groton	1-6 x8	1-10 hp. gasoline	1,163	116.30
Wrentham	1-8 x10	1-25 hp. oil	2,821	112.80
Wrentham State School	2-6 x8	2-10 hp motors	1,784	89.20

*Double acting.

Note.—All pumps are vertical, single-acting, triplex pumps, unless otherwise noted.

TABLE III.—COST OF DISTRIBUTING RESERVOIRS IN RECENTLY CONSTRUCTED SMALL WATER WORKS IN MASSACHUSETTS.

Town.	Distributing Reservoir— Kind.	Size, diameter, height.	Capacity, gals.	Cost of foundation.	Cost including foundations.	Cost per 1,000 gals.
[illegible]	Concrete standpipe	40x32	300,000	.....	$5,812	$19.35
[illegible]	Steel standpipe	20x100	235,000	$1,030	6,640	28.25
[illegible]	Reservoir	.....	[illegible]	.....	2,385	10.60
[illegible]	Reservoir	71x26	[illegible]	.....	20,232	26.20
[illegible]	Steel standpipe	25x50	181,000	300	3,550	19.30
[illegible]	Concrete standpipe	45x18	211,000	.....	4,524	21.15
Leicester, Cherry Valley and Rochdale	Concrete standpipe	40x21	197,000	.....	4,976	25.25
[illegible]	Steel standpipe	35x40	288,000	700	4,638	16.10
[illegible]	Steel standpipe	20x100	235,000	.....	5,882*	25.00*
North Chelmsford	Steel standpipe	22x125	355,000	.....	9,772	27.50
[illegible]	Steel standpipe	27x50	214,000	400	5,060	23.60
[illegible]	Steel standpipe	45x40	476,000	839	6,707	14.10
[illegible]	Steel standpipe	25x67	246,000	710	4,979	20.25
[illegible] (Fire Dist. No. 2)	Steel standpipe	35x60	432,000	.....	6,165*	14.30*
[illegible]	Steel standpipe	20x100	235,000	.....	6,835*	29.10*
[illegible]	Steel standpipe	30x40	212,000	613	4,021	18.95
[illegible]	Steel standpipe	30x50	264,000	800	6,000	22.70
Wrentham State School	Steel standpipe	22x50	142,000	368	2,596	18.25

*Without foundation.

struction of a water works system in a small town is the problem of convincing the voters that a water works system will not bankrupt the town. There are always those who believe that a water works system is not necessary or desirable, that the water obtained from private wells is better than that which has been stored in pipes or reservoirs, and that the notion that well water may be injurious to health is nonsense. Such men are generally not to be convinced by any argument which can be put forward. They are the men who object on principle to improved roads, improved schools, and improvements of any kind. But there are always in every town thinking men who honestly believe that the construction of a water works system would be such an expensive undertaking that it would be a great mistake for the town to enter into it. The best solution of this problem—the problem of convincing the honest doubters—is to use the experience gained by other towns of similar size which have already put in works. With this in view, I have collected certain information from the small towns in Massachusetts which have been supplied with water, and tabulated the returns. The great difficulty in getting together this information is the lack of proper systems of accounts. In most cases it is impossible to obtain information of any value from the printed reports, and in many cases it cannot be obtained even from a study of the books. The officials themselves cannot dig out the information. Construction accounts and maintenance accounts are hopelessly mixed, and the vouchers in many cases do not show for what the money has actually been spent.

Perhaps it is only fair to say here that the salaries of superintendents in small towns are from $50 per year up—and some of them do not go very far up, either. As the superintendent is generally, to use the language of one of them, "registrar, clerk of board, draftsman, engineer, etc.," it is not strange that the system of accounts is not very elaborate. The average salary of the superintendents in Massachusetts towns having steam pumping systems is $1,080. The average salary of those where pumping is done by oil engines or electric motors is $818. Of the latter group practically all operate the pumps themselves.

Tables I, II and III, p. 312, shows the cost of pumping stations, pumping plants and distributing reservoirs in small Massachusetts towns where the works are municipally owned. For obvious reasons the privately owned plants are not included, but in general it may be said that the cost, both of construction and maintenance, of privately owned plants is fully as great as that of the plants owned by the towns.

The Field of the Slow Sand Filter—Comparative Capacity Cost Data.

It is generally admitted that the field of the slow sand filter is narrowing while that of the rapid sand, or mechanical filter, is widening. Granting this we believe many readers will appreciate something definite to which to tie in choosing between the two types for a given locality. The choice will naturally rest in any case upon the comparative cost and efficiency of the two types. The question of cost is readily worked out by the designing engineer and the efficiency to be expected will depend upon the characteristics of the raw water to be filtered. Definite figures on the latter point were proposed as a guide by Mr. Robert Spurr Weston of Boston in discussing the paper on "Present Day Water Filtration" read by George A. Johnson before the latest annual convention of the American Water Works Association. The following matter is quoted from Mr. Weston's discussion:

Each locality has its problem and should attempt its own peculiar solution. Undoubtedly, many slow filters were built and installed where rapid filters should have been chosen. Pittsburgh and Washington, D. C., are cases in point, and which have occurred in the writer's practice, but there are other cases where slow filters are not only more economical but more efficient than mechanical filters, and some of these cases are where high rates of filtration may be employed without detriment, because it is obvious that the cost of slow filters per million gallons decreases rapidly and inversely with the rate of filtration.

Generally speaking most engineers who are competent in the field of water purification, that is, those who have studied the problem for a long time in connection with many types of waters, are agreed that mechanical filters are best where the color is over 50 parts and the turbidity over 30 parts per million, or where there are unusual fluctuations in the character of a stream, which ordinarily is but slightly colored or turbid. Again, there are other cases where the water is so clear that there are no nuclei to start the coagulation of the applied chemical, upon which coagulation the efficiency of mechanical filters depends. To use mechanical filters in these cases may necessitate the addition of clay or other suspended matter to produce an artificial turbidity, or the use of an excessive amount of coagulant, so that the coagulant will precipitate by virtue of its large mass.

Before this paper was written the speaker made some studies of the comparative costs of slow sand and mechanical filters for presentation at a town meeting, and came to the conclusion that an average cost of $14,500 per 1,000,000 gals. for mechanical filters, and a cost of $100,000 per acre for slow sand filters were good average figures. These figures are higher in the case of mechanical filters and lower in the case of slow sand filters than given by Mr. Johnson, see ENGINEERING AND CONTRACTING of May, 1914. In Mr. George W. Fuller's report on the Montreal Water Supply, dated July 22, 1910, the cost of sand filters was estimated at $33,840 and mechanical filters $21,091 per million imperial gallons daily capacity. The cost of slow sand filters per million gallons varies with the rate, as the following tabulation, assuming covered filters and appurtenances to cost $100,000 per acre, shows:

Rate of filtration mgd.	Cost per mgd.
10	$10,000
6	16,667
5	20,000
4	25,000
3	33,333
2.5	40,000

The cost of mechanical plants generally varies with the character of the water, and the writer believes the estimates in the following tabulation represent the best practice in plants of moderate size. The costs include the necessary subsiding and coagulating basins; also small filtered water basin or pump-well:

Character of water. Turbidity or color.	Cost per mgd.
0 — 100 or 0 — 25	$14,000
100 — 300 or 25 — 75	16,500
300 upwards or 75 upwards	18,500

The above costs are higher than those of the largest and best plants constructed under favorable conditions, but lower than those of plants where unusual conditions prevail.

A mechanical and a slow sand filter, each of 1,000,000 gals. daily capacity, designed and constructed under the speaker's supervision during 1913, cost $16,000 for the mechanical and $18,000 for the slow sand plant. The cost of engineering for the mechanical filter was unusually high and the slow sand filter was in connection with a deferrization plant, where the costs included a coke trickler, 15 ft. high and 30 ft. in diameter, and where the filter operated at 10,000,000 gals. daily rate.

In the paper under discussion many references have been made to the hygienic efficiencies of plants treating waters which have been disinfected with chlorine, and in many cases these have been compared with waters which have not been so treated. The cost of sand filtration depends upon the rate. In the early days, as will be remembered, slow filters were designed at rates of 2,650,000 gals. per acre per diem, and less, this rate being the German official rate of 100 vertical millimeters per hour. If bleaching powder be used to sterilize the effluent, there is no reason why this rate cannot be raised much higher than 6,000,000 gals. per acre, which is the maximum in ordinary present day practice.

Apart from the cost and efficiency factors in some cases, the speaker is in hearty agreement with the conclusions of Mr. Johnson's paper, namely that the mechanical filter is an efficient and reliable sanitary machine, one that will reduce the death rate due to water-borne diseases. It is true that the zone in which slow filters are used is narrowing, while that in which mechanical filters are used is broadening, but the speaker still contends that there is still a field for the slow filter, namely, where clear waters have to be purified and where local conditions make the cost of water furnished by the slow filter less. In comparing the efficiency and economy of the two types, one should use the same basis. One should not compare the efficiency of filters where the effluent is sterilized with those where no sterilizing chemicals are used.

Demonstration of Water Main Cleanings.—Among the special features provided by the New England Water Works Association at its recent annual convention in Boston, were demonstrations of the mechanical cleaning of water mains. The work was done for the city of Boston by the National Water Main Cleaning Co. of New York City. A section of about 700 ft. of 6-in. water pipe was sucessfully cleaned. The procedure was as follows:

Openings in the ground and into the pipe were made at either end of the pipe being cleaned. By means of a special carrier device, a cable was passed through the line and the cleaning machine attached to the end of it. By means of a windlass and this cable, the machine was drawn through the pipe, effectually removing all of the incrustation and dirt. While the machine was being drawn through, the water was allowed to flow in the same direction as the machine moved, carrying ahead and out through a riser pipe to the surface of the ground all wash water and debris.

Timber Preservation.—A recent report of the American Wood Preservers' Association states that notable progress in timber treatment for preservation was made in 1913. This industry is comparatively a new one in the United States. Less than 30 per cent of the crossties used in this country are treated and practically nothing has been done in the way of preserving wooden poles.

Real progress in the United States dates from 1832, when the Kyanizing process, using bichlorides of mercury, was developed. In 1837 two other processes were introduced, the Burnett process, using zinc chloride, and the Bethel process, using coal tar creosote. These last processes are very largely in use today. The idea of timber preservation at first made very slow growth in this country, on account of the large supply of cheap and durable timbers and the general disregard shown toward economy in the use of natural resources. In 1885 there were only three pressure plants in the United States, and in 1895 only 15. Since then, however, the industry has grown rapidly. In 1913 there were 117 plants.

The report states that 93 wood-preserving plants in 1913 consumed over 108,000,000 gals. of creosote oil, 26,000,000 lbs. of dry zinc chloride, and nearly 4,000,000 gals. of other liquid preservatives. With these the plants treated over 153,000,000 cu. ft. of timber, or about 23 per cent more than in 1912. The output from additional plants unrecorded would increase the totals given.

Panama Dredge Retired.—The seagoing suction dredge "Culebra," built for Panama Canal service, has been retired from service. This dredge was placed in commission at Balboa in January, 1908, and worked in the Pacific entrance channel until last January, when it was transferred to the Gatun Lake level and put to work removing gravel washed into the channel near Gamboa by the Chagres River. During its service the "Culebra" has dug nearly 19,000,000 cu. yds. of material.

Panoramic View of the Buildings and Grounds of the Panama-Pacific International Exposition,

Structural Features of the 150-ft. Steel Framed Dome of the Palace of Horticulture, Panama-Pacific International Exposition, San Francisco, Cal.

Contributed by A. W. Earl and Thomas F. Chase, Assistant Structural Engineers, Panama-Pacific International Exposition.

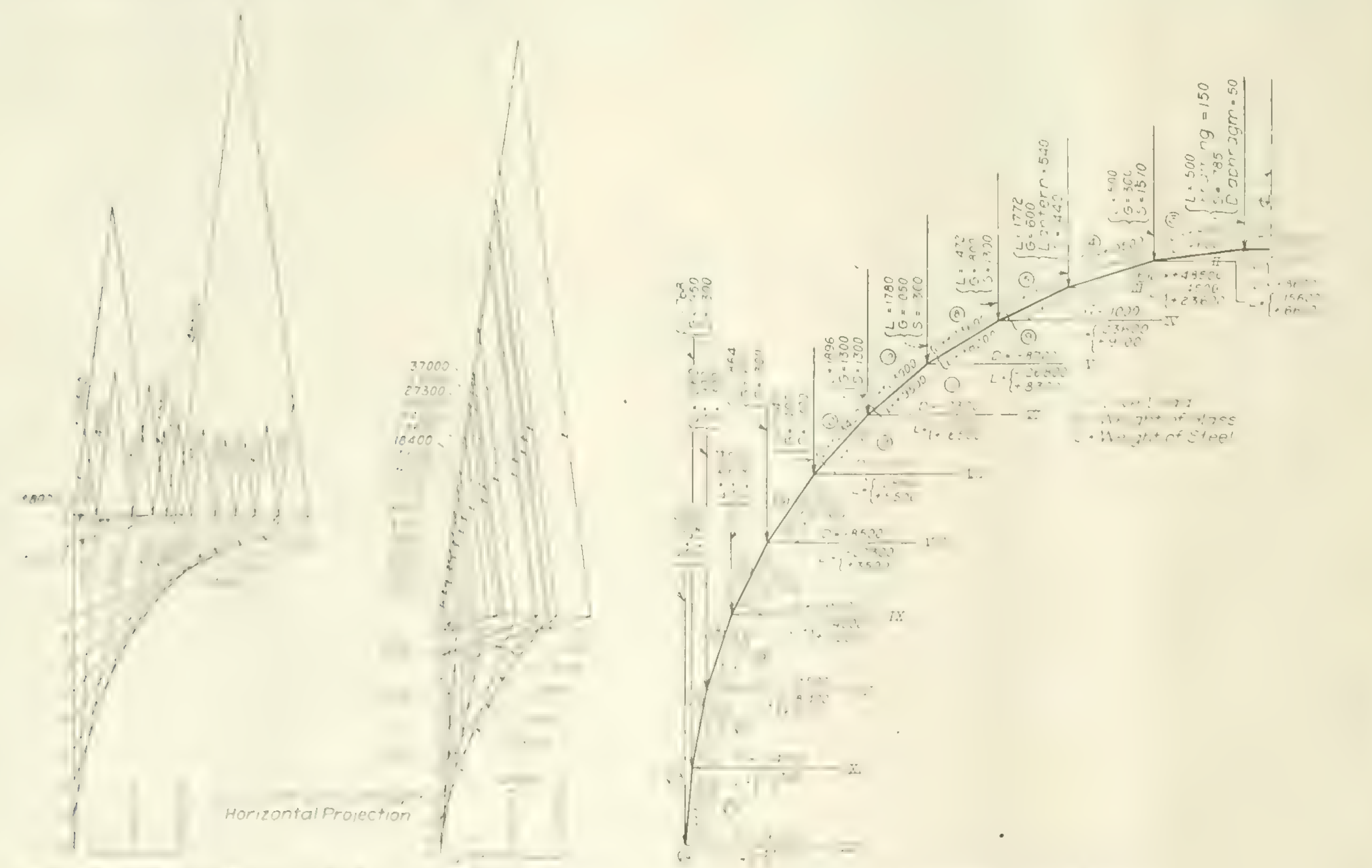
Before designing the steel framed dome of the Palace of Horticulture, Panama-Pacific International Exposition, San Francisco, Cal., a careful search was made of existing literature on framed dome designs, and it appears to the writers that American professional papers are lacking in discussions of this subject, this being especially true with respect to methods of providing for wind and similar stresses produced by other than uniform vertical loads.

The size of this dome, coupled with a desire to keep the framework light and airy in conformity with the character of the building, made a careful study of the problem advisable, and it is believed that a description of the structural features and the methods by which the problems of design were met is of sufficient interest to the engineering profession to merit recording in some detail.

GENERAL FEATURES.

The following brief statement will give a general idea of the building. The Palace of Horticulture is rectangular in plan, approximately 660 ft. long by 300 ft. wide and covers an area of 4½ acres. The eastern, or main portion, which is square in plan, forms the base for the imposing glass-covered dome, the salient feature of the design. This dome is in the shape of a half sphere of 150 ft. diameter resting upon a cylinder of the same diameter and 25 ft. in height. The dome is surmounted by an architectural staff motive in the form of an immense flower basket. The extreme height from the ground to the top of the basket is 186 ft. Rising from the low roof and intersecting the large dome there are eight small glazed half domes, and at the four corners of the square are four octagonal domes of timber construction.

The three main entrances on the north, east and south sides are accentuated with elliptical domes built of wooden lattice work. On each side of these entrance domes are ornamental spires, or flechi, having a height of 130 ft. The building is very richly ornamented and harmoniously colored, and as a whole is graceful in outline and symbolic of the purpose for which it is intended. The western section of the building, which is the main exhibit portion, is a low frame structure of simple wooden construction, rectangular in plan and approximately 300 ft. x 360 ft. The floor in this portion consists of 2-in. shiplap on 2 x 6-in. joists spaced 2 ft. on centers. The joists are in turn supported by 3 x 4-in. sleepers spaced about 6 ft. 8 ins. on centers and embedded in the ground. The floor is designed for a live load of 100 lbs. per square foot. In the front part of the building there is no floor except in the aisles. The frame of the rear portion of the building is of wood and is carried on wooden spread footings.



Figs. 1-3. Stress Diagrams and Summary of Stresses in Ribs and Rings for One Panel of Dome of Palace of Horticulture, San Francisco, Cal.

San Francisco, Cal.—San Francisco Bay Shown in Background—View Taken August 20, 1914.

The heavy concentrated loads in the front part of the structure are carried on pile foundations. The preceding description and Fig. 20 give a general idea of the architectural features of the building.

A better realization of the size of this dome may be obtained by comparing it with some of the best known domes now in existence, the diameters of which are given in the accompanying table:

	Diameter, in feet.
Pantheon, at Rome	142
Duomo, or Sta. Maria del Fiore, at Florence	139
St. Peter's, at Rome	139
United States Capitol, at Washington	135½

The dome of the Horticulture Palace at the Chicago Exposition was 185 ft. in diameter.

STRUCTURAL FRAME OF DOME.

The type of framing is similar to that first used by the noted German engineer, Schwedler. It consists of 24 latticed ribs, each 36 ins. in depth, which frame into a spider at the top and are connected by 11 horizontal latticed rings.

The outer chord of the rib is composed of two 4 x 4 x 5/16-in. angles, while the inner chord is built of two 3 x 3 x 5/16-in. angles, the two being connected by 2½ x 2 x ¼-in. single angle lacing. The lower portions of the ribs are straight, forming the sides of a vertical cylinder, as has been noted previously, the top of this cylinder being the spring line of the dome. The bases of the ribs are supported on plate girders and trusses, the elevation of which is 65 ft. above the floor level.

The spider at the apex of the dome is 6 ft. in diameter, 36 ins. deep, and it is reinforced with four diaphragms. The circular flanges are 4 x 4 x 5/16-in. angles, and the web is 5/16 in. thick. The diaphragms, which are alike, are each composed of four 3 x 3 x 5/16-in. angles with a 5/16-in. web plate.

The horizontal rings, with the exception of the one at the spring line, are 18 ins. deep. This latter ring has the same depth as the ribs.

Intermediate between the main ribs and framing into the rings are 4-in. I-beam purlins. All panels are braced by two adjustable diagonal rods, these rods varying in size from 9/16-in. square at the top to 1⅜-in. square at the bottom. Wooden purlins and the usual skylight bars complete the framework for the glass.

Fig. 11 shows the details of the dome framing. Particular attention is called to the use of angle lacing bars in the ribs and rings instead of the usual lattice bars. While theoretically taking no primary stress these angles are capable of transmitting much more shear than flat lacing bars of the same weight which, while effective for tension, are of an uneconomical section to transmit compression.

SUBSTRUCTURE OF DOME.

A clear idea of the system of framing supporting the superstructure may be obtained by referring to Fig. 11. The dome is supported on eight piers, each 65 ft. in height. Each of these piers is composed of four columns placed at the corners of a quadrilateral. This quadrilateral (see Fig. 13) has two sides normal to a radius of the dome passing through its center, the shorter side being 4 ft. 6 ins. in length and two sides, *A B* and *B C*, 15 ft. 10 ins. in length and diverging at an angle of 45°. The bracing in the parallel sides, *B C* and *D A*, extends from the top of the piers to the bottom, but on the diverging sides, in order to provide a passage way, it had to be discontinued at a height of 14 ft. 7 ins. from the bottom.

Between the tops of the interior columns of adjacent piers are plate girders, 6 ft. 4½ ins. in depth and 57 ft. 5⅞ ins. long. These, with the aid of trusses, which frame from girder to girder across the interior faces of the piers, support the 24 ribs of the dome.

Between the exterior columns of adjacent

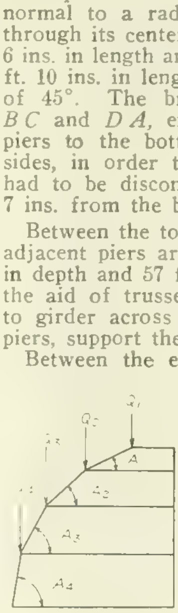


Fig 4

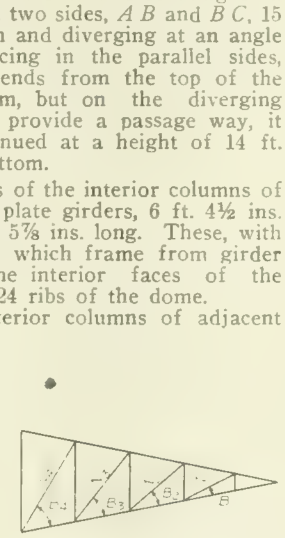


Fig 5 Developed Plan of One Bay

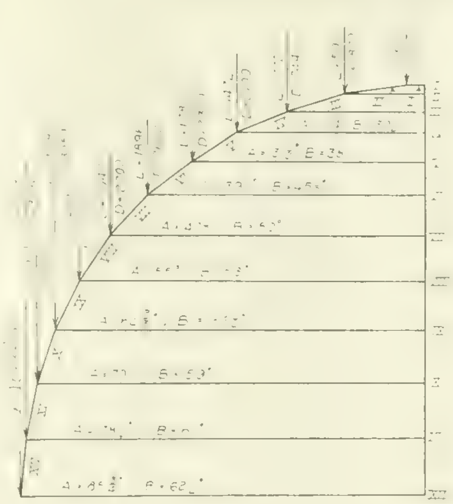


Fig.6. Load Diagram

Figs. 4-6. Diagrams Explanatory of Hütte Method of Determining Stresses in Diagonals of Dome of Palace of Horticulture.

piers steel trusses support a portion of the flat roof and the staff ornaments which form the decoration of the wall surrounding the lower part of the dome. Both the plate girders and the trusses have knee-braces connecting to the columns of the piers. The interior columns of the piers are 20-in., 59-lb. I-beams, laced front and back with 5 x 3 x 5/16-in. angles. Single lacing is used below the knee-braces and double lacing above them, to provide for portal action. The columns are reinforced with plates where the lacing is discontinued between the interior and exterior columns, as will be indicated later. The interior columns are also reinforced at points of maximum bending moment. As mentioned above, the girders supporting the dome ribs are 6 ft. 4½ ins., out to out; the web is 5/16-in. thick, and each flange consists of two 6 x 6 x 7/16-in. angles with one cover plate 14 ins. x ⅜ in. x 28 ft. Further particulars as to the composition of trusses and bracing will be found on the framing plan, Fig. 11.

The plate girders and trusses between the exterior columns are braced together in the planes of their upper and lower chords. This not only serves the purpose of bracing the compression chords, but it forms a stiff ring around the entire base of the dome, which effectually distributes the horizontal wind load to all the piers.

DESIGN OF DOME.

There are two methods commonly employed in the design of framed domes. The first, and perhaps the most common, consists of treating any two diametrically opposite ribs as forming a three-hinged arch, the hinges at the crown being replaced by a circular girder. In this method, wind stresses are taken care of by considering the wind as a live load on one side of the dome and as applied only in the plane of the two ribs forming the arch. Diagonal rods are usually provided in the panels of alternate bays to resist unequal loading and to give general stiffness to the structure.

The second method regards the ribs as members of a true dome, wherein all stresses are resisted by compression in the ribs and by compression or tension in the horizontal rings. Wind and all other than uniform vertical loads are taken care of by placing diagonal rods in every panel of the dome.

By applying the second method outlined above to structures of this type more material is saved than might at first be supposed. In this structure the dome was completely designed by both methods—the second method resulting in a saving of one-third the steel called for by the first design in the portion of the structure above the plate girders. An idea of the relative proportions of the respective frames may be gained when it is stated that, with the same number of ribs in both cases, the ribs in the first design were 72 ins. deep, with chords consisting of two 6 x 6 x ½-in. angles and with 14 x ⅜-in. cover plates, while the ribs as actually built are 36 ins. deep, with chords whose sections do not exceed two 4 x 4 x 5/16-in. angles, with no cover plates.

The writers lay no claim to originality in the following mathematical discussion, but rather have applied formulas and methods already developed in the design of similar structures. In the course of the discussion reference is made to books and papers on the subject by "catchwords," while a complete list of the principal sources of information is given at the end of the article.

The rib and ring stresses were determined by the method outlined by E. Schmitt in a paper presented before the American Society of Civil Engineers, and there is needed no further explanation than the dead and live load stress diagrams and the load and stress summary shown in Figs. 1, 2 and 3, respectively.

The ribs and rings of the dome of the Horticulture Building at the World's Columbian Exposition, Chicago, were taken as a basis for a preliminary estimate of the weights, and from this a tentative design was made. The weight of the roof covering was estimated by direct computation. Using the sections obtained by first computing the stresses, a final estimate of weight was made; the stresses were then recomputed; and the members were modified as required.

The problem of finding the diagonal stresses was slightly more intricate than that of finding the stresses in the ribs and rings. Three methods were used—all based upon different assumptions. The first method is that outlined by "Hütte," the second is somewhat similar to that given by "Hazlehurst" for stresses in a steel water tower, and the third is like that followed in the article on a gas holder published in the Engineering News.

Hütte Method.—The following is explanatory of the Hütte method:

Let Q_1, Q_2, Q_3, etc. (Fig. 4), represent the total dead and live loads per panel, and let P_1, P_2, P_3, etc., represent the total dead loads per panel. Also let A_1, A_2, A_3, etc., represent the angles between the ribs and the horizontal, and B_1, B_2, B_3, etc. (Fig. 5), the angles between the ribs and the diagonals. Finally, let N_1, N_2, N_3 represent the diagonal stress then:

$$N_1 = \frac{Q_1 - P_1}{2 \sin A_1 \cos B_1}.$$

$$N_2 = \frac{Q_1 + Q_2 - (P_1 + P_2)}{2 \sin A_2 \cos B_2}$$

$$N_3 = \frac{Q_1 + Q_2 + Q_3 - (P_1 + P_2 + P_3)}{2 \sin A_3 \cos B_3}$$

The angles A and B were scaled from a carefully constructed diagram and their values are recorded on the diagram in Fig. 6. Further, it will be noted that the diagonals do not extend to the top of the dome but discontinue at ring 3. Calling the diagonal [illegible] 1, N_1, and numbering the others consecutively toward the base of the dome the resulting stresses, in pounds, were found to be:

$$N_1 = \frac{[illegible]}{2 \sin [illegible] \cos [illegible]} = [illegible]$$

$$N_2 = \frac{3{,}250}{2 \sin [illegible] \cos [illegible]} = 3{,}960.$$

$$N_3 = \frac{5{,}030}{2 \sin 39\tfrac{1}{2}^\circ \cos [illegible]} = 5{,}630.$$

$$N_4 = \frac{6{,}920}{2 \sin 47\tfrac{1}{2}^\circ \cos 50^\circ} = 7{,}300.$$

$$N_5 = \frac{8{,}960}{2 \sin 55^\circ \cos 53^\circ} = 9{,}100.$$

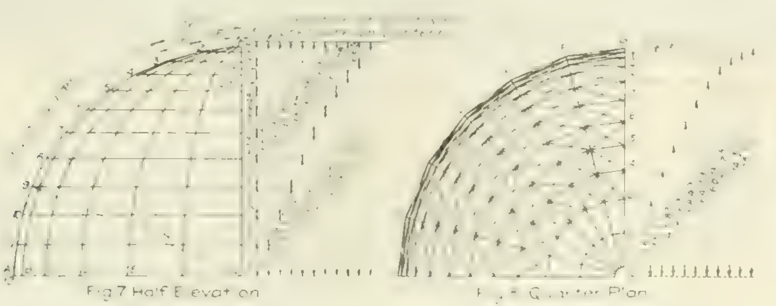
$$N_6 = \frac{10{,}830}{2 \sin 62\tfrac{2}{3}^\circ \cos 56\tfrac{1}{2}^\circ} = 11{,}050.$$

$$N_7 = \frac{12{,}430}{2 \sin 70^\circ \cos 59^\circ} = 12{,}850.$$

$$N_8 = \frac{13{,}680}{2 \sin 78\tfrac{1}{4}^\circ \cos 60^\circ} = 14{,}000.$$

$$N_9 = \frac{14{,}450}{2 \sin 85\tfrac{3}{4}^\circ \cos 62\tfrac{1}{2}^\circ} = 15{,}700.$$

With the exception of the "2" in the denominator, the above formulas are the result of simple resolution of given forces in given directions, and it assumes that half of the dome lying on one side of the meridan through the center of the panel containing the diagonal in question is fully loaded while the other half of the dome carries no load at all. The "2" in the denominator is introduced on account of results obtained by "Müller-Breslau" and others from exact analyses which gave stresses about one-half as large as those given by the original "Schwedler" formula. It should also be noted here that the exact analysis of a dome by methods of least work, while presenting an interesting and instructive academic problem, is by far too laborious and complicated for commercial office use, since the first principle of design has been met when a structure has been designed which safely and economically serves the purpose for which it is intended. The method is thus seen to be largely empirical, and its application requires experience and judgment in the selection of live loads. In this case a live load of 20 lbs. per square foot of horizontal projection was taken.



Figs. 7 and 8. Half Elevation and Quarter Plan of Dome Framing of Palace of Horticulture Showing Type of Framing and Dimensions.

Hazlehurst Method.—On account of the empirical nature of the "Hütte" formula for stresses in the diagonals it was deemed advisable to check the stresses by some method based on wind pressure, which would be the most likely cause of stress in the diagonals of a glass-covered dome of this kind. To this end the method for the solution of the stresses in a water tower to be found in "Hazelhurst's" book on the subject was adapted to the dome design. The system may be briefly illustrated by reference to Figs. 9 and 10.

Let Fig. 9 represent a section through any trussed pyramid with inclined posts and acted upon by wind.

Let P = the total wind shear from all zones above.

Let $p = P$ divided by the total number of ribs; in this case $p = P/6$. In other words the shear is assumed to be equally divided among the ribs.

In addition to the forces p acting at each rib, there will be the horizontal component of the ribs. This component is most easily found by first computing the uplift, at every point, assuming all ribs on the windward side of the neutral axis to be in tension while those on the leeward side are in compression; and further assuming that the magnitude of the tension or compression varies directly as the distance from the neutral axis. Then, knowing this vertical component and the angle of inclination of the rib with the horizontal the horizontal component of the rib stress may be computed. The resultant of this horizontal component of the rib stress and the force p at every rib is then found, and finally this resultant is projected onto the two ring members. When this is done it will be seen that there is still an unbalanced condition at the point. The diagonal furnishes the balancing component. The application of this method to the various diagonals gave the following results:

N^1 =	1,000
N^2 =	1,700
N^3 =	2,700
N^4 =	4,000
N^5 =	6,700
N^6 =	7,100
N^7 =	10,000
N^8 =	13,000
N^9 =	14,500

Third Method.—The third and final method of computing the diagonal stresses was adapted from the "Engineering News" article. It may be seen from a study of the same that the bays most nearly parallel to the direction of the wind take the greatest diagonal stress, which in amount is equal to twice the average of that for all the bays. Thus referring again to Figs. 9 and 10 the horizontal component "H" of the maximum diagonal stress in any zone is equal to the total wind shear "P" from all the zones above divided by one-half the number of panels; in Figs. 9 and 10, $H = \frac{P}{3}$.

Having determined this horizontal component of the stress it is a matter of simple resolution of forces to obtain the total stress in the rod. The preceding method was followed in the present analysis of the dome, using graphic methods which are not of sufficient importance to reproduce.

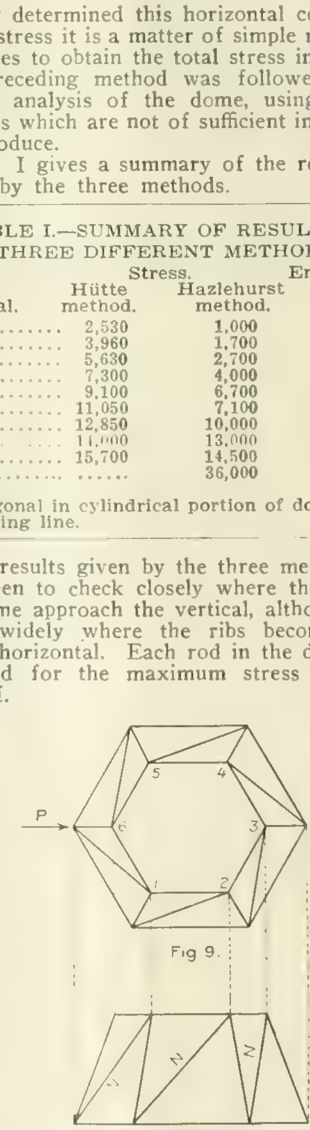
Tabl I gives a summary of the results obtained by the three methods.

TABLE I.—SUMMARY OF RESULTS BY THREE DIFFERENT METHODS.

	Stress.		
Diagonal.	Hütte method.	Hazlehurst method.	Engineering News' method.
N^1	2,530	1,000	600
N^2	3,960	1,700	800
N^3	5,630	2,700	1,500
N^4	7,300	4,000	2,700
N^5	9,100	6,700	4,000
N^6	11,050	7,100	6,100
N^7	12,850	10,000	8,000
N^8	14,000	13,000	11,300
N^9	15,700	14,500	14,000
*N^{10}	......	36,000	36,000

*Diagonal in cylindrical portion of dome below the spring line.

The results given by the three methods are thus seen to check closely where the ribs of the dome approach the vertical, although they differ widely where the ribs become more nearly horizontal. Each rod in the dome was designed for the maximum stress given in Table I.



Figs. 9 and 10. Diagrams Explanatory of Hazlehurst Method of Determining Stresses in Diagonals of Dome of Palace of Horticulture.

It has been pointed out above that the "Hütte" method for finding the stresses in the diagonals is largely empirical. The same criticism may be made of the other two methods; since in each of these the assumption is made that the wind shear is distributed equally among the ribs, an assumption whose truth depends largely upon the stiffness of the horizontal rings. The close agreement

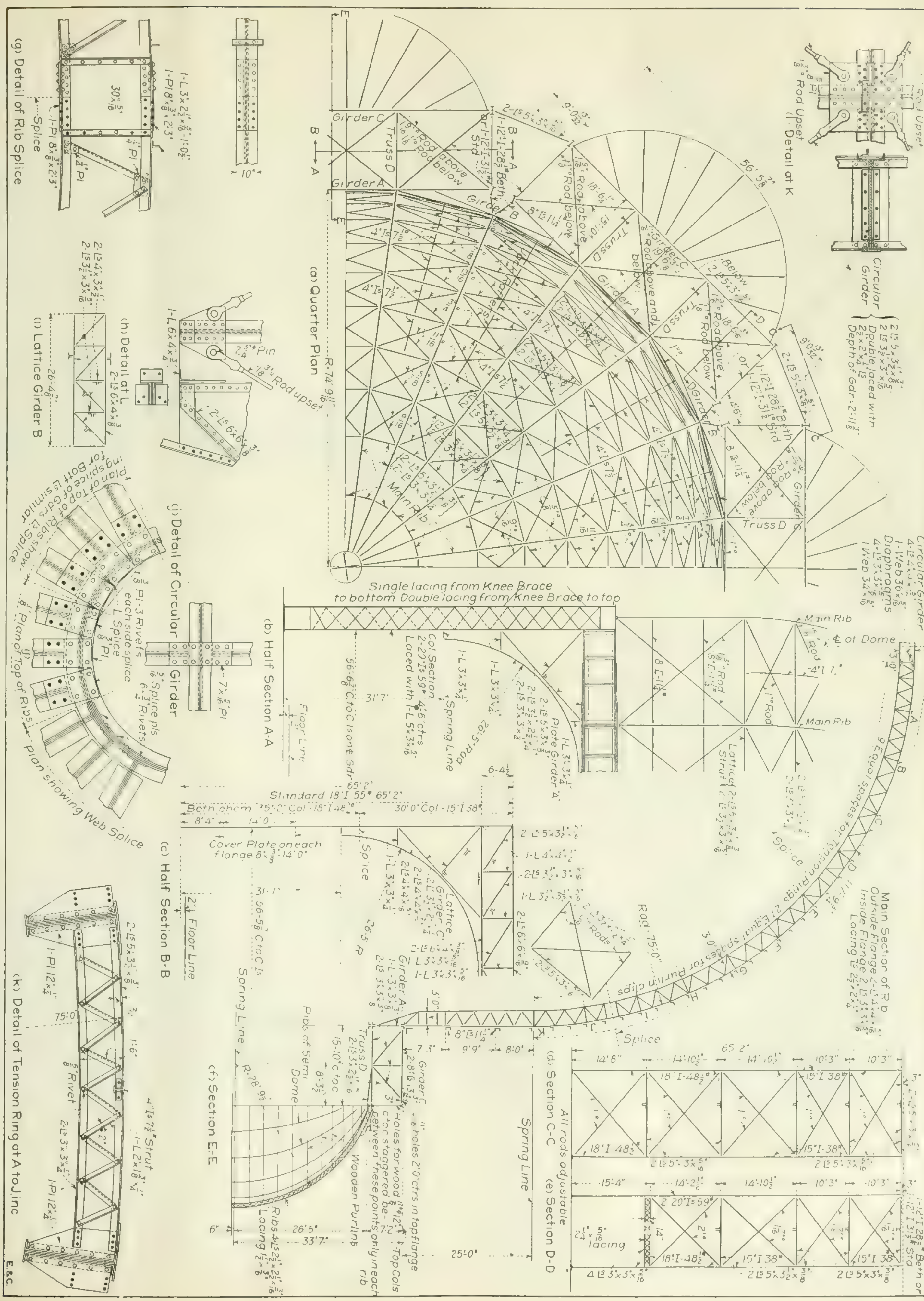


Fig. 11. Design Drawings of Steel Framed Dome and Its Substructure and Details of Principal Features of Same, Palace of Horticulture, San Francisco, Cal.

shown by the preceding table may not always be realized.

It is therefore obvious that in the design of a structure of this kind, a satisfactory design depends to a great extent on the judgment and experience of the designer, and even though the methods here outlined are applicable to the general case of domes, no conclusion based upon this study of one dome should be applied to any other dome without careful consideration of the particular case.

In the selection of members to resist the computed stresses careful consideration was given to various types of ribs and rings. The trussed type was decided upon because of its stiffness and ability to resist the secondary stresses due to bending. This latter consideration is an important factor, since the line of action of the stress between panel points in both ribs and rings is assumed to be a straight line, while as a matter of fact the members are curved. Furthermore, where there is a heavy lantern on the dome, as in the present case, there should be a stiff ring at the top to insure a uniform distribution of this load.

DESIGN OF SUBSTRUCTURE.

In computing the wind load for the design of the dome the wind pressure was taken at 20 lbs. per square foot of vertical projection, which amounts to slightly more than 30 lbs. per square foot reduced 40 per cent for a cylindrical surface. In the design of the substructure, however, it was considered improbable that a wind pressure of 20 lbs. per square foot would exist over the entire area of dome surface at the same instant. It was therefore thought consistent in the following computation to reduce the wind pressure on the smooth surface of the dome by multiplying it twice by the factor 0.6, once because of the circular horizontal section and again because of the circular vertical section. The resulting pressure $= 30 \times 0.6 \times 0.6 = 10.8$ lbs. per square foot of projected area. The lower part of the dome and the lantern was loaded with 20 lbs. per square foot of projected area, since the ornamentation forms pockets for the wind.

The total horizontal load on the substructure was found to be as follows:

	Lbs.
Wind on lantern [illegible]	17,500
Wind [illegible] to spring line [illegible]	100,000
Wind [illegible] spring line [illegible]	75,000
Wind [illegible] of structure to the flat roof of building	120,000
Total horizontal load	312,500

An increase of 25 per cent was allowed for wind stresses over those allowed for dead load stresses, and therefore the equivalent horizontal dead load $= 312{,}500 \times 0.80 = 250{,}000$ lbs.

Two assumptions were made as to the distribution of this horizontal load among the piers: (a) that the piers were loaded in proportion to their respective rigidities; and (b) that they received equal loads. The most unfavorable conditions given by the two assumptions were used in the design. In calculating the stresses in the two non-parallel sides, *C D* and *A B* (see Fig. 13) it was considered that each pier acted independently as a cantilever, but in calculating the stresses in the knee-braces and in the two parallel sides it was [illegible] rigid structure with its neutral axis passing through the center of the octagon formed by the eight piers. The reason for this is apparent since the knee-braces are not very effective when they become normal or nearly normal to the direction of the wind.

(a) Piers Loaded in Proportion to Their Respective Rigidities.—The assumption that the piers take their loads in proportion to t'eir varying rigidities may be more precisely stated as follows: The horizontal load at the top of each pier will be to the total horizontal load as the moment of inertia of each pier about an axis through its center of gravity, normal to the direction of the wind, is to the sum of the moments of inertia of all eight piers about similar axes.

It was found by trial that the direction of the wind giving maximum stresses was, as indicated by the arrow in Fig. 12, parallel to the long diameter of the octagon formed by the eight piers. It was also found by trial that 18-in. 48.5-lb. I-beams fulfilled the requirements for the outer columns and 20-in. 59-lb. I-beams, for the inner columns.

In calculating the moments of inertia of the piers about axes normal to the wind the following well-known formula was used: If *X-X* and *Y-Y* are axes at right angles to each other and *Y-Y* is an axis of symmetry, then the moment of inertia about an axis *U-U*, making an angle α with axis *Y-Y*, may be expressed as follows:

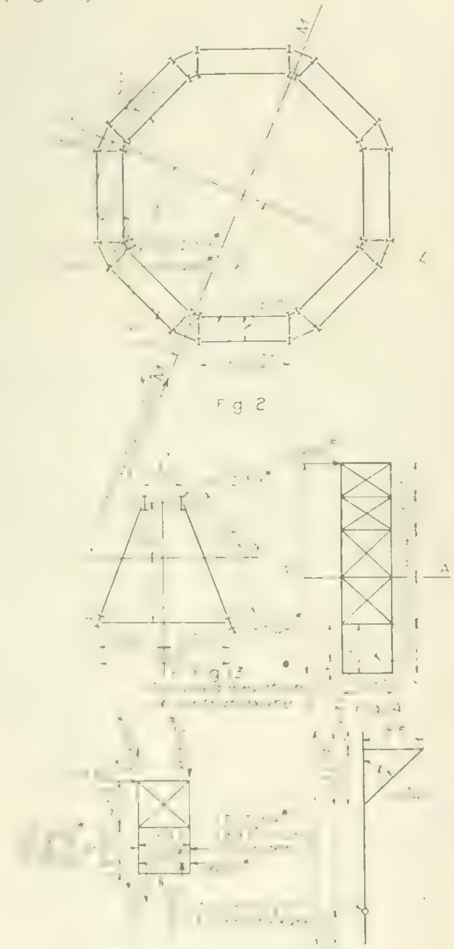
$$I_u = I_x \cos^2 \alpha + I_y \sin^2 \alpha$$

Referring again to Fig. 13 the axis *X-X* is distant from the center line of the 20-in. I-beams,

$$\frac{59 \times 0.83 + 48.5 \times 15.46 - 0.83}{59 + 48.5} = 6.60 \text{ ft.}$$

Then, to obtain:

Moment of inertia of pier about axis X-X (Fig. 13):



Figs. 12-16. Diagrams Explanatory of Methods Used to Determine Stresses and Sections of Members of Substructure of Dome of Palace of Horticulture.

I of two 18-in. I-beams about own axis parallel to X-X $= 2\,(798.3 \cos^2 22°\,30' + 36.2 \sin^2 22°\,30') =$	1,370
I of same I-beams about X-X axis $=$ [illegible]	[illegible]
I of two 20-in. I-beams about own axis $=$	2,344
I of same I-beams about X-X axis $= 34.72 \times (6.60)^2 \times 144 =$	218,000
Total I	485,914

Moment of inertia of pier about Y-Y axis:

I of two 18-in. I-beams about own axis parallel to Y-Y $= 2\,(798.3 \cos^2 67°\,30' + 36.2 \sin^2 67°\,30') =$	298
I of same I-beams about axis Y-Y $= 28.50 \times (8.34)^2 \times 144 =$	286,000
I of two 20-in. I-beams about own [illegible]	97
I of same I-beams about Y-Y $= 34.72 \times (2.25)^2 \times 144 =$	25,300
Total I	311,695

Moment of inertia of pier about axis at 45° with Y-Y $= 311{,}695 \cos^2 45° + 485{,}914 \sin^2 45° = 398{,}804$.

Since there are two piers with axis *X-X* at 90°, four with axis *X-X* at 45°, and two with axis *X-X* at 180° with the direction of the wind, as shown in Fig. 12, we have the following for the sum of the moments of inertia of the eight piers about axes normal to the wind:

For 2 piers with *X-X* at 90° to the wind, $I = 485{,}914 \times 2 =$	971,828
For 4 piers with *X-X* at 45° to the wind, $I = 398{,}804 \times 4 =$	1,595,215
For 2 piers with *X-X* at 180° to the wind, $I = 311{,}695 \times 2 =$	623,390
Total $I =$	3,190,434

Therefore the horizontal loads at the tops of the various piers under the assumption are:

On pier No. 1 with axis *X-X* at 90°, horizontal load $= \dfrac{250{,}000}{3{,}190{,}434} \times 485{,}914 = 38{,}000$ lbs.

On pier No. 2, with axis *X-X* at 45°, horizontal load $= \dfrac{250{,}000}{3{,}190{,}434} \times 398{,}804 = 31{,}300$ lbs.

On pier No. 3, with axis *X-X* at 180°, horizontal load $= \dfrac{250{,}000}{3{,}190{,}434} \times 311{,}695 = 24{,}400$ lbs.

These loads are parallel to the direction of wind; but in order to apply them in computing stresses they must be resolved into their components parallel to the principal axes *L-L* and *M-M* (Fig. 12). The following table gives the components of the loads parallel to the principal axes *L-L* and *M-M* for wind acting as shown in Fig. 12:

Axis.	Pier No. 1.	Pier No. 2.	Pier No. 3.
L-L	0	22,100	24,400
M-M	38,000	22,100	0

The governing loads then are 24,000 lbs. parallel to axis *L-L* and 38,000 lbs. parallel to axis *M-M*.

(b) Piers Receive Equal Loads.—Assuming that the piers are of equal rigidity and that all take equal loads, the load at the top of each pier in the direction of the wind $= \dfrac{250{,}000}{8} = 31{,}300$ lbs.

Resolving this load into its two components parallel to *L-L* and *M-M*, the following loads are obtained for the various piers:

Axis.	Pier No. 1.	Pier No. 2.	Pier No. 3.
L-L	0	22,100	31,300
M-M	31,300	22,100	0

The governing load under this assumption is 31,300 lbs. parallel to either axis.

Considering the results given by the two assumptions we obtain as designing loads, 31,300 lbs. parallel to *L-L* and 38,000 lbs. parallel to *M-M*.

Calculating first the stresses in system *A-B* (Fig. 13): The load applied at the top is $\dfrac{38{,}000}{2}$ (sec. 22° 30′) $= 20{,}600$ lbs. This system may be considered as a vertcal cantilever down to section *A-A* (Fig. 14). Below *A-A* it must be considered as a portal.

The stress in each strut above section *A-A* will be 20,600 lbs.

The maximum stress in the diagonals $= 20{,}600$ (sec. 43° 10′) $= 28{,}200$ lbs.

Below the section *A-A* it was assumed that the columns would take their horizontal loads in proportion to their rigidities. The sections finally obtained at the bottom of the portal frame (the plane of maximum bending) are as follows:

For column *A* (Fig. 13), 1, 20-in., 59-lb. I-beam with an 8 x 1-in. plate on each flange.

For column *B*, 1, 18-in., 48.5-lb. I-beam with an 8 x ⅜-in. plate on each flange.

Then (see Fig. 13), to obtain:

Moment of inertia of column A about axis parallel to X-X:

I of 20-in. I-beam about own axis parallel to X-X $=$	1,172
I of two 8 x 1-in. plates about own axis $=$	1

I of two 8 x 1-in. plates about axis of I-beam = ...1,763

Total I = ...2,937

Moment of inertia of column A about axis parallel to Y-Y:

I of 20-in. I-beam about own axis parallel to Y-Y = ...48

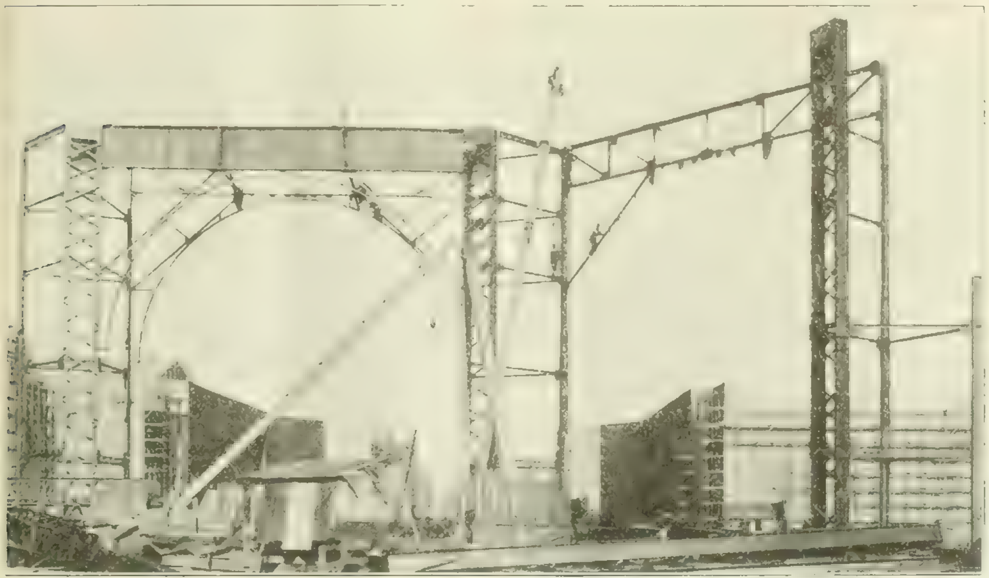


Fig. 17. View Showing Manner of Erecting Dome Piers and Supporting Girders of Same of Palace of Horticulture.

I of two 8 x 1-in. plates $= \dfrac{(8)^3 \times 1}{12} =$ 85

Total I = ...133

Moment of inertia of column A about axis normal to A-B = 2,937 cos^2 22° 30 + 133 sin^2 22° 30′ = ...2,525

Fig. 18. View Showing Manner of Erecting Steel Framed Dome of Palace of Horticulture. View Also Shows Other Details of Building.

Fig. 19. View Showing Steel Frame of Dome and Its Supports, Palace of Horticulture.

Moment of inertia of column B about axis normal to A-B:

I of 18-in. I-beam about own axis = ... 798

I of two 8 x ⅜-in. plates about axis of I-beam = 6 $(9.187)^2$ 506

Total I = ...1,304

The horizontal reaction on column A =

$$\frac{20,600}{2,525 + 1,304} \times 2,525 = 13,600 \text{ lbs.}$$

The horizontal reaction on column B =

$$\frac{20,600}{2,525 + 1,304} \times 1,304 = 7,000 \text{ lbs.}$$

It was considered that the columns would be partially fixed and the plane of contraflexure was assumed at 3.5 ft. from the bottom, which is approximately ¼h (Fig. 14). Then the vertical reaction at the top of the portal

$$\text{frame} = \frac{20,600 \times 35.84}{15.83} = \text{........}46,600 \text{ lbs.}$$

the vertical reaction at the plane of

$$\text{contraflexure} = \frac{20,600 \times 61.50}{15.83} = 80,000 \text{ lbs.}$$

and the horizontal load (see Fig. 15) at the top of the portal = 20,600 lbs.

The tension in diagonal (Fig. 15) (80,000-46,600) sec. 47° 21′ = ... 49,300 lbs.

Compression in upper strut =

$$\frac{13,600 \times 11.1}{14.58} (20,600) = \text{.....} 31,000 \text{ lbs.}$$

Compression in lower strut =

$$\frac{13,600 \times 25.68}{14.58} = \text{...........} 24,000 \text{ lbs.}$$

Stress in Column A.—The dead load on the 16 inside columns = 1,765,700 lbs., or the dead load on one column = 110,000 lbs. The compression on the column due to overturning = 80,000 lbs.

The uplift due to 1½ dome ribs on the basis of 20 lbs. per square foot of projected area = 13,000 × 1.5 × 0.6 × 0.8 = 9,300 lbs.

The maximum bending in column A = 13,600 × 11.1 × 12 = 1,810,000 in.-lbs. The distance of the extreme fiber of the column from its neutral axis = 11.0 *cos* 22° 30′ + 4 *sin* 22° 30′ = 11.70 ins. Therefore the summation of the stress in the column is:

$$\text{Fiber stress} = \frac{1,810,000 \times 11.70}{2,525} = 8,400$$

lbs. per square inch.

$$\text{Dead load stress} = \frac{110,000}{33.36} = \text{.....} 3,300$$

lbs. per square inch.

$$\text{Wind load stress} = \frac{80,000 - 9,300}{33.36} = 2,100$$

lbs. per square inch.

Total stress = ...13,800

lbs. per square inch.

Since this maximum stress occurs at the point where the column is supported by the lower strut of the portal it is well inside safe limits.

Stress in Column B.—The dead load on the 16 outside columns = 344,000 lbs., or the dead load on one column = 21,500 lbs. The compression due to the overturning of the pier = 80,000 lbs. and the maximum bending moment at the bottom of the portal frame = 7,000 × 11.1 × 12 = 930,000 in.-lbs. The distance from the neutral axis to the extreme fiber = 9.375 ins.

The total stress on the column is then as follows:

$$\text{Fiber stress} = \frac{930,000 \times 9.375}{1,304} = 6,700$$

lbs. per square inch.

Dead load stress $= \dfrac{21,500}{20.25} = \ldots$ [illegible] lbs. per square inch.

Wind load stress $= \dfrac{80,000}{20.25} = \ldots\ldots 3,950$ lbs. per square inch.

Total stress $= \ldots\ldots\ldots\ldots 11,710$ lbs. per square inch.

This stress occurs at a point of support and is well on the side of safety.

Stresses in Sides of Pier Parallel to X-X (Fig. 13).—The following arbitrary assumptions were made: First, that 75 per cent of the total load of 31,300 lbs. is taken by the bracing between the inside columns A and D and 25 per cent by the bracing between the outside columns C and B (Fig. 13); second, that there would be sufficient fixing effect at the bases of the pier to cause contraflexure 10 ft. from the bottom of the piers when the structure is considered as a rigid unit, tending to overturn about an axis through the centers of the bases of two opposite piers.

The total moment tending to overturn the structure was found to be 26,525,000 ft.-lbs. The vertical reactions on the piers caused by this overturning moment were considered as

Fig. 20. View of Completed Palace of Horticulture, San Francisco, Cal., Showing Architectural Treatment.

acting on the large interior columns. Let V_1, V_2 and V_3 represent the reactions at the foot of the pairs of columns at distances of 0 ft. 56 ft. and 80 ft., respectively, from the neutral axis of the dome; then if "a" is the reaction on a pair of columns at a unit distance we have $V_1 = 0$, $V_2 = 56a$ and $V_3 = 80a$.

Then $4 \times (56)^2 a + 2 \times (80)^2 a = 26,525,000$; hence $a = 1,050$ lbs.

Therefore $V_1 = 0$, $V_2 = 1,050 \times 56 = 58,800$ lbs., and $V_3 = 84,000$ lbs.

The corresponding horizontal reactions are [illegible]

$H_1 =$ [illegible]

$H_2 = 22,100 \times 0.75 = 16,600$ lbs.

$H_3 = 0$.

The section required at the foot of the knee-braces was found to be 2, 20-in., 59-lb. I-beams [illegible] plate on the outside of each web. The section modulus of this section is 1,112 ins³.

The fiber stresses due to bending in each of the piers are therefore as follows (see Fig. 16):

$$K_1 = \frac{23,500 \times 32.75 \times 12}{1,112} = 8,300 \text{ lbs.}$$

$$K_2 = \frac{16,600 \times 32.75 \times 12}{1,112} = 5,900 \text{ lbs.}$$

$$K_3 = 0$$

The dead load unit stress in each column $= \dfrac{110,000}{23.36} = 4,700$ lbs.

The direct stress due to overturning for each column is as follows:

$$P_1 = 0$$

$$P_2 = \frac{58,800}{2 \times 23.36} = 1,260 \text{ lbs. per square inch.}$$

$$P_3 = \frac{84,000}{2 \times 23.36} = 1,800 \text{ lbs. per square inch.}$$

The maximum stress in the column then is found to be in the pier at the neutral axis and is:

Dead load stress, $P =$	4,700 lbs. per square inch.
Vertical stress due to overturning, $P_1 =$	0
Bending stress, $K_1 =$	8,300 lbs. per square inch.
Total stress $=$	13,000 lbs. per square inch.

The latticing between the 20-in. I-beams was designed for a shear below the knee-brace of 23,500 lbs. and above the knee-brace of

$$\frac{23,500 \times 32.75}{15.92} = 48,000 \text{ lbs.}$$

The stress in the knee-brace is

$$\frac{23,500 + 48,000}{2 \cos 40^\circ 40' \cos 22^\circ 30'} = 51,000 \text{ lbs.}$$

The bracing between the 18-in. I-beams was designed for a shear below the knee-brace of $31,500 \times 0.25 = 7,850$ lbs., and above the knee-brace of $\dfrac{7,850 \times 32.75}{15.92} = 16,000$ lbs.

The stress in the outside knee-brace is

$$\frac{7,850 + 16,000}{2 \cos 40^\circ 40' \cos 22^\circ 30'} = 17,000 \text{ lbs.}$$

ERECTION FEATURES.

The steelwork of the substructure rests upon a timber grillage supported on piles. The timber caps are well drift-bolted to the piles, which is relied upon to produce some fixing effect upon the columns, as has already been noted.

The substructure was completely erected before any of the dome ribs were placed. For raising the ribs of the dome a battered tower was constructed in the center of the dome area, the top of which extended slightly above the tope of the plate girder. Upon this tower there was placed one guyed mast and two guyed derricks. The mast supported the top ring of the dome by means of rod hangers with turnbuckles. By means of the turnbuckles in these hangers and in the guys, an adjustment of the position of the top ring was made.

The portion of the ribs between the girders and the first splice, including the rings and diagonal rods, were erected first. Meanwhile the remainder of the ribs were assembled in pairs on the ground, and a pair of ribs with the connecting rings and diagonal rods were riveted together and swung into place by the guyed derrick, thus completing the erection of the steel. The weight of such a pair of ribs complete was approximately 8 tons.

Figure 17 shows a view taken during the erection of the dome piers and the supporting girders; Fig. 18 shows a view taken during the erection of the steel frame of the dome; Fig. 19 is a view of the completed frame of the dome; and Fig. 20 is a general view of the completed structure.

All field connections of the dome were riveted. In the substructure, however, the field connections were bolted, using ordinary bolts.

The weight of steel for that portion of the structure above the top of the plate girders is 200 tons. This amounts to 8½ lbs. per superficial square foot of roof surface, or 0.304 lb. per cubic foot of volume of the portion considered. The total weight of steel in the dome and its substructure is 503 tons, giving an average weight of 0.358 lb. per cubic foot of volume enclosed by the frame. The corresponding values for the Horticultural Building at the Chicago Exposition were:

Weight of steel per superficial square foot, lbs.	10.7
Weight of steel per cubic foot, lb.	0.312

COST OF BUILDING.

The total cost of the structure and its accessories, taken from the latest compilation of costs and estimates, is \$360,000, being made up as follows:

Steel frame	\$ 43,513
Erecting steel frame	11,115
Pile driving	2,311
General construction	208,658
Plumbing	3,992
Hot water heating	5,775
Boiler house	2,215
Fumigating building	1,047
Lumber, 2,524,244 ft. B. M.	36,042
Electric wiring (estimated)	18,000
Additional plumbing and heating (estimated)	5,000
Decorative painting (estimated)	5,000
Interior cold water painting and painting of steelwork (estimated)	3,000
Incidental additional requirements	14,332
Total	\$360,000

REFERENCES CONSULTED.

A list of the leading discussion upon which the above study is based follows:

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Engineering News, Design of a Gas Holder.—Vol. 66, p. 641.

World's Columbian Exposition, Dome of the Horticulture Building.—Vol. 27, March 12, 1892.

Johnson, Bryan and Turneaure, The Theory and Practice of Modern Framed Structures, Part I.

Greene, Structural Mechanics.

Eddy, Researches in Graphical Statics.

Hütte, Des Ingenieurs Taschenbuch.—Abteilung II, Berlin.

Müller-Breslau, Die Neueren Methoden der Festigkeitslehre.

PERSONNEL.

The plans for this structure were prepared by the Division of Works of the Panama-Pacific International Exposition Co., of which H. D. H. Connick is director and A. H. Markwart assistant director. The dome and its substructure were designed by the writers under the immediate supervision of H. D. Dewell, chief structural engineer of the Division of Works. After the design was completed the stresses in the dome were checked by Richard G. Doerfling, civil engineer.

Bakewell & Brown, of San Francisco, were the architects of the building, and Dyer Bros., of the same city, were the contractors for the fabrication and erection of the steelwork.

Samples of materials available in Panama for making concrete, will be sent to the United States for tests to determine best proportions for the required strength at a minimum production cost, and for making small concrete blocks and standard mortar briquettes.

ROADS AND STREETS

Rules and Regulations Governing Street Traffic in Philadelphia.

(Staff Article.)

Streets in Philadelphia are noted for the orderliness and lack of congestion of the traffic passing through them. Although they are, as a rule, narrow and every reasonable cause exists for congestion, in addition to the difficulties from a severely rectangular plan and a congested business district, no appreciable congestion is apparent to even the casual observer.

Undoubtedly the improvements in rapid transit facilities by the use of shallow subways for suburban street cars and the partial elimination of double track surface lines, thereby greatly simplifying traffic control, have contributed to this result. In addition to these measures, however, Philadelphia has a comprehensive and effective plan for the regulation of vehicular traffic which is enforced in detail.

The regulation of street traffic is under the control of the Department of Public Safety, James Robinson, superintendent, and is a function of the Bureau of Police. In the administration of its affairs this department has sought the co-operation of various trade organizations and by a campaign of publicity and education of the residents of the city in traffic rules, and the value of their co-operation in the enforcement of these rules, valuable results have been accomplished. While it is reasonable to believe that such a plan would be peculiarly successful in Philadelphia, due, perhaps, to the type of citizenship of that city, it, nevertheless, illustrates the possibilities of co-operation in expediting traffic and making city streets safe.

Copies of the rules and regulations pertaining to street traffic compiled with the co-operation of the Board of Trade, Automobile Associations, Team Owners' Associations and the Philadelphia Rapid Transit Company, after official adoption were distributed free to all residents of the city. The regulations state that it is the desire of the department of safety to bring about public benefit by regulation and courteous intervention rather than by embarrassing arrests and legal procedure. Further,

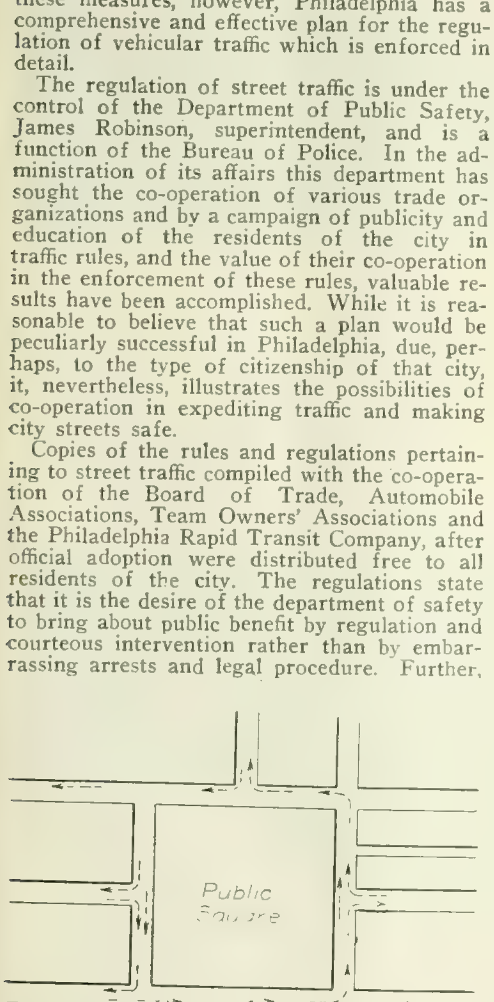


Fig. 1. Showing Rotary System of Traffic Around a Public Square.

that regulations devised may, at times, bring hardship or inconvenience to a few, but will be of the greatest benefit to all.

TRAFFIC CONTROL.

Both the block and rotary systems of traffic handling are used, depending on the adaptability of each system to the needs of different sections of the city. Each has its advantages for use under certain conditions determined by the character and extent of the traffic.

Rotary System.—The rotary system of traffic handling may be defined as a system in which all vehicular traffic on a given street travels in the same direction. This system is especially desirable for use with rapidly moving traffic —motor and street car traffic. On narrow business streets its adoption results in an astonishing reduction in traffic congestion.

At street intersections the use of the rotary system is limited to those points where public squares, circles, or buildings occupy the center of the intersection—the city hall in Philadelphia for example. At such points all vehicular traffic is usually diverted to the right when approaching the intersection, vehicles continuing around the square, or circle, until the street into which they desire to turn is reached. Figure 1 illustrates a typical case of the use of the rotary system at intersections.

Figure 2 illustrates the arrangement by which traffic, both vehicular and street car, goes in different directions on alternate streets. The advantages of this arrangement need not be mentioned; the greatest disadvantage seems to be in the change of type of shoppers in a downtown district and alternate congestion morning and afternoon on streets on which cars run to and those on which cars run from the downtown district.

Block System.—In the block system of traffic control, traffic is alternately halted and allowed to proceed by a traffic officer stationed at important street intersections. This is the system most generally used in the United States. The various signals used by the traffic officers in Philadelphia are stated in the rules and regulations included in this article.

Several methods and devices to aid in traffic control have been devised, among them the semaphore and the use of zones of safety.

TRAFFIC SEMAPHORES.

The traffic semaphore illustrated in Fig. 3 is used by the traffic officer. The great height of the arms permits the signal to be more readily seen than a motion by the officer. The semaphore consists of a movable pedestal supporting a tubular iron post about 12 ft. high at the top of which three semaphore arms are hinged; two for vehicular traffic and one for pedestrians. The method of operation is clearly shown by the illustration. In Fig. 3 the "board is against" vehicular traffic and clear for pedestrians. Figure 4 is a plan of the street intersection at which the photograph was taken.

ZONES OF SAFETY.

For the safety and convenience of persons boarding street cars and the better control of vehicles frequent zones of safety, marked by stretching a rope to indicate a protected area, are used. This rope is supported at intervals of 15 to 25 ft. by iron standards about 4 ft. high. The standards have a flat base and are frequently surmounted by a circular disk painted red, with the words "Zone of Safety" marked on it in white.

The length of a zone is from 50 to 150 ft. and is varied to meet the conditions imposed by traffic at different street intersections. At night the rope is taken down and the standards stored at a convenient point out of the way of traffic. Figures 5 and 6 illustrate a zone of safety along a street car track.

RULES AND REGULATIONS.

The rules and regulations for street traffic are bound in a booklet 3⅝x6¾ ins. in size and are distributed free. An abstract of these rules is given below.

PASSING, TURNING, STOPPING, STANDING AND STARTING.

1. A vehicle, except when passing a vehicle going in the same direction, shall keep to the right and as near the right-hand curb as possible (except on streets where traffic is only in one direction).

2. A vehicle **meeting** another shall pass to the right.

3. A vehicle **overtaking** another shall pass to the left, AND NOT PULL OVER TO THE RIGHT UNTIL SO FAR AHEAD AS NOT TO INTERFERE WITH THE PROGRESS OF THE VEHICLE PASSED.

4. Vehicles shall keep to the right on all avenues and streets divided by a parkway, rope guides, walk, sunken-way or viaduct.

5. A vehicle turning into another street to the right shall turn the corner as near the right-hand curb as practicable. (Fig. 7.)

6. A vehicle turning into another street to the left shall turn around the intersection of the center lines of the two streets. (Fig. 7.)

7. A vehicle crossing from one side of the street to the other on streets where there is two-way traffic shall head in the same direction as the traffic on that side of the street. (Fig. 7.)

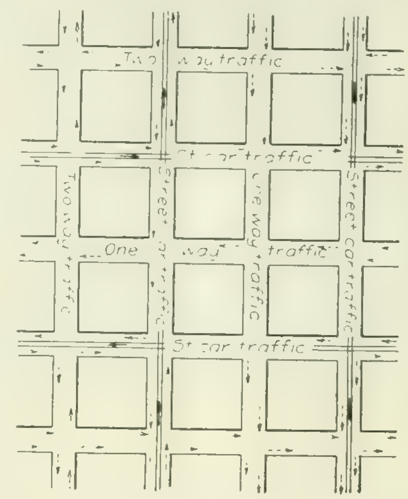


Fig. 2. Showing a Typical Arrangement of One-Way and Two-Way Traffic. Street Car Traffic One-Way.

Note.—Such turns should **not** be made at the street intersections.

8. On two-way streets no vehicle shall stop with its left side to the curb, except on established cab, hack and truck stands.

9. On one-way streets no vehicle shall stop except near the right-hand curb thereof, and so as not to obstruct a crossing, except to reasonably discharge or take on passengers or merchandise, or to allow another vehicle or pedestrian to cross its path.

10. No vehicle shall stop in any public street, except close to the curb (or in the place designated for parking), except in an emergency or to allow another vehicle or a pedestrian to cross its path.

11. No vehicle shall back to make a turn in any street, if by so doing it interferes with other vehicles, but shall go around the block or to a street sufficiently wide to turn without backing.

12. On two-way streets the heavy and slow vehicles shall keep as close to the right-hand curb as conditions permit, in order to allow the rapid moving and lighter traffic to proceed independently.

13. On all streets having a single track, vehicles desiring to pass a street car stopped on the near side of street intersection must move to the left of said street car (except where the traffic officer directs otherwise).

14. On all streets having double car tracks, vehicles proceeding to the right of the street car must come to a stop when said street car is stopping on the near side of the street intersection, and remain at a standstill until said car is started (except where the traffic officer directs otherwise).

15. No vehicle will be allowed to pass to the left of a street car on streets having double tracks.

16. No horse or vehicle shall be driven, propelled or allowed to stand on any sidewalk, except for the purpose of crossing the same when necessary, and then only in the shortest way from the street to the abutting premises.

17. NO VEHICLE SHALL EMERGE FROM AN ALLEY, STABLE, GARAGE OR OTHER AREA OR BUILDING ABUTTING ON A SIDE WALK AT A PACE FASTER THAN A WALK

SIGNALS AND HORNS.

1. A signal shall be given by all vehicles to those behind when slowing up or stopping by raising the whip or hand horizontally when physically possible.

2. A visible or audible signal shall be given by all vehicles when turning while in motion, or in starting to turn from a standstill, indicating the direction in which the turn is to be made.

3. Before backing, ample warning shall be given by visible or audible signal, and while backing unceasing vigilance shall be exercised not to injure those behind.

4. One blast of the police whistle indicates North and South traffic shall stop and the East and West traffic may proceed. Two blasts that East or West traffic shall stop and the North and South may proceed.

5. VEHICLES MUST STOP IN SUCH A WAY AS NOT TO INTERFERE WITH THE PASSAGE OF PEDESTRIANS AT THE CROSSINGS, always stopping so as that no [illegible] feet of the crossing.

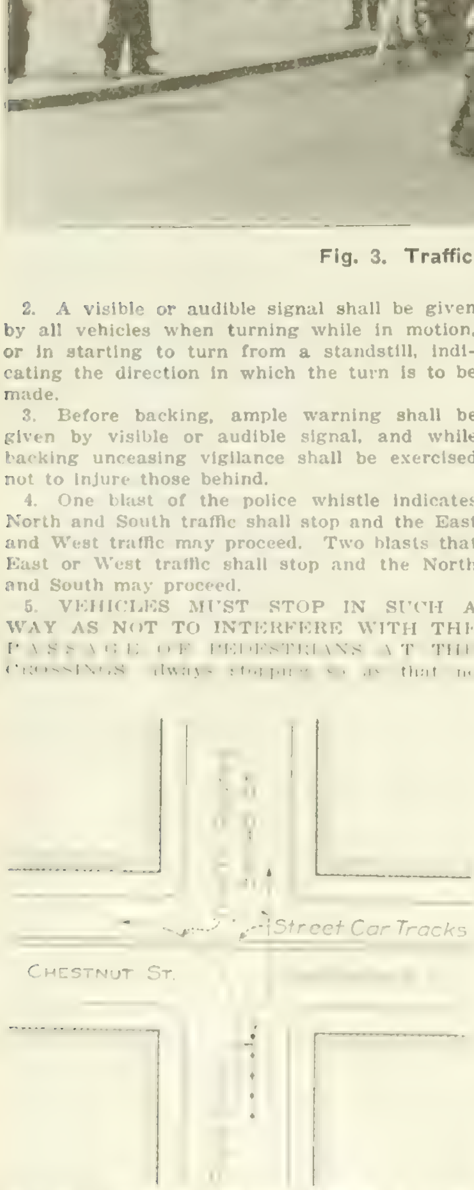


Fig. 1. Diagram Showing Use of a Guide Rope to Divert Traffic.

6. Three or more blasts of the whistle are the signal of alarm and indicate the approach [illegible]

7. Patrolmen when halting traffic will face the line of traffic to be halted, extend the arms at an angle of 90 degrees, with the palm of the hand outward. When the traffic has stopped the arms will be dropped to the side.

8. To start traffic the officer shall face the traffic to be started, and, raising his right arm to an angle of 90 degrees, shall swing or wave it through a half circle in the direction in which traffic is to move, at the same time turning his body in that direction.

9. All automobiles, wagons and other vehicles operated by power other than animal or hand power, shall use and carry a horn sounding only one note (or a bell, when specially permitted).

10. Bicycles shall be provided with a gong or bell, which must be sounded to warn persons of their approach.

11. Every operator of a motor vehicle shall sound his horn when overtaking any person, horse, vehicle or other animal thereon upon a highway, and also shall sound his horn when approaching a street crossing, when rounding a curve or corner, or places where any sign appears, such as "DANGER BLOW YOUR HORN."

12. THE USE OF THE MUFFLER CUT-OUT UPON A MOTOR VEHICLE OR ANY OTHER VEHICLE IS PROHIBITED.

13. The signal of the horn is meant to warn, and must be reserved exclusively for that purpose. Unauthorized persons sounding signals on standing vehicles are subject to arrest.

14. In starting, a little smoke is sometimes unavoidable, but continuous smoke is unnecessary and is prohibited.

15. When signalled to do so by the driver of any horse or other animal, the operator shall stop the motor vehicle, and, if circumstances require it, shall stop his engine until all danger has been avoided.

16. In case of injury or damage to person or property, due to the operation of a vehicle, the operator or driver of the said vehicle shall stop, and, upon the request of the person injured, or any one present, give his name and address and that of the owner of the vehicle.

17. Any operator of any vehicle shall stop upon the request of any police officer in uniform or exhibiting his badge. An operator shall exhibit his certificate and license upon request, and shall furnish all information in possession as to his identity and that of owner of said vehicle.

RIGHT OF WAY.

1. Police, fire department, fire patrol, traffic emergency repair, United States mail vehicles and ambulances for persons and animals shall have the right of way in any street and through any procession.

2. Physicians' vehicles bearing the "Red Cross" or other insignia of the profession serially issued by this Department, will be permitted to pass by traffic officers as soon as possible and will not unnecessarily be delayed.

3. The driver of any vehicle proceeding upon the track in front of a street car shall turn out upon the signal by the motorman or driver of the car.

4. No vehicle or street car shall, except in emergency, occupy any street so as to interfere with or interrupt the passage of other street cars or vehicles.

Fig. 3. Traffic Semaphore. Vehicular Traffic Closed, Pedestrian Traffic Open.

5. A vehicle waiting at the curb shall promptly give way to another about to take on or let off passengers or merchandise.

6. The driver of a vehicle, on the approach of a fire engine or other fire apparatus, shall immediately draw said vehicle as near as practicable to the right-hand curb, and parallel thereto, and bring it to a standstill.

7. The driver of a street car shall immediately stop the car between intersecting streets, and keep it stationary upon the approach of a fire engine or other fire apparatus.

8. During blockades and stoppages a clear space shall be kept open between all street cars at crossings.

9. Street cars and vehicles traveling in the same direction as such cars have the right of way, and the right to the track when meeting any other vehicle going in the opposite direction, and the driver of the vehicle going in the opposite direction shall be compelled to turn entirely off the tracks.

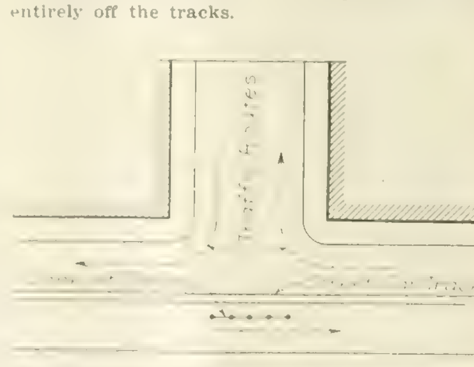


Fig. 5. Diagram Showing the Location of a Zone of Safety.

10. In all streets, lanes and alleys where the cartway is less than twelve feet wide all vehicles must enter or pass over the same as follows: In those running North and South the direction of the traffic shall be from North TO SOUTH exclusively, and in those running EAST AND WEST the direction of the traffic shall be from EAST TO WEST exclusively.

11. Funeral processions on their way to a cemetery have the right of way over other vehicles, and other vehicles are forbidden to drive through them, except as noted in Article I, iii.

12. THE DRIVERS OF ALL VEHICLES MUST LOOK OUT FOR AND GIVE RIGHT OF WAY TO VEHICLES APPROACHING FROM THEIR RIGHT AT STREET INTERSECTIONS.

SPEED.

1. No vehicle shall proceed at any time at a greater speed than the law allows, and is safe and proper under the conditions then obtaining. Where "DANGER! RUN SLOW" signs appear speed must at no time exceed twelve miles per hour.

2. No vehicle shall cross any street or avenue, or make any turn at a dangerous speed, and shall at no time exceed the legal limit.

3. No person shall operate a motor vehicle recklessly, and only at a rate of speed that is proper and reasonable considering traffic at the time, and the width of the highway, so as not to endanger life and property.

4. No person shall drive a motor vehicle at a rate of speed exceeding one mile in two and one-half minutes. In the rural portions of Philadelphia, where signs have been erected "DANGER! RUN SLOW," the speed must not exceed the rate of one mile in five minutes.

5. Trucks and heavy wagons, with horse or horses attached, must not be driven recklessly so as to endanger the public.

CONTROL OF HORSES AND VEHICLES.

1. No horse shall be unbitted in any street or highway unless secured by a halter.

2. No person shall hitch any horse to any tree, shrub or to any telegraph, telephone or trolley pole, lamp post, U. S. mail post, fire alarm post, police signal post or fire hydrant.

3. No one in any street or highway shall remove a wheel, pole, shaft, whiffle-tree, splinter-bar or any other part of a vehicle, or any part of a harness likely to cause accident if the horse or horses start without first unhitching the horse or horses attached to the said vehicle.

4. No one shall let go of the reins while riding, driving or conducting a horse or horses, nor when not riding cease from walking by the head or the shaft of the wheel horse, either holding or keeping within reach of the bridle or halter of the said horse or horses.

5. No person shall ride or drive or have the care of two or more horses harnessed respectively to different vehicles.

6. No firm, person or corporation owning, possessing or having the care of any street car or vehicle of any description shall store, or permit it to be stored or remain unemployed and out of use in any public street or part of the highway, except temporarily in case of emergency or at regular authorized stands.

7. Before leaving any motor vehicle unattended out of sight, the driver shall stop the engine, disengage it and set the emergency brake; or if an electric vehicle, he must remove the plug and set the emergency brake.

VEHICLES.

1. No one in any street or highway shall drive a vehicle that is so covered in or constructed as to prevent the driver thereof from having sufficient view of the traffic at the sides and in front of such vehicle.

2. No one shall drive or conduct any vehicle in such condition, or so constructed, or so loaded as to be likely to cause accident or injury to man or beast.

3. No one shall load a vehicle with iron or other material without deadening so that the material may not strike together and cause unnecessary noise.

4. No one shall load any vehicle with ashes, coal, mortar, snow or similar material so that the matter is scattered along the streets. Drivers of such wagons are liable to arrest.

5. Vehicles used exclusively or principally for advertising are prohibited in the congested districts.

6. No one shall drive a public numbered, licensed or business vehicle who is less than sixteen years of age, unless specially licensed.

7. No one shall ride on the rear end of any vehicle without the consent of the driver, and when riding no part of the person's body shall protrude beyond the limits of the vehicle.

8. Carts, drays, wagons and vehicles for hire, or to carry merchandise or passengers, must not stand or remain in the highways of the city except for so long a time as may be necessary for the reasonable transaction of the business in which the said vehicle may be employed, except at markets or stands, for hackney coaches or other vehicles.

9. No street car shall stop or stand within the intersection of any street.

10. No vehicle shall be kept standing abreast of or opposite another vehicle lengthwise of the street in the main traffic district.

11. Wagons that unload from the side should stand parallel to the curb.

12. No vehicle shall stand on the streets not designated for parking or standing except while loading or discharging a load, and if such vehicle is horse-drawn and has four wheels the horse or horses must stand parallel to the curb and face in the direction of the traffic, except where the congestion may render it advisable to turn the horses at an angle of 45 degrees with their respective vehicles.

13. No one shall crack or use a whip so as to annoy, interfere with or endanger any person, or excite other animals other than which he is using.

14. No coal or other merchandise vehicle shall be stopped or commence to unload on any car track street so as to arrest the passage of street cars.

Fig. 6. View Showing a Zone of Safety in Use.

RESPECTIVE RIGHTS AND DUTIES OF DRIVERS AND PEDESTRIANS.

1. The roadbeds of highways and streets are primarily intended for the use of vehicles, but pedestrians have the right to cross them in safety, and drivers of vehicles must exercise all possible care not to injure pedestrians. Pedestrians should, on their part, never step from the sidewalk to the roadbed without first looking to see what is approaching, and should not heedlessly interfere with the passage of vehicles.

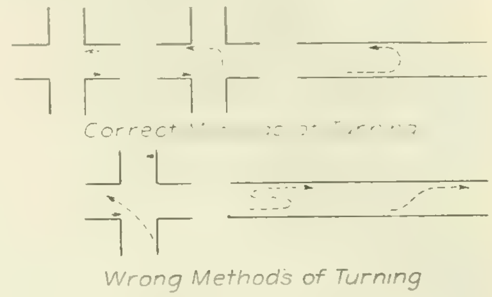


Fig. 7. Diagrams Illustrating Various Methods of Turning.

2. By crossing as nearly as possible at right angles, and at the regular crossings, pedestrians will add greatly to their own safety, facilitate the movement of traffic and make it less difficult for the horses, which often have to be reined in suddenly and painfully to avoid careless and unthinking pedestrians. Nothing in the foregoing excuses drivers from constant vigilance in order to avoid injuring pedestrians.

3. Hand-drawn vehicles and pedestrians should habitually cross the street intersections only and at right angles, as promptly as circumstances permit.

4. Pedestrians must wait the signal of the traffic policeman wherever stationed, and move in the direction of the traffic only.

5. Pedestrians must be cautious, particularly in crossing alleys and obvious entrances for vehicles into buildings and areas.

6. Pedestrians will aid in expediting traffic on sidewalks by keeping to the right while walking, and when stopping for any purpose by doing so on one side and out of the way of a crossing or doorway.

TRAFFIC RULES FOR SPECIAL DISTRICTS.

1. **Walnut Street.** From Front Street to 17th Street, all vehicular traffic will move west at all hours. Waiting vehicles shall stand on the north side and face west.

2. **Chestnut Street.** From 17th Street to 3rd Street. All vehicular traffic will move east at all hours. Waiting vehicles shall stand on the south side and face east.

3. **Market Street.** From Delaware Avenue to 32nd Street. All vehicular traffic will move east on the south side and west on the north side. Waiting vehicles will face in the direction of the traffic.

4. **Arch Street.** All vehicular traffic will move east on the south side and west on the north side. Waiting vehicles will face in the direction of the traffic.

5. **City Hall.** All vehicular traffic moving east and west on Market Street and north and south on Broad Street will keep to the right when passing around the City Hall.

6. **Broad Street.** Vehicles approaching the City Hall will turn to the right. South bound vehicles about to turn east on Chestnut Street will pass to the left of the guide rope at Broad and Chestnut Streets, and those wishing to continue south will pass to the right of the guide rope.

7. All south-bound vehicles desiring to turn east on Arch Street will pass over the intersection of this street into the north-bound traffic, thence to the turn desired.

8. All north-bound vehicles desiring to turn west on Arch Street, will pass over the inter-

section of this street into the south bound traffic, thence to the turn desired.

9. All waiting vehicles will park in the centre of Broad Street, excepting the space between Chestnut and Arch Streets, facing east and west.

. . . .

10. **Metropolitan Opera House.** All vehicles unloading passengers for the Broad Street entrance of the Opera House will approach from the south and will cross to the west side of Broad Street at Parrish Street and then proceed to the entrance. After discharging their passengers they will continue north on the east side of Broad Street and thence east on Poplar Street or north on Broad Street.

11. Vehicles unloading passengers for the Poplar Street entrance of the Opera House will approach from the north, on the west side of Broad Street, and turn west on Poplar Street.

12. All vehicles waiting for passengers at the three Broad Street landings will park in centre of Broad Street starting at a point south of the south line of the Opera House and face in a northwesterly direction at an angle of 30 degrees. This parking will continue south in the centre of Broad Street.

13. All vehicles waiting at three Poplar Street landings will park in the centre of Broad Street, starting at a point of 25 yards north of Poplar Street and face in a southwesterly direction at an angle of 30 degrees. The parking will continue north in the centre of Broad Street.

14. All chauffeurs and drivers of vehicles must remain in their places until they are called or their numbers displayed, or until they are otherwise directed by the police to start.

15. When the opera breaks, all vehicles will remain in their respective places until the proper signal is given by the police to move.

. . . .

LIGHTS ON VEHICLES.

1. All vehicles and bicycles must carry between sunset and sunrise a light or lights in a conspicuous position, so as to be readily seen from the front.

2. Motor vehicles from one hour after sunset until one hour before sunrise must show at least two (2) white lights, visible not less than 200 feet in the direction that the vehicle is proceeding, and except upon motor-cycles one red light visible in the opposite direction; but motor-cycles need display only one (1) white light in the direction in which they are proceeding.

3. The light of the rear signal lamp must be thrown on the rear license tag; and the said license tag must be kept clean from oil, grit and sand.

4. Automobiles equipped with two pairs of lamps or lights must use only the less brilliant pair on Broad St., Allegheny to Washington Ave., Arch St., Market St., Chestnut St. and Walnut St., from River to River.

DEFINITIONS.

1. The word vehicle includes equestrians, horses hitched to a vehicle, led horses and everything including gasoline and electric cars, on wheelers or runners, except street cars, invalid chairs and baby carriages.

2. The word horse includes all domestic animals.

3. The word driver includes the rider and [illegible] and the operator of a motor vehicle or street car.

4. The word curb shall means the lateral boundary of that portion of the street designated for the use of vehicles, whether marked by curbstones or not so marked.

Traction Tests.—[illegible] the U. S. Office of Public Roads has been extended to include motor cars—a Winton Six carrying [illegible] The object in using a motor car for this work is to cover greater distances, and also to determine the resistance offered to motor car traffic, and the effect of that traffic on the surface itself. The mechanism installed in the car records the power required to turn the drive shaft, the speed of the car, the distance traveled and the time consumed.

Method of Labeling and Filing Plans Used by the New York Highway Commission.

Plans prepared by the New York Highway Commission are unusually complete. The method of labeling and filing these plans is standardized and the instructions given are detailed. All plans of completed highways are filed at the main office in Albany. Division offices have on file blueprints of completed roads in the division.

File numbers are usually placed in the lower or left end of the binding strip. Notes are filed on shelves and working drawings and blueprints on racks of standard design.

DEFINITION OF PLANS.

The component parts of a highway plan are classified as follows:

Survey Notes, consisting of preliminary transit and level notes, special data, field inspection of plans, etc.;

Working Drawings, consisting of maps, profiles, cross sections, quantity sheets, sketches of special structures, etc.;

Contract Tracings, consisting of all tracings included in the contract plans;

Land Taking Tracings, consisting of all tracings of parcels of land to be acquired for highway improvement;

R. O. W. Tracings, consisting of all tracings of parcels of land upon which right of access is to be acquired;

Inspection Prints, consisting of tentative contract tracings, on which have been placed inspection notes;

Construction Notes, consisting of all memoranda concerning grades, structures and other miscellaneous data of the construction work;

Final Estimate Notes consisting of all notes pertaining to or necessary for the compiling of the final estimate of quantities.

METHOD OF LABELING AND FILING.

Survey Notes.—Loose leaf notes, as soon as turned in by the surveyor, are removed from leather covers and bound in standard filing covers; the blanks on these covers are carefully filled out in ink. Notes are then placed in book filing cases and the file number carefully printed in ink on the label affixed to the case for that purpose.

Labels corresponding to the front cover for loose leaf books are furnished to be pasted on the front cover of all bound books, and are filled out in the same manner.

Working Drawings.—Maps, profiles, sections, special drawings, etc., are filed in standard 20x26-in portfolios. In order that continuous sheet plans, such as maps and profiles, may be kept in this portfolio after the contract is let, it is necessary that they be limited in one dimension to 24-in.; hence, after advertisement for contract (when active use ceases), plans on continuous sheets 18 ins. or less in width may be folded alternately in direction (accordian plaited) at intervals of 24 ins., while sheets over 18 ins. wide will be folded at intervals of 18 ins. If both sides of paper are used, care must be taken in cutting to leave one fold at each end of sheet to act as a cover. The 18-in. paper folded at intervals of 24 ins. is much to be preferred.

Contract Tracings.—Contract tracings are of standard size and are bound with Dennison fasteners, set ¾-in. in from the left hand edge of the sheet.

In shipping contract tracings care must be taken to roll them glossy side out and to place them in tubes of a proper length to prevent their joggling in transit. Contract tracings must always be shipped by express to enable their being traced in case of miscarriage.

Land Taking Tracings.—Land taking tracings are of standard size, bound in standard L. T. binders, the blanks on which must be carefully filled out in ink. They are bound

FILING CLASSIFICATION.

FORMS
N—Notes.
M—Map.
P—Profile.
X—Cross-sections.
T—Tracing.
B—Blue print or any print; i. e., pictures, white, black and brown prints.
S—Specification.
D—Drawings other than regular M., P., or X.

CLASSES, DIVISIONS, SECTIONS, SUB-SECTIONS.

100—HIGHWAY PLANS. 110—Construction. 120—Repair. 130—Reconstruction. 140—Location. 150—Special work by permit. 160—Relocations. 170—Land taking and R., O. W. 180— 190—Standard details.	Sections 1, State; 2, county; 3, town; 4, city, village, hamlet; 5, institutional; 6, private. Accession No.—Route and Sec., Pet. or Contract No., Ex.—(6 Alb. 1.) or (2593 War. 4.) or (542).
200—	
300—RAILROAD PLANS. 310—Elimination of grade crossing. 320—Grade crossings. 330—Additional tracks (spurs, etc.) 340—Relocation of tracks.	Accession No.—Abbreviation of, Railroad and Road affected, Ex.—(N. Y. C.—6 Alb. 1), (W. S.—N. Y. T. B.), (D., L. W.—310).
400—CANAL PLANS.	Accession No.—Abbreviation.
500—WATER SUPPLY PLANS.	Accession No.—Abbreviation.
600—BRIDGE PLANS. 610—Standard (Department). 620—Special. 630—Foundations. 640—Approaches. 650—Culverts.	Sections 1, steel; 2, wooden; 3, concrete; 4, masonry, 5, forms. Accession No.—Dimensions or Abbreviation. Ex.—(18 x 150).
700—SEWER PLANS.	Accession No.—Abbreviation.
800—DIKE PLANS.	Accession No.—Abbreviation.
900—MISCELLANEOUS. 910—Tables and calculations. 920—Diagrams and charts. 930—Offices and furniture. 940—Office paper, blank forms. 950—Maps, sections (same as 100).	Accession No.—Abbreviation.

System of Numbering for Filing Records of the New York Highway Commission.

with three or five Dennison fasteners placed ½-in. from left hand edge.

R. O. W. Tracings.—Uniform with L. T. tracings.

Inspection Prints.—Inspection prints bear in the lower left corner an inspection print label carefully filled out in ink.

Construction Notes.—Construction notes are treated in the same manner as survey notes.

Final Estimate Notes.—Same as survey notes.

FILE NUMBERS.

The File Numbers for any plan consist of: (1) a letter to indicate the form; (2) a class number; and (3) an accession number.

Some Notes on Macadam Road Construction in Texas.

The important bearing of climate and local conditions of labor, traffic and topography upon the types and methods of road construction suitable for use in various sections of the country are familiar to all experienced road builders. The state of Texas presents problems for the road engineer differing widely from the problems encountered in other states. This is especially true in the central portion of the state where soils of the so-called "black land" type are found. The following notes by J. F. Witt, County Engineer of Dallas County, Texas, were abstracted from a paper before the Texas Road Congress.

GRADING.

Particular attention should be given to drainage and to obtaining the proper cross section. For use in Dallas County the cross section shown in Fig. 1 has been adopted extensively. In prairie sections where the fall in the side ditches is small additional fall is obtained by gradually deepening the ditches as the outlet is approached, no attempt being made to maintain a constant difference in elevation between the ditch bottom and the center of the roadway. Also, wherever possible the roadway is placed on embankments thus materially simplifying drainage problems.

FOUNDATIONS AND BASES.

On Texas "black land" soil it is often economical to use local clayey gravel, or soft, chalky limestone, for a foundation or subbase under macadam roads, especially where satisfactory sub-drainage cannot be obtained. This material will "set up" and form a solid mat to support the base course proper. The thickness of this mat under similar soil conditions should vary with the quality of the material of which it is composed. While rolling the mat sufficient water should be used to keep it moist but not enough to permit it to become "mushy" under the roller.

If the sub-grade is firm and stable it is better, but more costly, to add to the depth of the base course of the macadam roadway proper instead of using the local material as mentioned.

CROWN.

For an equal distribution of wheel loads the crown should be as flat as consistent with the imperviousness and wearing qualities of the surface of the paving metal. Macadam roads should have a crown of from ½ to ¾ in. to the foot; the smaller crown being used on a surface treated with bitumen. On steep grades the crown of waterbound macadam roads should not be decreased, since it is quite important that all storm water reach the side ditches as soon as possible.

WATERBOUND MACADAM.

The base course of all macadam roads should be of sufficient depth to insure thorough interlocking of the stones and is best constructed of ¾-in. to 2½-in. crusher run stone. This course should be thoroughly watered and rolled.

The wearing course of ½-in. to 1½-in. stone is placed, puddled and rolled. After drying the surface should be swept with wire brooms and sufficient ⅜-in. limestone screenings to fill the interstices swept over it. The surface is then finished by an additional watering and rolling.

The total depth of metal used should be sufficient to guarantee the transmission of a wheel load of 1,000 lbs. on an area 2 ins. long by 1 in. wide to the subgrade without excessively loading the subgrade. Assuming an angle of distribution of 45° from the vertical at the point of application of the load, a pavement thickness of 8 ins. results in a bearing of 5 lbs. per square inch on the earth subgrade, a safe load.

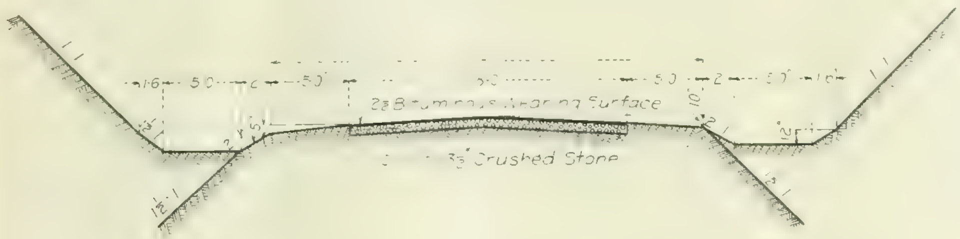


Fig. 1. Cross Sections of the Dallas-Ft. Worth Road in Dallas County, Texas.

ASPHALT MACADAM.

Unless the stone is quite dry it is almost impossible to secure satisfactory results by the penetration method of constructing asphaltic macadam. It seems probable that any moisture in the stone is drawn to the surface by the heat of the binder making impossible a firm bond between the stone and the asphaltic cement. The presence of dust in the stone also often causes trouble. The surfacing should by all means be prepared in a suitable mixer.

Asphaltic cement for use in the Texas climate should conform to the following specifications: Melting point, 160° F. Penetration (Dow method), at 77° F. to be between 45 and 50. A paving cement meeting these requirements in addition to the other ordinary standard tests will result in a pavement that will not crack at 15° F. below zero and will reduce to a minimum the crawling noticeable in hot weather in pavements in which a binder of low melting point have used.

ASPHALTIC OIL TREATMENT.

It is probable that at the present time the most satisfactory method of treating the macadam roads of the state is to apply a suitable asphaltic oil at proper intervals. The secret of success in applying oil lies in a clean surface on which to apply the oil.

After the surface is completed as an ordinary waterbound macadam road it is swept clean. On this surface apply ½ gal. of 60 per cent asphaltic oil to the square yard, using a pressure distributor where possible, and cover evenly with 1 cu. yd. of clean ⅜-in. limestone screenings applied to 120 sq. yds. of surface. Clean sand may be used if screenings are not available.

English Specifications for a Bituminous Concrete Mixing Plant and Details of the Acceptance Test.

A brief abstract of the specifications used in purchasing a plant for preparing bituminous concrete and a description of the acceptance test used is given by W. H. Grieves in a report to the Urban District Council of Surrey (England) and published in the London Surveyor. The adaptability of the plant to work in hand is also discussed.

PLANT REQUIRED.

The following is a short specification of the plant for preparing the bituminous material:

One Coleman's patent stone and sand drier, consisting of one 5 to 6 HP. gasoline engine, with water tank and fittings complete; one drier and furnace capable of drying and heating to a temperature of at least 400 degrees F.; stone, slag, sand, chippings, or any other similar material for bituminous road making, and including all pulleys and fittings and elevator. The whole mounted on a steel frame, lock, steel wheels and axles of sufficient strength to carry the required load. The machine to be fitted with brake and draw-bar.

The total output of bituminous compound of the above machine is guaranteed to be approximately 40 to 60 tons per day, according to weather conditions.

One Coleman's mixer, mounted on steel frame, fore-carriage and lock with steel wheels and axles, the mixer elevated to a sufficient height to allow the material to empty into a cart.

One enclosed elevator for lifting the material into measuring bin, with the necessary raised platform for men to work upon.

One 10 HP. gasoline engine, with water tank and fittings.

One small hand winch and wire rope for elevating bitumen.

Fig. 2. An Asphalt Macadam Road Near Dallas, Texas. Surface 18 Ft. in Width.

One measuring bin of 1,000 to 1,500 lbs.

Brake fitted, and mixer supplied complete with draw-bar. The above machine is capable of thoroughly mixing the material with the bitumen at the rate of about 1,500 lbs. in three to four minutes.

The total cost of the machinery would probably be about $4,500.

The main idea is that the sand should not only be absolutely dry, but it must be heated to a temperature not exceeding 180 degrees C.

WORK TO BE ACCOMPLISHED.

Assuming that the machine could be worked 150 days in the year at 40 tons per day, it would turn out 6,000 tons per annum Allowing for 1 foreman, 2 men feeding the machine, 2 mixing, 1 man at the bitumen tank, 1 engine driver, 2 men wheeling, 3 men laying the material, the cost of sand, bitumen and filler, and adding charges for maintenance, haulage, depreciation, depot rents and steam rolling, the cost per ton would equal about 50 to 60 cts. per square yard.

The cost of the main roads already repaired amounted to over $30,000. There still remains to be repaired, on the main roads, an area of 24,870 sq. yds., which, at $1.20 per square yard, would cost $29,844. Putting the cost of the new method at the higher figure of 60 cts. per square yard, this would amount to $14,922, effecting a saving of about $15,000, or more than enough to pay the cost of the machinery three times over. The machinery could also be used for mixing tar or asphalt macadam, and for this purpose stone taken up from other roads could be used in addition to new stone.

TEST OF THE MACHINERY

The following test of the machinery was made at the manufacturers' works in the presence of a sub-committee appointed by the council for the purpose:

Wet sand was put into a measure 3 ft. 6 in. square by 9 in. deep, and then put into the drier, and immediately came out of the drier at a temperature of 360 degrees F. Within 5½ minutes 750 lbs. were put through the machine, and came out at a temperature of 600 degrees F.

A second lot was put in, to which 4 gals. of water was added to the already wet sand, and within 1½ minutes this was turned out of the machine at a temperature of 500 degrees F. The temperature of the wet sand before it was put in was 70 degrees F. One man fed the mixer with 750 lbs. in seven minutes. Ordinary sand on a dry day was then put in, and the 750 lbs. was passed through the machine in 3½ minutes at a rising temperature of over 400 degrees F. In the combined stone and sand drier 2 tons of wet limestone macadam were put through the machine, and after remaining in the same for ten minutes, came out at 470 degrees F.; 2,100 to 2,200 lbs. of sand were passed through this machine in 8¼ minutes, and came out at a temperature of 465 degrees F.

This mixer will discharge 1,000 to 1,500 lbs. in three minutes.

Stone revolves in this machine at the rate of 14 miles per hour, and sand at the rate of 25 miles per hour.

CONSTRUCTION PLANT

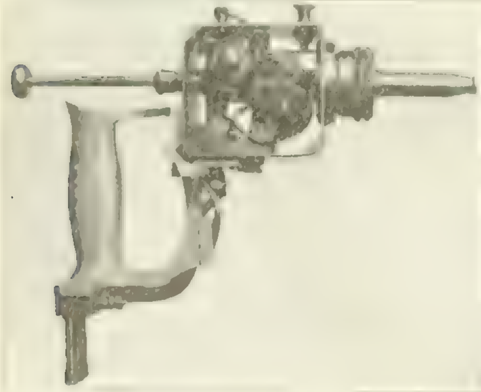
MACHINES DEVICES MATERIALS

A Metal Spraying Process of Protecting Metal Against Corrosion.

(Contributed.)

A metal coating process, which is capable of wide application in the arts but which has greatest interest to the civil engineer because it offers a practicable method of protecting structures from corrosion, is described briefly here. By means of a small simple apparatus, described fully in a succeeding paragraph, a thin coating of molten metal is sprayed over any surface without heating or burning that surface. The process is known as the Schoop process and the spraying device is called a "pistol." It is controlled in America by the Metals Coating Co. of America, Peoples Gas Building, Chicago, Ill.

Present State of the Coating Art.—Despite a rapid advance during the past decade in our knowledge of the internal structure and characteristics of metals in all physical states and a general desire for the conservation of our mineral resources, the steel and iron structures of the engineer and manufacturer remain to a very large extent a prey to corrosion and the art preservative has lagged behind.

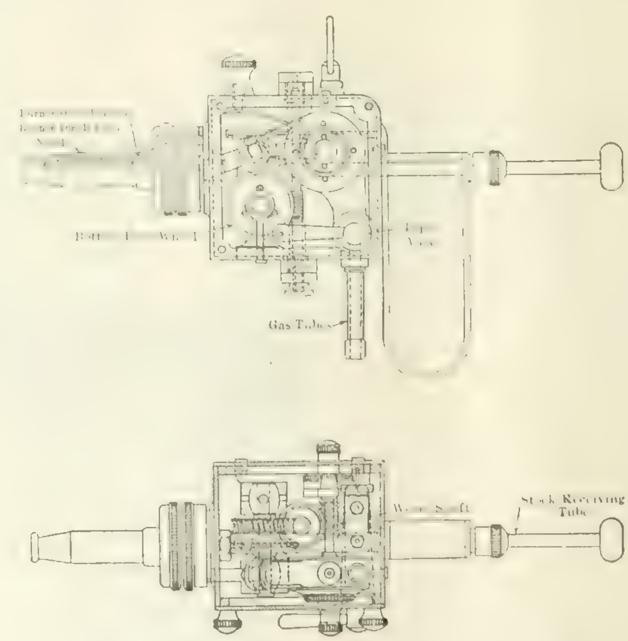


View of Pistol Used in Schoop Metal Spraying Process.

The electro-plating bath, while coating more or less successfully a large amount of metallic hardware with nickel; copper, silver or gold, greatly limits the size of the articles handled. It exposes steel and iron to injurious moisture and chemicals, is unable to deposit the commoner stable and protective metals lead, tin, aluminum and zinc and their numerous alloys, and is of no value towards the protection of the larger industrial products.

The stable metals have long been desired in the form of thin adherent protective coatings not only on other metals, but also on many coherent substances used in the arts, such as wood, paper, leather, glass, stone, cement, textiles, etc., but no practical solution of the problem has been furnished through electrical deposition.

The electrical fusion of copper dust sprinkled over non-conducting surfaces, has been accomplished for purely decorative purposes but this expensive process of limited scope with one metal only has no economical application. Two other processes dealing with a single metal—zinc—occupy the field today. A large quantity of iron and steel receives a purely temporary coating of that protective metal by the wet galvanizing process and a much smaller quantity of metal is more effectively coated by the dry galvanizing method known as "Sherardizing" conducted at high temperature. The various chemical processes for obtaining "finishes" need not be considered here.



Plan and Section of Schoop Metal Spraying Pistol.

The above methods so narrowly restricted in the choice of coatings and using objectionable temperature, moisture or chemicals and unaccommodating apparatus have greatly restricted their field of application and for the protection of structures and apparatus in general the engineer and manufacturer have still to rely mainly upon the temporary arrest of decay afforded by paints, enamels and their substitutes. There is in this steel age no more serious practical problem than the effective prevention of the decay which awaits our most essential structural metals.

Theory of the Schoop Process.—In considering what occurs in the Schoop process it is necessary to bear in mind a singular fact; the sprayed metal moves only 6 ins. between its point of fusion and the object to be coated, yet it does *not* reach the object in a molten state. It even possesses so low a temperature that it can be safely directed upon the hand for a moment without injury and can be continuously impacted upon materials such as cloth, paper, wood, and even celluloid, which would be immediately consumed by the heat of any molten metal. A box of matches can be coated all over to from a seamless metal case without a single match being ignited.

The reason is that the gaseous medium used is much larger in volume at any moment than the drop it has pulverized and is carrying and the gas is expanding so rapidly that its temperature is far lower than that of the spray. A rapid exchange of heat therefore takes place between them which consolidates the molten particles and gives them a temperature far below the melting point. If the particles arrived in a liquid state at the base with the observed velocity of 3,000 ft. a second, they would simply splash on the plate and largely rebound. As a matter of fact, they impact and inter-penetrate freely and the later bombarding particles unite with the earlier ones to form homogeneous compact bodies. In accounting for the observed action of the Schoop spray at the receiving base, it is supposed that the cooled particles of metal just before impinging with great velocity on a hard surface are in an abnormal physical condition. Due to the heat of collision they pass directly into a vapor which condenses and solidifies after penetration and is effectively dovetailed into the base. The hammering and bombardment of the solidified first coat by the minute succeeding particles is practically a process of cold working. The entrained particles liquefy and solidify so rapidly that the metal has not time to return to its natural crystalline state.

The theory just outlined seems to be supported by examinations of the sprayed deposits for the structure of the metal coatings and of the whole bodies produced by the Schoop process is always found to be amorphous and vitreous throughout. This characteristic is of great importance to many large metal industries.

Development of Schoop Apparatus.—The first practical apparatus disclosing and using the principle of Schoop's invention involved the use of a large reservoir of molten metal, the liquid drops of which are pulverized, or atomized, by pressure, and simultaneously projected with great velocity so as to adhere to a base. The apparatus lacked portability for commercial purposes and is practically restricted to spraying lead and tin, which it does successfully.

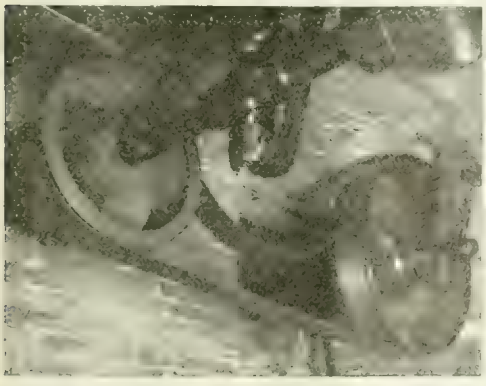
In the next form of apparatus, carrying out the principle of the invention, the large reservoir of molten metal is replaced by a container of manufactured metallic dust resembling a small sand-blast outfit in arrangement; the dust being simultaneously acted upon by heat and pressure and successfully projected and impacted upon a base. This "Cyclone" apparatus is still in use for special purposes, but the high cost of metallic dusts and their liability to oxydize rapidly has restricted the use of this form of apparatus chiefly to zinc.

The next step was to create from solid metal a succession of minute drops and instantly pulverize, or atomize, and project the same under the simultaneous action of heat and pressure. The third form of the apparatus obtained this end, and has been used as a spraying "Pistol" for some time in Europe, though in its latest improved form it was only introduced into this country six months ago. As a result of the efforts of inventors the original apparatus weighing over a ton has been reduced to a compact portable hand instrument weighing only 3½ lbs., which we now describe in detail.

The Metal Spraying "Pistol."—Figure 1 is a view from a photograph of the "Pistol" with one side plate removed from the box. The three hose pipes lead oxygen, hydrogen and air from their respective containers to the burner tube and blast nozzle, which are concentric. The geared air turbine and friction wire feed wheels are also visible in the cut. Provision is made for ready control of the gases. Figure 2 shows the same instrument in plan and section. As already indicated, the small box accommodates only the feed apparatus and the gas and air leads, the whole action of melting, pulverizing and projection taking place at one spot just inside the blast nozzle. The wire is fed down the core of the inner burner piece and at the end of it passes into the flame zone due to the ignition of the mixture of hydrogen and oxygen which simultaneously passes between the two burner pieces. Immediately in front of the flame zone the nozzle discharges a violent blast of air completing the disintegration of the wire and causing its particles to be entrained in an expanding blast of gas and air. Tanks of commercial oxygen and hydrogen with reducing valves and a supply of compressed air are the only requisites for operating the pistol on any metal in the form of wire.

An Excellent Example of Slack Belt.

(Contributed.)

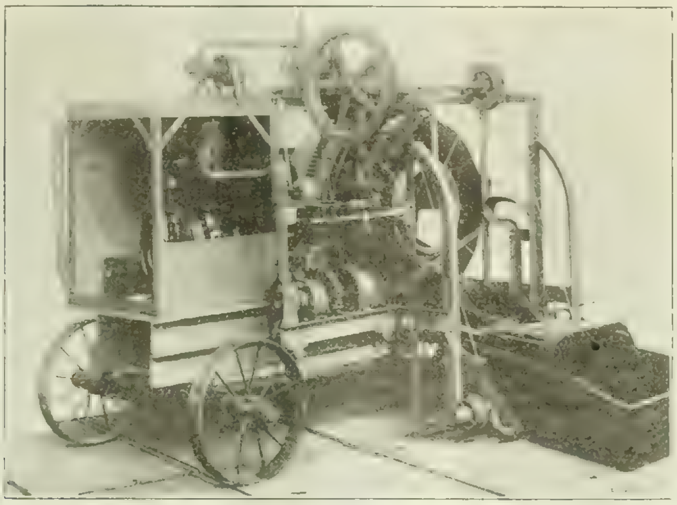


An Interesting Slack Belt Drive.

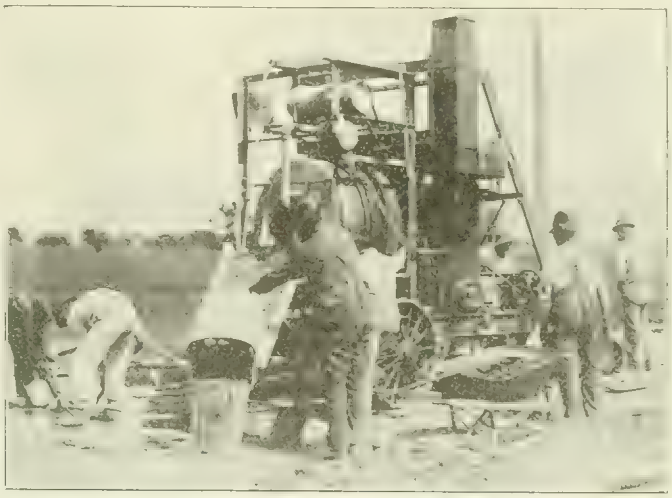
That real slack belts on short centers are quite practicable is conclusively proved by the illustration which shows a 5-in. single leather belt transmitting power from a 15-in. to a 33-in. pulley. The distance, center to center of shafts, is only 5 ft. The speed of the driving pulley is 500 r.p.m. This belt, operating in the engine room of the Lambeth Public Baths, Kensington Road, S. E., England, drives two 36-in. towel washing machines, one 36-in. fan, one 36-in. roller mangle and one 38-in. hydro-extractor. Before treating with Cling-Surface great difficulty was experienced in keeping this belt on the pulleys even with much tension, and the great tension caused the bearings to overheat, a trouble that was enhanced by the high temperature in the engine room—100° F. at the time the photograph was taken. The exceptional slack made possible by the use of Cling-Surface has permitted the removal of all initial tension except that due to the weight of the belt and has been the means of a 25 per cent increase in the pulley contact areas. The belt no longer runs off the pulleys for there is no slip.

A New Smith-Chicago Paver and an Interesting Small Mixer.

A new non-tilting paving mixer known as the Smith-Chicago, and a small mixer of this type for various other purposes, are illustrated here. It is claimed that the design of the paver illustrated and the other mixer mentioned is such that splashing is eliminated without reducing the speed of discharge. In designing concrete mixers of the non-tilting type, it is held desirable that the material in process of mixing continue to be thrown back and forth in the mixer up to the moment of discharge, rather than heaped on the discharge side of the drum in order to facilitate rapid discharge. The design of the Smith-Chicago mixer provides a long steep chute projecting at least two-thirds the depth of the drum; this length and angle being made possible by the concave face of the drum. The mixing blades are located and shaped so that the material is poured continually into the center of the drum. The former feature facilitates the discharge, the latter prevents splashing. The entire contents of the mixer is discharged in from 15 to 20 seconds.



Small Smith-Chicago Mixer.



Smith-Chicago Paver in Operation.

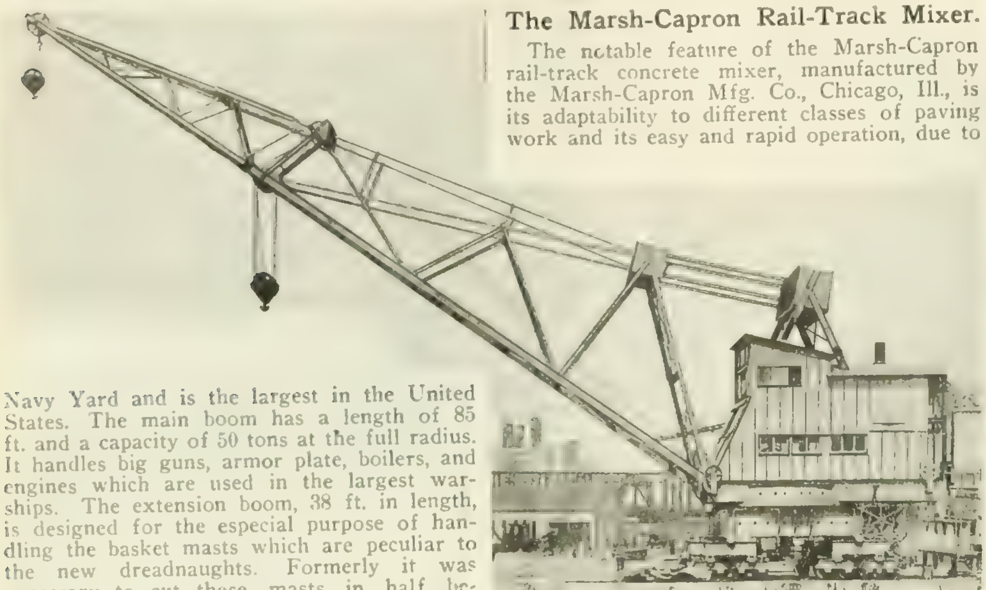
All levers on the paver are banked in one place so that it is possible for one man to control all the operations of the machine. Another feature of interest and value is the provision for the extension of the frame on which the loading skip travels, permitting the use of the skip as an elevator should such an arrangement be desirable for stationary work. The skip is 5½ ft. wide and 6 ins. high at the back, permitting two wheelbarrows emptying into it at the same time. A two-way traction drive with compensating gears is provided on both rear wheels. The speed both backward and forward is ½ mile an hour.

The pavers are made in two sizes of 9 and 14 cu. ft. batch capacity, respectively. Either steam or gasoline power is provided. The capacity of the small paver at 45 batches an hour is 10 cu. yds.; the large paver, 15 cu. yds. The weight of the small paver will vary from 4,300 to 7,500 lbs., depending on the equipment; the large paver, from 6,050 to 9,500 lbs.

The small special machine illustrated has a rated capacity in 3 cu. ft. batches of 5 cu. yds. an hour. A 6 cu. ft. mixer of the same type is also manufactured. The loading skip 10½ ins. The machine is compact, unusually of the small mixer has a charging level of portable and completely housed. The weight of the smaller machine varies from 2,100 to 4,800 lbs., depending upon the equipment. These mixers are manufactured by the T. L. Smith Co., Milwaukee, Wis.

Locomotive Crane for the New York Navy Yard.

One of the best known of American construction tools is the locomotive crane and machines of moderate size call rarely for special mention. There is particular interest, however, attached to the locomotive crane shown by the accompanying illustration, because of its unprecedented size and capacity. This machine is installed at the New York Navy Yard and is the largest in the United States. The main boom has a length of 85 ft. and a capacity of 50 tons at the full radius. It handles big guns, armor plate, boilers, and engines which are used in the largest warships. The extension boom, 38 ft. in length, is designed for the especial purpose of handling the basket masts which are peculiar to the new dreadnaughts. Formerly it was necessary to cut these masts in half because there was no equipment of sufficient capacity to convey them as a unit. The new machine has ample stability for such work. This locomotive crane was furnished in complete working order at the Navy Yard by The Browning Co. of Cleveland, Ohio.

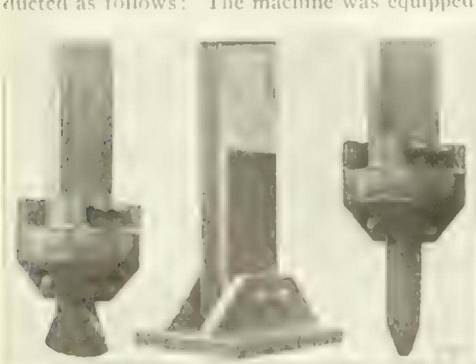


Fifty-ton Locomotive Crane for New York Navy Yard.

A Machine for Trench Tamping and Cutting Pavements for Street Openings.

The machine illustrated was devised for the purpose of providing a rapid and economical means of cutting through pavements where trenches are to be opened and also to effectively tamp backfilled material at a low cost. The interesting feature of the machine is the ease with which it may be converted for service on one type of work after use on another type. The only change necessary consists in substituting the pick or chisel point illustrated for the tamping head, also shown.

A recent test of this device by the Wisconsin Telephone Co. of Milwaukee was conducted as follows: The machine was equipped with the concrete breaking pick and was tried out in competition with a hand picking gang. The machine removed 372 sq. ft. of 6-in. concrete base in 410 mins., an average of 0.91 sq. ft. a minute. By hand labor one man removed 23½ sq. ft. in 71 mins., an average of 0.33 sq. ft. a minute. On asphalt the machine cut 64 lin. ft. of groove in 26½ mins., a rate of 72½ lin. ft. an hour, or in square yards of 36 ins. wide trench, 16 sq. yds. per hour. Hand labor cut, in one case, 6 lin. ft. in 20 mins., and in another 5 lin. ft. in 15 mins., equivalent to 18 lin. ft. an hour or 2 sq. yds. of surface in the first case, and 20 lin. ft. or 2.22 sq. yds. an hour, in the second case.



Chisel and Pick Heads and Tamper Head for Use with Power Picking and Tamping Machine.

The machine is manufactured and sold by Pawling & Harnischfeger Co., Milwaukee, Wis.

A Pavement Picking and Trench Tamping Machine.

The Marsh-Capron Rail-Track Mixer.

The notable feature of the Marsh-Capron rail-track concrete mixer, manufactured by the Marsh-Capron Mfg. Co., Chicago, Ill., is its adaptability to different classes of paving work and its easy and rapid operation, due to special traction equipment. The paving mixer illustrated is fitted with rail tracks around the mixing drum and has all other features of the standard M-C mixer. Special features of the paver are as follows: A 12-HP. steam or air-cooled gasoline engine is furnished. A high and low speed traction gear is provided, the former permitting a speed of 1¾ miles an hour while traveling on the road, the latter, ½-mile an hour for use while laying concrete. Two types of distributor may be obtained, the chute type illustrated, or a conveying distributor for use on wide roads.

The No. 1 type mixer shown in the illustration is engaged in laying concrete base for a wood block pavement on Monroe St., Chicago, Ill. A. N. Todd, contractor. This machine at the time the photograph was taken was laying 670 sq. yds. of 8-in. concrete base, 38 ft. wide, in a 10-hour working day in the downtown section of the city, where conditions are unfavorable to the placing of concrete materials. The same mixer averaged 1,100 sq. yds. of 6-in. base on work completed just before beginning the work on which it is at present engaged.

Rail-Track Concrete Mixer Showing Chute Type of Distributor.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design and to Methods and Cost of Construction

Volume XLII. CHICAGO, ILL., OCTOBER 7, 1914. Number 15.

Effect of Water Waste on Cost of Water and Sewer Service.

The article on water waste which we publish in the water works section of this issue pertains wholly to the additional cost, due to water waste, of constructing and operating water and sewerage works. Our contributor, Mr. R. O. Wynne-Roberts, makes an analysis of the cost of water and sewer service in two hypothetical cities, each of 250,000 population, the one using 50 gals. of water per capita per day and the other 150 gals. It is assumed that the water is filtered, sterilized and pumped against a head of 200 ft., the sewage is collected by the separate system, treated in tanks and sprinkling filters, and the final effluent is disinfected. Considering all annual costs it is found that the total annual cost for the city with the higher water consumption exceeds the corresponding figure for the other city by $284,350. This sum will pay 5 per cent interest on $5,687,000.

While the figures given are intended to serve chiefly as an illustration of the method of analysis they are all drawn from reliable sources and the final figure above quoted is probably a fairly accurate measure of the increase in annual expense due to the waste assumed in the premises. Doubtless some of the assumptions made are open to debate, and some of the values used may be questioned, but the form of analysis is to be commended. Such an analysis may well be made in every city when water filtration and sewage interception and disposal are under consideration, as well as when the cost of an additional water supply must be balanced against that of curtailment of water waste by meterage or rigid inspection.

The Control of Pavement Openings in Small Cities.

There is perhaps no matter which arouses the ire of the average citizen with such frequency, or calls upon the heads of the city administration such forceful expressions of public sentiment with regard to their efficiency, as the frequent opening of newly-laid pavements. Of course there is almost invariably a good excuse for making such openings. Quite frequently the failure of the average citizen to appreciate the necessity and economy of placing subsurface structures prior to paving and his demand for a pavement, even though it provides only the "appearance without the substance of worth," is the primary cause for the frequent unkempt appearance of city streets, often termed "intolerable" by the newspapers.

A city street is primarily an avenue of communication. Communication, in this case, refers not only to vehicular and pedestrian traffic, but to water, sewage, gas, electric power, telephone and other similar mediums of rapid communication. All these things facilitate intercourse and are appurtenances of the street, and a proper conception of the function of the street is fundamental to the correction of many abuses, of which dangerous and unsightly pavement openings is but one. The pavement is then but a portion of the street structure, in point of cost representing but a part of the total investment of money in the street. The education of citizens to an appreciation of this fact will constitute a step in the direction of better control of pavement openings.

While modern pavements with a concrete base are, as has frequently been pointed out, reasonably permanent structures, it must be remembered that from a standpoint of economy it may frequently be better to destroy the pavement rather than to lay unnecessary subsurface structures prior to paving. This fact complicates the solution of the problem. Conversely, it is readily seen that the type of pavement economical for use on a street is a function of the degree of completeness of the subsurface structures. Aside from the inconvenience and unsightliness of pavement openings, there exists a delicate and interesting balance between the relative economy of placing subsurface structures prior to paving and the cost of pavement openings.

The element of danger and inconvenience to traffic due to openings is a maximum on business streets and a minimum on residential streets. Also, the proportional cost of subsurface structures is, ordinarily, of greatest importance on business streets and of least importance on residential streets. Hence, it would appear that while there may be a monetary saving in opening business streets the inconvenience to business, the cost of which is difficult to estimate but which, undoubtedly has a considerable value, enters and, it is believed, openings should be avoided. On residential streets the reverse may be true.

It is, however, possible to avoid destructive openings on residential streets. It is a notable fact that within the past few years there has been a very general reduction in the paved width used on residential streets. This has resulted in allowing the use of parkway areas between the pavement curbs and sidewalks for tree and grass planting. In these parkway areas all water, sewer, gas, power and telephone pipes should be laid. There are, undoubtedly, objections to this practice, but reduced to a basis of service and ultimate economy, both to residents along the street and utilities corporations, the parties most affected and upon whom the burden of unprofitable expense falls most heavily, this practice will result in the greatest good to all. Under the conditions existing in the average American city openings on residential streets are economical; the reduction of the damage occasioned by such openings is the end to be sought.

Various makeshifts have been resorted to by some cities to reduce the pavement opening nuisance. In several cities ordinances have been passed prohibiting openings for a period of years after the completion of the pavement, in one case, five years. Other ordinances require all house connections to be laid to curb lines prior to paving; still others, that connections made after the pavement is laid must be accomplished by tunneling under the pavement.

The whole question of abolishing pavement openings hinges on the value to be placed on the clean and orderly appearance of the street and the convenience of traffic. From a financial standpoint pavement openings are economical, from an aesthetic standpoint, which also has a monetary value to the city, such openings are a nuisance. The residents of different cities assign different values to each factor.

Does Publication of Costs Endanger the Contractor?

A successful contractor whose practice it was to keep very careful costs of his construction operations once refused the editor permission to publish some of these costs because they constituted his *only* stock in trade. This contractor was unusually expert in organizing construction forces, in selecting and handling men, in planning construction plants, in purchasing supplies and materials and he had a reputation for honesty and general reliability. His unit costs were less than those of his competitors; in general he bid more closely than did his competitors, and his profits were more certain and frequently larger than were theirs.

This example is not stated because it is exceptional. Many contractors refuse to permit publication of their costs. It is of course their privilege. To refuse publication on the plea that their "stock in trade" is thus made free to their competitors is, however, not logical reasoning. Considering the example cited, it is evident that these construction costs were not his "stock in trade," but were merely the results of his managerial ability and his business integrity which were his real "stock in trade." As we see it, this contractor and other contractors who conceal their costs fear two possible dangers from publication: First they fear that competitors will imitate their methods and be thus enabled more frequently to underbid them. Second, they fear that engineers knowing the actual costs may lower their estimates and force the contractor to take work at lower prices. Let us consider the force of these suppositions.

Personal knowledge of the operations of several competing groups of contractors in widely separated parts of the country is had by the editor. As a rule the contractors in each group operate in a limited area and bid against each other on nearly every job coming up during the course of the year. Although competitors, these men are usually friendly, clannish and even "chummy." They know each other and each other's methods thoroughly. Each man frequently visits the work the others are carrying on, often traveling considerable distances to do so. It is not necessary for one of these men to read about his competitor's methods since he is familiar with them as a result of direct observation. The experienced contractor knows a good method when he sees it and is quick to adopt it without any published prompting. He can tell at a glance if a job is being handled well and economically. As a result of this practice of inspection the alert contractors in any group employ substantially the same methods. Therefore the only ones who conceivably might profit from the description of a construction method to the disadvantage of the one giving out the description are familiar with the method before it is described. The reason for describing it is to help men in other localities whose operations, as a rule, do not interfere with those in the locality in which the method originated. Since costs are a direct result of conditions and methods there can be no more objection to their publication than to the publication of descriptions of methods. In either case objection to publicity is illogical since, as we have pointed out, methods and costs in any locality are continually undergoing a process of adjustment which trends toward a common level.

Nowadays nearly all engineers keep cost data with reference to the contracts under their charge. These data are kept with sufficient accuracy to serve the engineer's purpose in making estimates. While it is not the engineer's purpose to spy upon the contractor such cost data as he compiles are entirely adequate to show whether the contractor is making an exorbitant profit or not. Further than this, the engineer's interest in this connection does not extend. He expects the contractor to make a fair profit and is much distressed if the contractor is losing money. This is well shown by the fact that engineers usually make all the concessions possible when contractors are working at a loss. Therefore, since engineers keep cost data sufficient to their requirements, the contractor should not withhold the publication of such data in the belief that they will be turned to his disadvantage by engineers.

It should be borne in mind that there are other forms of valuable out-put data which can be published without throwing any light upon the contractor's profits whatsoever. Only cost data of the dollar and cent variety, covering the entire job, taken into account with the contract price, serve to indicate the contractor's profit or loss. If the contractor appreciates the value of giving publicity to his work and wants to help others without giving an insight to his profits he can readily do so.

It is seldom necessary, in fact it is sometimes altogether undesirable, to give unit costs for every item on a large construction job. Thus where a large part of a job consists of trenching and pipe laying, for example, and other less common features are included in the same contract, preference may well be given to the latter in giving out cost data. Thus the contractor may often give out highly interesting, valuable and complete cost data on a part of his job without furnishing any idea to the outsider of the amount of his profit or loss on his entire contract.

Again, where there is a large amount of trenching and pipe laying, as in the above illustration, typical sections 1,000 ft. or more in length may be selected for observation and subsequent publicity purposes, even if costs are not kept on the entire job, or if it is considered undesirable for any cause to give out costs relating to the entire job.

Finally, it should be borne in mind that one can entirely avoid the dollar and cent form of presentation and still give publicity to valuable data in a very convenient form. To the editor's mind one of the most valuable articles, giving out-put data, ever published in this journal occupied less than a column of space and didn't have a dollar sign in it. The article related to the laying of pipe sewers in a Wisconsin city. All the soil and other conditions were briefly but adequately described, as were the construction plant and methods employed. An intensive time study was made per 1,000 ft. of pipe line completed. The data obtained were recorded as so many days and fractions of days required by each class of labor employed in laying 1,000 ft. of sewer complete. This included supervision by foreman, the contractor's personal time spent on the work, etc. We commend this form of presentation to the contractor who has an ill-defined dread of seeing the dollar mark too prominently displayed in published accounts of his work.

Alert contractors are coming to appreciate the value of publicity. The benefits of publicity can be obtained as here suggested without revealing profits and without greatly reducing the serviceableness of the data made public. Moreover, as shown herein, the dangers arising from the publishing of even very full and complete data of the dollar and cent variety are largely but figments of the imagination.

New Ruling on Reinforced Concrete Flat Slab Construction by Building Department of Chicago.

There has been much discussion concerning the methods used in designing reinforced concrete flat slab floor systems, and the checking of a number of designs shows that there is little agreement among designers of this type of construction in determining the thickness and reinforcement of flat slab floors. The Commissioner of Buildings of Chicago has recently signed a ruling on the design and testing of reinforced concrete flat slab floor systems which is the result of long study and numerous extensometer tests of full sized structures. On account of the thoroughness of the investigation and the prominence of those engaged in making the tests, certain features of the ruling are of special interest to designers of reinforced concrete construction. Before laying down rules governing the design of flat slab floors it was first necessary to define terms, the following definitions being given in the ruling:

Flat slabs, as understood by this ruling, are reinforced concrete slabs supported directly on reinforced concrete columns with or without plates or capitals at the top, the whole construction being hingeless and monolithic without any visible beams or girders. The construction may be such as to admit the use of hollow panels in the ceiling or smooth ceiling with depressed panels in the floor.

The column capital shall be defined as the gradual flaring out of the top of the column without any marked offset.

The drop panel shall be defined as a square or rectangular depression around the column capital extending below the slab adjacent to it.

The panel length shall be defined as the distance center to center of columns of the side of a square panel, or the average distance center to center of columns of the long and short sides of a rectangular panel.

The ruling requires that the least diameter of any concrete column shall be not less than one-twelfth the panel length, or one-twelfth the clear height of the column, while the minimum total thickness of the floor slab is to be determined by the formula:

$t = 0.023 L \sqrt{w}$,

in which

t = total thickness of slab in inches,

L = panel length in feet,

and w = total live and dead load in pounds per square foot.

The diameter of the column capital is required to be at least 0.225 of the panel length, this diameter being measured where the vertical thickness is at least 1½ in., while the slope of the column capital must nowhere make an angle of more than 45° with the vertical.

The ruling states definitely the method which shall be used in designing the flat slab in a square panel, the following description indicating the division of the panel into strips:

For purposes of establishing the bending moments and the resisting moments of a square panel, the panel shall be divided into strips known as strip A and strip B. Strip A shall include the reinforcement and slab in a width extending from the center line of the columns for a distance on each side of the center line equal to one-quarter of the panel length. Strip B shall include the reinforcement and slab in the half width remaining in the center of the panel. At right angles to these strips, the panel shall be divided into similar strips, A and B, having the same widths and relations to the center line of the columns as the above strips. These strips are for designing purposes only, and are not intended as the boundary lines of any bands of steel used.

These strips shall apply to the system of reinforcement in which the reinforcing bars are placed parallel and at right angles to the center line of the columns, hereinafter known as the two-way system, and also to the system of reinforcement in which the reinforcing bars are placed parallel, at right angles to, and diagonal to the center line of the columns, hereinafter known as the four-way system.

In designing an *interior* panel for the two-way system, the negative bending moment at a cross section of each strip A, at the edge of a column capital, or over it, is to be taken as $WL^2/15$; the positive bending moment at a cross section of each strip A, midway between the column centers, as $WL^2/30$; the positive bending moment at a cross section of each strip B, in the middle of the panel, as $WL^2/60$; and the negative bending moment at a cross section of each strip B, on the center line of the column, as $WL^2/60$. In these formulas W = the live and dead loads per linear foot of each strip, and L = the panel length in feet.

In designing an *interior* panel for the four-way system, the negative bending moment at a cross section of each strip A, at the edge of the column capital, or over it, is to be taken as $WL^2/15$; the positive bending moment at a cross section of each strip A, midway between the column centers, as $WL^2/40$; the positive bending moment at a cross section of each strip B, in the middle of the panel, as $WL^2/60$; and the negative bending moment at a cross section of each strip B, on the center line of the column, as $WL^2/60$.

In designing the *wall* panels, wherever the coefficients 1/15, 1/30, 1/40 and 1/60 appear in the formulas for either the two-way or the four-way system the coefficients 1/12, 1/25, 1/33, and 1/50, respectively, shall be used in the moments for wall panels supported on concrete columns and girder; when brick walls are used to support partly the wall panels these walls are required to be stiffened by pilasters or piers, and the corresponding coefficients shall be 1/10, 1/20, 1/27 and 1/40.

In making calculations for the bending moments at sections away from the column capital the point of inflection is to be taken as being one-quarter of the distance center to center of columns, both crosswise and diagonally, from the center of the column.

The ruling considers the flat slab as a beam, and the tensile stress in the steel and the compressive stress in the concrete can be computed on that basis as soon as the loads and moments are obtained. The distribution of load in rectangular panels is considered as a function of the cube of the span. After the structure is completed the ruling requires that it shall be tested, the deflections being used as a measure of the workmanship. Overloading is checked by requiring that all buildings used for the manufacture and storage of goods shall be placarded, a card stating the safe live load per square foot being placed at each floor.

WATER WORKS

Economics of Water Waste in Cities.

Contributed by R. O. Wynne-Roberts, Consult- [illegible]

A perusal of the technical press and of the papers and discussions at the various conventions of water supply engineers will indicate that the subject of water consumption and waste is one of considerable importance.

That water is being wasted in cities is recognized by all engineers, and that it cannot be completely eliminated is admitted by all. But the quantity which is used or wasted in *excess* of allowable or unpreventable waste, plus that actually consumed for all legitimate purposes, represents a tangible and potential source of wealth. The means by which such wealth can be conserved is dependent on the method adopted and the manner in which it is organized.

The influences which affect the consumption of water are the nature of the industries, the wealth and habits of the people, the extent to which water is used for fountains or other ornamental objects, watering of lawns, street sprinkling and other public purposes. Climate has also a very considerable influence especially as to the amount used for sprinkling purposes, and that which is wasted in winter to prevent freezing. It is probable, however, that the most important factors in determining the consumption of water is the degree of care taken to detect leakage and other waste, and the fact as to whether the water is sold by measure or otherwise.

It will be assumed that the actual consumption of water on the North American continent is on a more generous scale, and that the climate as a rule is less humid and consequently the gardens and streets receive more watering than in Europe. This, however, cannot account for the great difference in the average consumption per capita. While it is not always a sure method of comparison to consider the consumption in any one city with that of another, owing to the different conditions which obtain, when several cities are compared this limitation loses some of its force.

Even granting that the actual use of water for ablutionary and other purposes is more generous here than in European cities, it is palpable that the excess will not be great.

An American authority (Turneaure and Russell's Public Water Supplies, p. 22), estimates that the daily average for domestic

purposes would be	25 gals. per capita
Commercial purposes	20 gals. per capita
Public purposes	5 gals. per capita
Loss	20 gals. per capita
Total	75 gals. per capita

There are many cities in the United States and Canada that are thriving on a smaller average. Domestic allowance of 25 gals. daily per capita is more than ample, and when it is borne in mind that a large number of industries, railways, etc., do not use city water, as a perusal of the Cincinnati Sewerage and other reports clearly show, an allowance of 20 gals. per head per day for industrial consumption is high. The loss of 20 gals. is excessive, while the preceding allowances evidently include waste also.

The average water consumption of 22 of the largest European cities is about 40 gals. per head per day. If this figure is increased by 25 per cent it will represent a reasonable quantity and includes unpreventable waste, which occurs in all cities.

COMPARATIVE ECONOMIC ANALYSIS OF TWO HYPOTHETICAL CASES.

The writer will for the purposes of this article assume two hypothetical cities, each of 250,000 inhabitants. One city will consume 50 gals. per capita daily, and the other 150 gals. Approximate estimates, based on published statements which will be quoted, will be submitted to show the economics of waste. The water is supposed to be filtered, chlorinated and pumped 200 ft. high or equivalent in pressure, and distributed. The sewage will be collected on the separate system and treated bacteriologically.

There is ordinarily one ratepaying consumer in every six inhabitants, so that in each city of 250,000 population there will be about 42,000 water consumers.

The daily consumption of water in the two cities will average:

$250{,}000 \times 50 = 12{,}500{,}000$ gals., and

$250{,}000 \times 150 = 37{,}500{,}000$ gals., respectively, and to these figures must, of course, be added an allowance in capacities of mains, pumps, etc., to meet the fluctuating hourly flows.

The water works of the six largest cities in Wisconsin, as shown by the 1911 report of the Wisconsin Railroad Commission, supplying an average of 81 gals. per head daily, cost about $187.25 per consumer, but eliminating one city, where the cost exceeded the average by nearly 100 per cent, the mean of five cities was $151.42 per consumer. Accepting this as the basis of cost, the water works system for a city of 250,000 inhabitants or 42,000 consumers will be about $6,360,000. It is therefore reasonable to estimate that for 50 gals. per head daily the cost will be about $6,000,000 and for 150 gals. per head $7,000,000. The extra $1,000,000 will annually cost 5 per cent for interest and say 2½ per cent for depreciation, a total of $75,000 per annum.

The cost of pumping water will be about 6 cts. per million foot gallons (see 1911 report of Wisconsin Railroad Commission, p. 453), so that 12,500,000 gals. raised 200 ft. will cost about $54,750, whereas 37,500,000 gals. raised to the same height will cost $153,250, an extra cost of $109,500 per annum. The average cost in 21 cities in Wisconsin in 1911 was $16.70 per million gallons pumped and on this basis the annual cost would be $76,000 and $228,000, respectively.

Filtration and sterilization will cost about $3.37 per million gallons (Fuller, Baltimore Works, Engineering Record, May 9, 1914), to which is added the cost of pumping into filters, making a total of $3.50. So that in the first city, this work will cost about $16,000 and in the second $48,000, a difference of about $32,000 per year.

The distributing mains sufficient for domestic, industrial and fire purposes should satisfy the National Board of Fire Underwriters (1910) standard and also allow for the usual maximum fluctuations in consumption.

The National Board of Fire Underwriters' general requirements may be expressed by the following equation:

$Y = 1{,}020\sqrt{X}\,(1 - 0.01\sqrt{X})$, in which $Y =$ gallons per minute and $X =$ population in thousands.

The cities under consideration have about 250 thousands population so that to satisfy the above requirements:

$Y = 1{,}020\sqrt{250}\,(1 - 0.01\sqrt{250}) = 13{,}570$ gals. per minute.

The consumption of 12,500,000 gals. per day is equal to an average of 8,700 gals. per minute, and 37,500,000 gals. daily represents an average of 26,000 gals. per minute, but the maximum rate will probably be about 150 per cent of the average. Therefore the relative requirements of the two cities will be as follows:

	City No. 1.	City No. 2.
Fire purposes	13,570	13,570
Domestic and industrial purposes.	8,700	26,000
Add 50 per cent for maximum hourly demand	4,350	13,000
Total gallons per minute.......	26,620	52,570

In other words, the capacity of the mains in the city No. 1 will be only one-half of that in No. 2.

According to published statistics, Proceedings American Water Works Association of 1911, p. 75, distributing mains absorb about 64 per cent of the total capital expenditure. There are other published figures which conflict with this percentage, but as the above result was evidently obtained by careful analysis of at least 22 different city water works, it may be taken for granted that it is reliable. The cost of the distributing mains in No. 1 city will therefore be about $3,840,000 and in No. 2 about $4,480,000, an additional expenditure of about $640,000. The cost of operating distribution works may be estimated at $2.50 per million gallons pumped (1911 report of Wisconsin Railroad Commission, p. 445), which in the first city would amount to about $11,400, and in the second about $34,200, a difference of $22,800 per annum.

EFFECT OF WATER WASTE ON COST OF SEWERAGE WORKS.

After having distributed the water to the people, the city must also provide sewers to drain it away after use or misuse. The lateral sewers are, of course, designed for flows which normally, will only partially fill the pipes. The trunk sewers must be calculated so as to be ample to accommodate the districts served.

Supposing that it was necessary to provide one main conduit to the outfall works, that the grade was 1 in 5,000 and that no ground water was admitted, the diameter of such a sewer to convey 12,500,000 gals. per day would have to be about 54 ins. and for 37,500,000 gals. 82 ins.; this does not take into account the hourly fluctuations otherwise the diameter would have in each case to be larger. Accepting Cincinnati prices (1913 Cincinnati Sewer Plan Report, p. 252), the cost of these conduits would be:

For 54-in. diameter sewer—	
Trenching, 8 cu. yds. at $1.25............	$16.00
Concrete, 1.38 cu. yds. at $15............	20.70
Cost per lin. ft..........	$36.70
For 82-in. diameter sewer—	
11.3 cu. yds. at $1.25......................	$13.95
2.24 cu. yds. at $15........................	33.60
Cost per lin. ft..........................	$47.55

That is, to convey three times as much sewage as would be required economically, the ratepayer would have to pay about 60 per cent more in capital expenditure (and, of course, in annual taxes for interest and maintenance) on such trunk sewers. When the cost of vitrified pipe sewers are analyzed, it will be found that the extra cost for sewers laid to carry say 1,200 gals. per minute and 3,600 gals. per minute will be in the following ratio:

For 15-in. pipe, grade 1/600—	
Vitrified pipes	$0.75
Trenching 10 feet deep....................	.70
Total per foot run.........................	$1.45
For 24-in. pipe, grade 1/800—	
Vitrified pipes	$2.00
Trenching 10 ft............................	.70
Total per foot run.........................	$2.70

The extra cost is 86 per cent, so that the additional cost to convey three times a given volume of sewage increases as the diameter of the sewers diminishes. The cost of sewering a city is probably about the same as to provide water mains, perhaps more, because water mains operate under pressure and sewers by gravity; the former are always full while the latter are generally only partially full and consequently larger in diameter or dimensions. Many of the lateral sewers could not be reduced in size even if the water consumed was maintained at 50 gals. per capita, but many of the larger sewers could, and the saving in capital expenditure would be tangible.

The next item of expenditure is for sewage disposal works. While to some degree it is true that an extravagant use of water does not necessarily entail the construction of works to treat sewage in proportion to the flow or volume, it nevertheless means works of a greater capacity than would be necessary in the case of an economical water consumption, for tanks and pumps must be in some relation to the hourly quantity of sewage. The capacity of the pumps (if any) must be more than equal to the maximum hourly flow of sewage, with reserve pumps and power as well, in case of breakdowns or other contingencies, common to such plant. The velocity of the flow of sewage through the tanks must not for long periods exceed a critical limit. To attain this condition it is evident that tanks capable of treating 37,500,000 gals. daily will be much larger than would be necessary for a discharge of one-third that volume.

Mr. George M. Wisner in his report (report on sewage disposal, Sanitary District of Chicago, 1911), supplies an interesting table of costs which is copied below:

Type of tank	Nominal period of settling.	Gals. per capita.	Cost per capita.
Imhoff	3 hours	200	$1.44
Dortmund	4 hours	200	0.84
Straight flow...	8 hours	200	.77
Straight flow...	6 hours	200	.58

As the Imhoff tank is now prominently before us, its estimated cost per capita will be provisionally accepted. To maintain the same velocity for 150 gals. per capita daily, the cost will be:

$\$1.44 \times \dfrac{150}{200} = \$1.08$, and for 50 gals. per day

$\$1.44 \times \dfrac{50}{200} = \$0.36.$

The writer does not contend that the cost of these tanks will be in strict proportion to the flow of sewage, as there are items of expenditure which are not proportionate, still, taken as an entity, the cost will not seriously exceed the above. Mr. Clark, when discussing a plant in course of construction in Baltimore, stated that the detention period with Imhoff tanks would be two hours (see Engineering Record of July 4, 1914), which is the ordinary standard detention period,—consequently to maintain this detention period as closely as possible the number or sizes of the tanks must in the cases under present discussion be approximately in the same ratio as to dimension and cost. But to allow for contingencies, assume that the cost would be $1.10 and 40 cts., respectively, per capita, then,

$250{,}000 \times \$1.10 = \$275{,}000$, and $250{,}000 \times \$0.40 = \$100{,}000$, a difference of $175,000, which at 5 per cent interest and 2 per cent maintenance, etc., means $12,250 per annum.

Perco-filters again are designed to deal work at 2,000,000 gals. per acre daily. Columbus filters were designed for this rating. Mr. George W. Fuller states in his book on Sewage Disposal, p. 697, that his practice has been to specify for average conditions, a 6-ft. filter at an average rate of 2,000,000 gals. per acre per day. This would be for a sewage flow of separate sewers approximately 100 gals. per capita daily. It is contended that perco-filters will deal with approximately the same quantity of organic matter per acre per day, regardless of the degree of dilution. In other words, the organic matter from a residential city will, in the aggregate, roughly amount to the same quantity whether it is contained in a large or small volume of water. The British Government (Local Government Board) ordinarily requires in the case where there is no land for subsequent treatment or a large river for effective dilution, a filter 6 ft. deep and one acre in area for each 1,000,000 gals. of sewage (dry weather flow), but when, as is the practice in America, a river is available for the ultimate oxidation of the filtrate, then the area is about one-half. It is therefore reasonable to postulate that for a consumption of 50 gals. per capita daily the area of filter will be about one acre per 1,000,000 gals., and for 150 gals. per day an area of one acre for each 2,000,000 gals., on which basis the respective areas will be about 12½ and 18¾ acres. A reserve must be added, say ten per cent, which will increase the areas to 13.75 and 20.625 acres. Mr. Wisner estimates the cost of perco-filters at about $28,000, while according to the experience of other cities the average was about $38,000 (Engineering Record, Aug. 22, 1914). Basing the cost at $30,000 per acre, then the first case will require an expenditure of $412,500 and the second $618,750, a difference of $206,250, which at 5 per cent interest is equivalent to an annual burden of $10,312. The cost of operating and maintaining these filters may be placed at $2 per 1,000,000 gals. (Engineering Record, Aug. 22, 1914); this will amount to $12.5 \times 365 \times 2 = \$9,125$, and $37.5 \times 365 \times 2 = \$27,375$.

Sterilization of the filtrate by hypochlorite of lime costs about $1.67 per 1,000,000 gals. treated (1913 Cincinnati Sewer Plan Report, p. 567) when 1½ parts of available chlorine per 1,000,000 parts are applied. This for the first case would mean $7,620, and in the second case $22,860.

Summarizing the items already mentioned in the foregoing observations, the following results are obtained:

	No. 1, city consumption, 50 g. p. c. d.	No. 2, city consumption, 150 g. p. c. d.
Water Works—		
Interest and depreciation on total capital on waterworks per annum	$450,000	$525,000
Annual cost of pumping	54,750	153,250
Annual cost of filtration and sterilization	16,000	48,000
Annual cost of distribution works	11,400	34,200
Total on water works	$532,150	$760,450
Sewerage:		
Annual cost of operating sewage tanks, plus interest	$ 7,000	$ 19,250
Annual cost of operating perco-filters, plus interest	29,750	58,312
Sterilization of filtrate	7,620	22,860
Total for sewerage	$44,370	$100,420

Adding the two expenditures together we arrive at a rough idea of what it means to the ratepayers:

	No. 1 city.	No. 2 city.
Water works	$532,150	$760,450
Sewer works	44,370	100,420
Total cost	$576,520	$860,870

The difference of $284,350 capitalized at 5 per cent will represent the decent sum of $5,687,000.

The writer has advisedly adopted published figures and in doing so has quoted the authorities, but it is manifest that the above estimates serve only as indications, and therefore each city must be considered separately, although the foregoing statistics answer as direction posts to those who will carefully analyze the financial results to be obtained in their own cities. The foregoing will afford sufficiently safe basis to warrant a close scrutiny into the relative cost to the ratepayers of an economical versus an extravagant consumption of water. Furthermore, in those cities where the water supply is controlled by companies, the foregoing observations will suffice to show what waste means to them, and to their customers. The dividend producing power of any franchise depends on an efficient management and this in its turn means the stoppage of all preventable waste.

An Effort to Secure Uniformity in Compiling and Reporting Statistics of Water Purification Plants.

About a year ago a committee was appointed by the New England Water Works Association to consider the subject of "Statistics of Water Purification Plants," and to recommend certain standard forms for reporting such statistics. A progress report was made at the annual convention of the association held early in September of this year.

The committee divided the subject into five topics, namely, descriptive data, statistics of analysis, engineering statistics, financial statistics and vital statistics. Only the first two topics are covered in the progress report, and these, in only a preliminary way. A final report will probably be made at the next annual meeting. The progress report, with the statistical forms here mentioned, will be published in full in the Journal of the Association. The following excerpts from this report are of wide interest to all water works engineers and operators.

DESCRIPTIVE DATA.

The number of water purification plants in the United States today is very large, and during each year new plants are constructed. Descriptions of the most important plants have been published and may be found scattered through engineering papers, magazines, and official reports. When reading these, it is found that a number of important facts are omitted. So numerous are these omissions that it is impossible from the published descriptions to make compilations of even the most general data which will be adequate. Thinking that it would be of service to those preparing descriptions of filter plants for publication, the following schedule of desirable data has been prepared. Further details would be useful in many cases, but the following items in connection with the study of the operation of a purification plant are always important:

1. City or town, and population at last census. 2. Name of owner. 3. Name of designer. 4. Date when plant was put into service. 5. Total cost, with statement as to what is included. 6. Source of supply. 7. Rated capacity. 8. Method of purification. 9. Total capacity of subsiding or coagulating basins. 10. Total capacity of filtered water basin. 11. What chemicals are used. 12. Where chemicals are applied. 13. How applications of chemicals are made. 14. Number of filter units. 15. Net area of filter surface. 16. Depths of filtering materials. 17. Sizes of filtering materials. 18. For slow filters—arrangements for cleaning sand surface, sand handling, sand washing. 19. For mechanical filters—method of cleaning filter beds. 20. Number of subsiding or coagulating basins. 21. How the rate of filtration is controlled.

STATISTICS OF ANALYSIS.

Blanks illustrating the committee's recommendations regarding the form of report, the tests to be made and the method of expressing the results, are incorporated in the preliminary report. These blanks are seven in number and are referred to as numbered tables. They give: Chemical and microscopical character, turbidity and color, and bacteria in raw water; also the chemical character, the turbidity and color, total bacteria, and number of B. Coli in the water delivered to mains. These tables are arranged to show the analytical data by months and years. The terms used in the tables are all fully explained.

Essential Determinations.—The elimination of unnecessary analytical work is a matter that needs consideration at this time. Laboratory practice at various purification plants has shown that many of the determinations which are ordinarily included in the standard water analysis schedule are here of little or no importance. For example, the determination of nitrogen in the usual forms of free and albuminoid ammonia, nitrites, and nitrates serves no particular purpose in water purification except in special cases. They neither assist the superintendent in the operation of the filter, nor give any adequate idea of the safety of the filtered water. On the other hand, some of the simpler physical tests, such as numerical determinations of turbidity, color, and odor, microscopical examinations and tests for alkalinity, iron, and carbonic acid, have come to be regarded as most valuable; and in special cases various other tests, such as dissolved oxygen and manganese. Bacteriological tests are of course important.

Samples for Analysis.—One step in making an analysis of water has never received half the attention that it deserves, namely, sampling. Of what value is it to use analytical methods of great refinement if the samples themselves are not representative, if the mass of water from which the sample is taken is not homogeneous, or if the water changes in character from one day to another? Samples for chemical analyses are almost never larger than 1 gal., and samples for bacteriological analyses are seldom larger than 100 c. c., while the quantities actually used for the different tests are still smaller. In counting the number of bacteria, the quantity used is less than a thimbleful.

On the other hand, we know that bodies of water are not homogeneous. In a lake or settling basin there are vertical and lateral variations; a river is constantly changing, not only in volume but in the character of the water; filter effluents vary, especially the effluents from mechanical filters where the runs are short and the rates are high. The causes of these variations which affect the results of water analyses through unfair sampling are so numerous that they cannot be studied by themselves, and the only course left is to apply to them the laws of probability, or, in other words, to arrange the data secured in some such way that the importance of the inevitable variations may be indicated and an index of the character of the water examined be obtained.

Thus we see that a question of fundamental importance is that of frequency of collecting samples. The question is: How often must samples be taken to obtain reliable results? As a general proposition it may be stated that the frequency of sampling should depend upon the frequency of change in the character of the water examined. For a water of constant quality, a few samples taken at infrequent intervals may serve to give a fair idea of the water, but if a water be subject to great fluctuations in character, a few samples taken at long intervals might or might not give a fair idea of the water. The reliability of the average result will be determined by the laws of probability. The average result does not tell the whole story, for it eliminates the individual results, and a water supply should be safe and wholesome all of the time.

The frequency of sampling has a limitation, which is controlled by practical and financial considerations. In a small plant the cost of daily analyses would usually be prohibitive, and even weekly analyses might be a burden. It would be recognized, however, that results based on infrequent samples are less valuable than those based on frequent samples; and that irregular sampling gives the most unreliable results. In order to emphasize this point it seems desirable to establish certain grades of control of operation, based upon the character of the records kept, as follows:

First Grade: Water purification plants under first-grade supervision are those where analyses of the filtered water are made one or more times a day, and where engineering and such other data regarding the operation of the plant as are necessary are collected by one or more attendants constantly employed.

Second Grade: Water purification plants under second-grade supervision are those where analyses are made regularly, say once a week or once a month by a trained analyst, and where an attendant constantly on duty makes simple daily tests.

Third Grade: Filter plants under third-grade supervision are those where analyses are made irregularly and infrequently, and where no daily tests are made by the attendant.

This grouping should not be considered as necessarily casting a stigma upon second or third-grade supervision. Some water supplied may not demand first-grade records. In general it may be said that the safer the raw water, the lower may be the grade of analytical supervision. In other words, polluted waters require the purification plant to be operated with a higher factor of safety, and to this end a more careful analytical control is needed. Stored waters are safer than unstored waters, and with them a lower degree of analytical supervision may suffice. A corollary to this would be that small plants which cannot afford high-grade supervision of filters should endeavor to protect the quality of the supply by storage or by incorporating a large factor of safety in the design of the plant.

Efficiency of Operation.—No provision is made in these tables for recording the percentage removal of bacteria, a method that has been much used in the past. While this method has certain advantages, it is so susceptible to misuse that the committee recommends that it be omitted from official reports of filter operation. The reasons why the percentage removal method is unsatisfactory were discussed at some length by the committee. Eight reasons were assigned for abandoning the "percentage removal" method of gauging the efficiency of a filtration plant. The reasons follow:

(1) The bacteria found in the effluent are not all derived from the raw water. A certain and varying number of them represent growths in the lower part of the filter. The bacteria from this source do not vary in number with the number of bacteria in the raw water, but with the rate of filtration, with disturbances occasioned by the collection of air within the filter, and with other factors of operation.

(2) It requires a certain time for the water to pass from the point where the raw water sample is taken to the point where the filtered-water sample is taken. Sometimes this amounts to several hours, and in the case of a water which changes rapidly in quality and character it is necessary to make an allowance for this time if a correct comparison is to be obtained. As a matter of convenience it is common to collect samples of the raw and filtered water at about the same time. Hence there may be an error due to this difference in phase. Whether or not this is of importance depends upon the uniformity in the character of the raw water.

(3) The percentage removal is to a certain extent a function of the number of bacteria in the raw water. It is a well-known fact that the percentage removal of bacteria is relatively high when the raw water contains large numbers of bacteria and relatively low when the raw water contains few bacteria. One reason for this is because the bacteria which develop within the filter are a smaller proportion of the total number of bacteria in the effluent when the number of bacteria passing through the filter is large.

(4) The percentage removed does not necessarily vary with the number of bacteria left in the filtered water. The following is an example:

Day.	Nos. of bacteria. Raw water.	Filter effluent.	Percentage removed.
First	50	3	90.4
Second	1,000	10	99.0
Third	600	20	96.6
Average	555	11	

It will be noted that on the day when the number of bacteria in the effluent was lowest, the efficiency as shown by the percentage removal was also lowest.

(5) The infectiousness of a water does not necessarily vary with the number of bacteria in the water, but in most waters there will doubtless be some connection between the two. A water polluted with surface wash would doubtless increase in infectiousness, and the numbers of bacteria would increase with increased stream flow. On the other hand, a water regularly polluted with sewage, other things being equal, would increase in infectiousness as stream flow decreased, that is, as dilution became less, while the numbers of bacteria under these conditions might decrease. Hence, there seems to be no very definite relation between the infectiousness of a filtered water and the percentage removed of bacteria by a purification plant, which percentage varies according to the numbers of bacteria in the raw water.

(6) If any comparison is to be made between the numbers of bacteria in the raw and filtered waters it would be better to use the "percentage of bacteria remaining," partly because the numbers themselves are smaller and easier to handle, and partly because the figures show to better advantage the variations in the operation of the filter; for example, when two filters, with the same raw water, are operating so as to produce 98 and 99 per cent "removals" of bacteria, respectively, the numbers of bacteria remaining in the effluent of the first will be twice as many as in the effluent of the second. Again, comparing one filter with an efficiency of 99 per cent with another filter with an efficiency of 99.9 per cent, both operating with the same raw water, the difference at first sight seems small, yet the effluent of the first of these filters contains ten times as many bacteria as the effluent of the second. Hence, the method of stating efficiency of a filter plant in terms of "bacterial removal" may be misleading.

(7) A single figure showing the percentage removal of bacteria during a certain time—as, for example, a month—gives no idea of the regularity of operation of the filter during this period. For example, in the illustration above mentioned the percentage of bacteria remaining in the effluent varied on different days from 1.0 per cent to 9.6 per cent of the number of bacteria in the raw water, while the average of the per cents remaining was 3.7 per cent and the median 3.4 per cent.

(8) A common fault of filter specifications is the provision that the plant shall show a certain percentage removal of bacteria when the numbers of bacteria in the raw water are below a certain limit. This specification is inadequate. A good water purification plant is one which produces a satisfactory effluent every day of the year and every hour of the day; and, if efficiency can be expressed on a percentage basis, it should be indicated by the percentage of the time during which the plant produced a good effluent, rather than by the percentage comparison of bacteria in influent and effluent respectively.

Works for the Development of a Ground Water Supply at Springfield, Ill.

The new water supply for Springfield, Ill., is drawn from a set of 22 wells sunk to hardpan through the water-bearing sands and gravel along the Sangamon River. These wells are connected in groups of from three to six wells by means of cast iron pipe to six substations. Each substation has a capacity of approximately 1,500,000 gals. for 24 hours. The location of the various groups of wells was determined by the sinking of test wells. Depth and coarseness of the sand and gravel strata were sought.

The formation in the Sangamon River bottoms consists of a layer of soil varying from 6 to 12 ft. in depth, under which is found a layer of sand and gravel extending to a depth of from 40 to 50 ft. and terminating at hardpan. The water-bearing medium has a porosity of about 30 per cent.

The wells are 12 ins. in diameter and are driven to hardpan, the sand and gravel being sand-pumped from the inside of the casing. After cleaning out the casing a 12-in. Cook strainer from 20 to 60 cut was installed. The cut of the strainer is important, as it is necessary to place the different cuts to suit the degree of fineness of the strata through which the well is driven. The pipe lines connecting the wells to the substations are located from 15 to 20 ft. below the surface.

Each dry well or pumping substation consists of a circular brick structure, 10 ft. in diameter, extending into the ground from 15 to 20 ft. In each substation an 8-in. centrifugal pump, made by the American Well Works Co. of Aurora, Ill., is installed. These pumps are direct connected by means of vertical shafting to electric motors set above the high water level. These pumps maintain about 25 ins. of vacuum while pumping 1,200 to 1,500 gals. per minute. The substations are located along the banks of the river at intervals of from 500 to 1,500 ft.

At the substations the water is not lifted above the ground, but is elevated just high enough to discharge into the flow line through which it is carried by gravity to the large receiving well at the main pumping station. Here it is picked up by the high duty pumps and delivered to the city. These substations are seldom if ever all in use, one or more being kept in operation, according to the demand for water. The idle stations, which are standing as a reserve supply, thus take the place of a storage reservoir.

The water drawn from this well system is not river water, but is merely intercepted in its passage through the gravel from the uplands to the river. When the temperature of the river water is 80° F. that of the well water is 52°. The number of bacteria in the river is 350,000 per cubic centimeter when the figure for the well water is 20.

ACKNOWLEDGMENT.

The foregoing information is derived from the latest annual report of Willis J. Spaulding, Commissioner of Public Property, and from editorial correspondence with Walter Reid, Superintendent of Water Works at Springfield.

The Use of Automobiles in Water Works Service at Worcester, Mass.

The uses to which automobiles are put in the water department at Worcester, Mass., the first cost and the cost of operation were stated and discussed by Mr. George W. Batchelder, water commissioner in that city, in his paper before the latest annual meeting of the New England Water Works Association. The department now has eight automobiles in service and expects to get more. The first one was placed in service in February of 1909. In the following matter, based upon the paper mentioned, the costs given cover in all cases all expenses with the exception of wages paid the chauffeur and the garage men.

Car No. 1, which as stated above was placed in service in February of 1909, was a two-cylinder Buick, equipped with a light express body. It gave good service for five years as a meter and light repair machine and also for emergency uses. It was replaced by a Ford machine. While in service it was used for all meter repair jobs and for many repair and emergency jobs. The trips for meter jobs alone amounted to 4,446 in 1909, 4,040 in 1910, 4,008 in 1911, 6,094 in 1912, and 4,752 in 1913. The average distance covered daily was 45 miles. The machine did the work of three horses. It cost $1,000 new and the operating and maintenance cost was $1,048 per year.

Car No. 2 was bought for $1,750 in 1910. It is a four-cylinder Buick, model 17. It is used to carry engineers and office men, a distance of 9 miles, from the city to Kendall reservoir, where a dam and reservoir have been under construction for several years. The trip which required 1 hour and 20 minutes by team takes only 25 minutes by the machine. The annual cost for this machine is $1,200.

Car No. 3 is a model 21 Buick touring car which was converted, after its purchase, to a service car. It is used at present by the meter and light repair gangs, taking the place of original car No. 1. It, also, makes an average run of 45 miles each day. The annual cost is $1,290.

Car No. 4 was purchased in 1913 for $2,025. It is a four-cylinder Pope-Hartford touring car. The water commissioner and committees of the city council use this machine for inspection trips. Such trips average 35 miles daily and would require two horses and nearly the entire day without the machine. The machine makes it possible for the commissioner to keep office hours in addition to making his inspection trips. The annual cost is $1,280.

Car No. 5 is made up of the running gear of a model 31 Pope-Hartford touring car upon which is mounted a truck body. It cost $1,800 in 1913. The addition of the truck body brought the cost up to $1,925. This machine has hangers attached to the sides of the box so that several lengths of service pipes can be carried in addition to the men and tools required for installing services. The average daily run is 45 miles. This machine has effected a saving of 10 per cent in the cost of installing services. The annual cost is $1,262.

Car No. 6 is a 3-ton Pope-Hartford truck, equipped with extra-size dual rear wheels. It is equivalent to a 4-ton truck. It has very frequently carried 8,800 lbs. of 48-in. pipe. The year before this car was placed in service the department paid teaming contractors the sum of $2,700. At these teaming rates the amount of hauling done by this machine in 1914 would aggregate $8,500. Deducting the sum of $1,352, paid for teaming this year, leaves the truck performing a service worth $7,148. In 1912 seven teams hauled 14 pieces of 48-in. pipe daily. The one auto-truck did the same work in 1913. The cost of horses and drivers was $38.50 per day. The cost of the auto-truck, operator and two helpers, including all expenses, interest, depreciation, etc., was $13.86 per day. This truck hauled, in one day, 101¾ tons of 36-in. pipe a distance of 40 miles.

Car No. 7, which replaced two horses, is a Ford runabout used by the general foreman. The annual cost is $291. Car No. 8 is a 1,500-lb. Velie truck used for general light hauling. It easily handles materials weighing more than a ton. It was so lately placed in service that its cost of operation and maintenance, and the saving it effects over horse-drawn vehicles, have not been determined.

BRIDGES

Design and Construction of the Larz Anderson Bridge Over the Charles River, Cambridge and Boston, Mass.

(Staff Article.)

The Larz Anderson Bridge, which spans the Charles River at North Harvard and Boylston Sts., Cambridge and Boston, is a reinforced concrete structure with earth approaches between concrete retaining walls. The bridge, which has a total length of 440 ft., consists of a 76-ft. 8-in. central span, two 65-ft. 4-in. side spans, and retaining wall approaches having lengths, from inside faces of abutments to outer ends of walls, of 104 ft. each. The clear width of the roadway is 60 ft. and that of each sidewalk, 10 ft. The solid concrete railing with its cast stone coping extends about 3 ft. 4 ins. above the finished sidewalks. The architectural treatment of the approaches and arches is especially striking. By means of courses of brick the railings arches and approaches are paneled, while the concrete surfaces are picked. Under the railing there is a belt course of cast stone, while the arch ring is faced with brick. The bridge replaces an old wooden structure, which had a central lift span consisting of two short leaves operated by hand power. The entire cost of the new structure was donated by Mr. Larz Anderson, for whom it has been named. The bridge is between Harvard University and the Harvard Stadium.

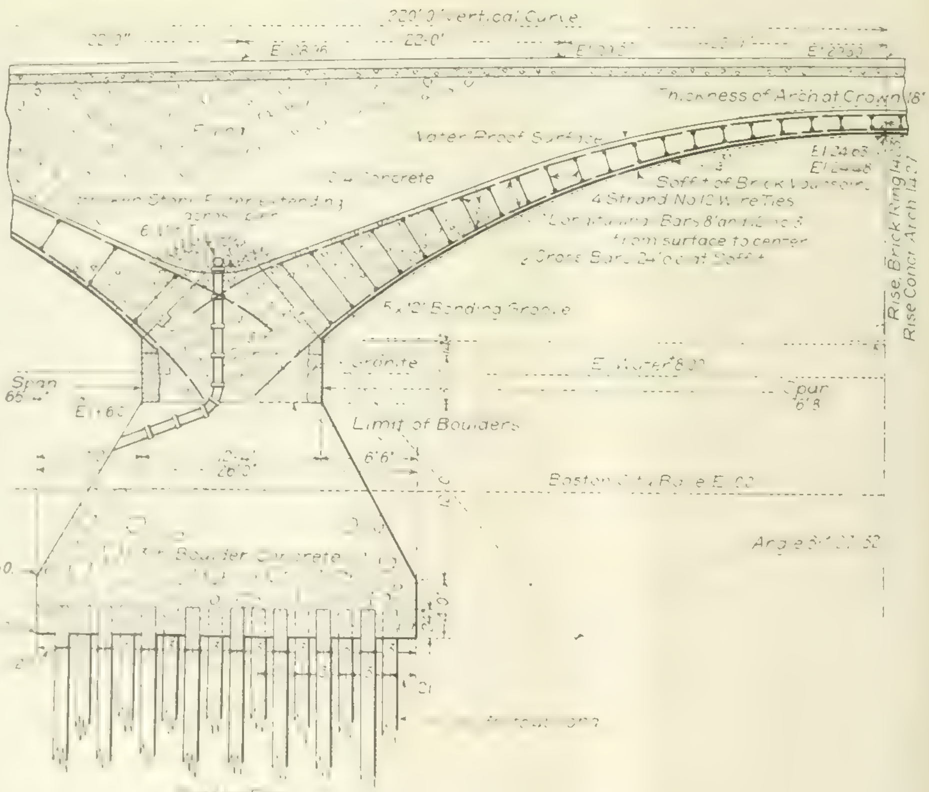


Fig. 2. Cross Section of Pier and Half Longitudinal Section of 76-ft. 8-in. Central Span of Larz Anderson Bridge.

MATERIALS.

Concrete.—The concrete used in the foundations, abutments, spandrels and wing walls consists of 1 part cement, 3 parts sand and 6 parts broken stone passing a 2½-in. ring; that used for the arches and for other reinforced concrete parts consists of 1 part cement, 2 parts sand and 4 parts broken stone passing a 1½-in. ring. It was specified that the concrete was not to be chuted nor [illegible] in a vessel. It was further specified that the concrete was to be spread in 6-in. layers and thoroughly compacted by rammers weighing approximately 25 lbs. each. Boulders were permitted to be deposited in the concrete used for the abutments and piers, it being specified that no boulder should have a volume greater than 4 cu. ft. In placing the boulders it was required that they should not be less than 6 ins. apart nor less than 10 ins. from the face of any form. In connecting concrete already set with new concrete the set surface was required to be cleaned and roughened and mopped with a 1:1 cement mortar.

All exposed concrete surfaces below the water line were required to be faced with mortar at least 1 in. thick, composed of 1 part cement and 2 parts sand and waterproofed with some material approved by the engineer. For the outside surfaces of the spandrel, wing and parapet walls it was specified that a layer of concrete 1½ in. thick, composed of 1 part cement, 1 part sand and 2 parts marble chips or crushed stone chips (ranging from ⅛ to ¾ in. in size) should be placed against the forms and brought up with the concrete of the walls, either by using a sliding board or

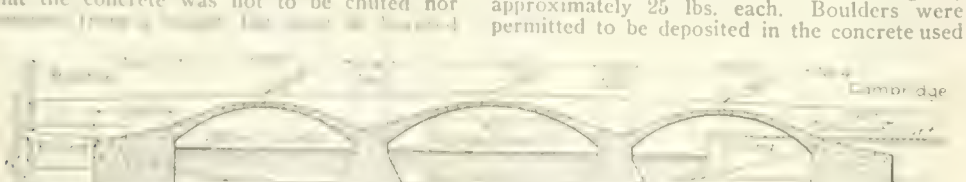


Fig. 1. Longitudinal Section of Larz Anderson Bridge, Cambridge and Boston, Mass., and Profile of Site Showing Character of Subsoil.

by some other approved method. The exposed faces in the spandrels, wing walls and parapet walls were given a picked finish.

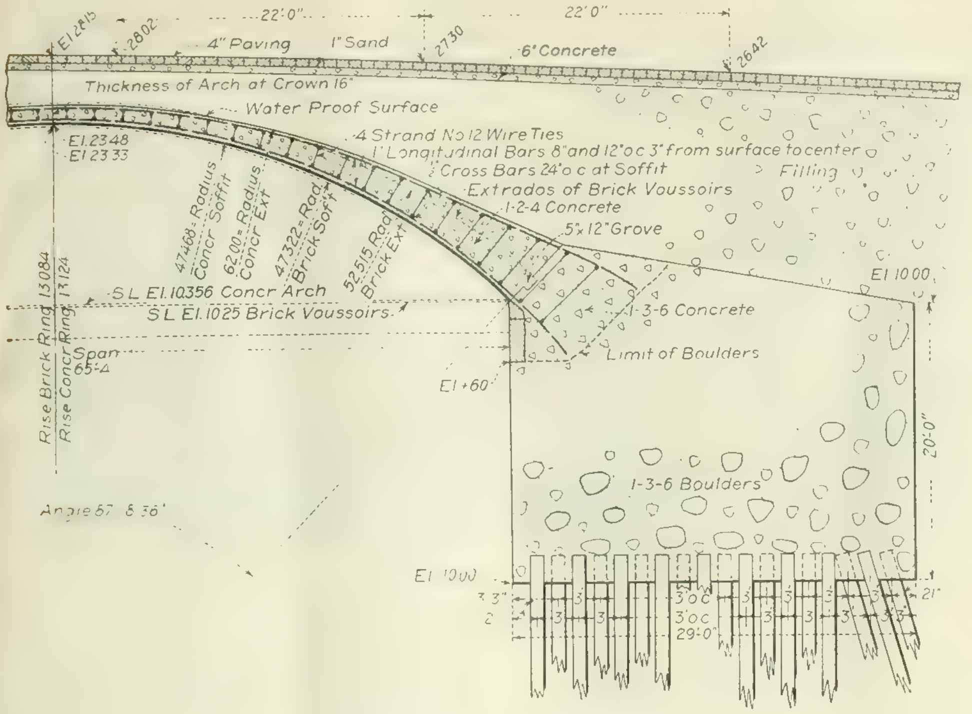


Fig. 3. Cross Section of Abutment and Half Longitudinal Section of 65-ft. 4-in. Side Span of Larz Anderson Bridge.

Granite.—The piers and abutments, from the spring line of the arches to a plane 4 ft. 3 ins. below the spring line, are faced with granite, the exposed surfaces being fine pointed. The stones are laid solid in 1:2 Portland cement mortar, with joints of not more than ½ in. Before the mortar had set the joints were raked out to a depth of 3 ins. and pointed with 1:1 Portland cement mortar.

Cast Concrete.—The cast concrete blocks for the belt courses and copings are composed of 1 part cement and 3 parts crushed stone chips, the chips being of such sizes as to pass through a ⅜-in. mesh screen. Waterproofing material is incorporated in the concrete of the cast stone. After being cast from 40 to 60 days, all exposed surfaces of the stones were dressed with six-cut bush hammers. The cast concrete blocks are laid in 1:2 Portland cement mortar, with joints of not more than ¼ in. Before the mortar had set the joints were raked out for a depth of 3 ins. and pointed with 1:1 Portland cement mortar.

Brick Masonry.—The bricks used for facing the arch rings and for the ornamental courses are up-and-down, hard burned, dark water struck bricks. They are laid in 1:3 Portland cement mortar, except the outer 4-in. faces where "Puzzolan" cement is used. One pail of lime putty was allowed to be added for each barrel of cement.

Foundation Piles.—It was specified that the foundation piles were to be spruce or Norway pine, with a minimum diameter at the butt (where cut off) of 10 ins. and a minimum diameter at the point of 6 ins. The lengths of the piles used for the foundations of the piers and abutments varied from 25 to 50 ft.

Cofferdams.—It was specified that cofferdams should be constructed of 6-in. yellow pine, tongue-and-groove sheet piling. After the concrete work was finished the sheet piling and the piles used in constructing the cofferdams were cut off at the top of the lowest lines of wales and the material below this elevation was left in place.

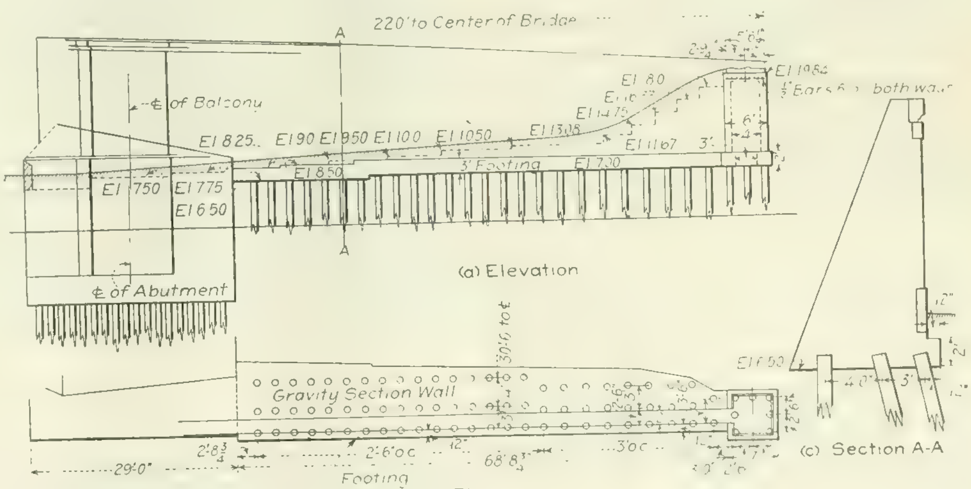


Fig. 5. Elevation, Plan and Cross Section of East Wall of Cambridge Approach of Larz Anderson Bridge.

Subsoil.—A series of borings taken at the sites of the piers and abutments showed that the river bed consisted of a stratum of gravel, or mud and gravel, having a depth of from 6 to 13 ft. This stratum was underlaid with one of blue clay having a thickness of from 14 to 40 ft. Directly under the blue clay was a stratum of hardpan. Figure 1 shows a profile of the site and a section of the arches, piers and abutments. It will be noted that the piles were driven to hardpan.

DESIGN FEATURES.

Piers and Foundations.—The two piers, which are identical, have dimensions at the base of 26 ft. x 75 ft. 6 ins. and a height, from base to spring line, of 20 ft. 3 ins. Figure 2 shows a cross section of one of these piers and gives details of its construction. It will be noted that a 1:3:6 concrete, with boulders imbedded in it is used below elevation + 6.0; above this level the concrete has the same proportion, but does not contain boulders. The sides and ends of the piers, below the spring line and above elevation + 6.0, are faced with granite blocks. Drainage of the roadway is secured by a 6-in. vitrified tile drain, which is placed as shown in Fig. 2. The arch rings are bonded to the piers by means of a continuous groove in the latter, 5 ins. deep by 12 ins. wide.

Each pier is supported on 392 wood piles, these piles being placed in 49 rows of 8 piles each. The piles in the alternate rows are staggered, the spacing of the piles in each row being 3 ft. and that of the rows, 1 ft. 6 ins. The outside lines are set 1 ft. 9 ins. back from the faces of the pier. The piles for pier No. 2 (see Fig. 1) vary in length from 30 to 35 ft., while those for pier No. 3 vary in length from 25 to 30 ft. The tops of the piles project into the concrete piers a distance of 24 ins.

Abutments.—The two abutments, which are identical, have dimensions at the base of 29 ft. x 75 ft. 6 ins. These abutments have a height at the back of 20 ft., and a height at the front, from base to spring line, of 20 ft. 3 ins. Figure 3 shows a cross section of one of these abutments, and give details of its construction. The abutments are also faced with granite from elevation + 6.0 to the spring line of the arches.

The abutment at each end of the bridge is supported on 441 wood piles, these piles being placed in 49 rows of 9 piles each. The piles in the alternate rows are staggered, their spacing being the same as those in the piers. The three lines of piles at the back of the abutment are inclined to resist the horizontal thrust of the arches.

Center Arch.—The center arch has a clear span of 76 ft. 8 ins. and a rise of 14.27 ft. The arch is single-centered, the radius of the intrados being 58.9 ft. and the thickness at the crown, 18 ins. Figure 2 shows a half longitudinal section of the center arch and gives the size and arrangement of the rein-

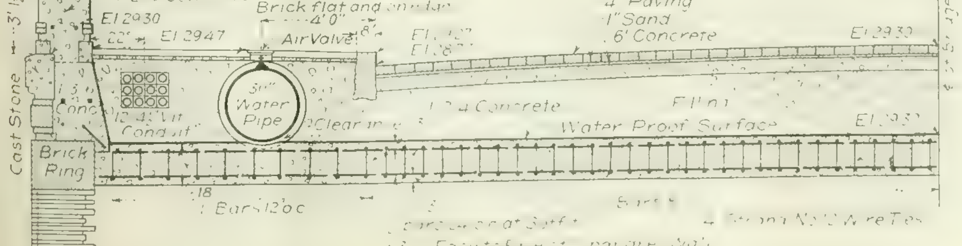


Fig. 4. Half Cross Section at Crown of 76-ft. 8-in. Central Span of Larz Anderson Bridge.

forcement. Figure 4 shows a half cross section of the arch at the crown, and gives details of the arch ring and railing. The longitudinal reinforcing bars at both the intrados and extrados are 1 in. square and are spaced either 8 ins. or 12 ins. on centers, as shown in Fig. 4. The cross reinforcing bars are spaced 24 ins. on centers. The bars in the intrados and extrados are tied together at the intersections of the longitudinal and cross reinforcing bars with four strands of No. 12 wire.

The solid concrete soffit is faced with a brick ring as shown in Figs. 4 and 13. The details of the concrete railing and its cast stone and brick courses are also shown in Fig. 4.

Side Arches.—The two side arches each have a clear span of 65 ft. 4 ins. and a rise of 13.12 ft. These arches are single-centered, the radius of the intrados being 47.468 ft. and the thickness of the arch at the crown, 16 ins. Figure 3 shows a half longitudinal section of the concrete arch ring and gives the sizes and arrangement of the reinforcement. The railing and other details not shown for these arches are similar to those shown for the center span.

Approach at Cambridge End.—The approach at the Cambridge end of the bridge consists of an earth fill between concrete retaining walls. At both approaches the roadway has the same clear width as for the arches, but the sidewalk width increases to 15 ft. 6 ins., making the width of the approaches, between inside faces of railings, 71 ft. instead of 60 ft. as for the arches. This approach extends 75 ft. beyond the end of the abutment. The east retaining wall is of the gravity type and rests on wood pile foundations. Figure 5 shows an elevation, a plan and a cross section of the east wall of this approach.

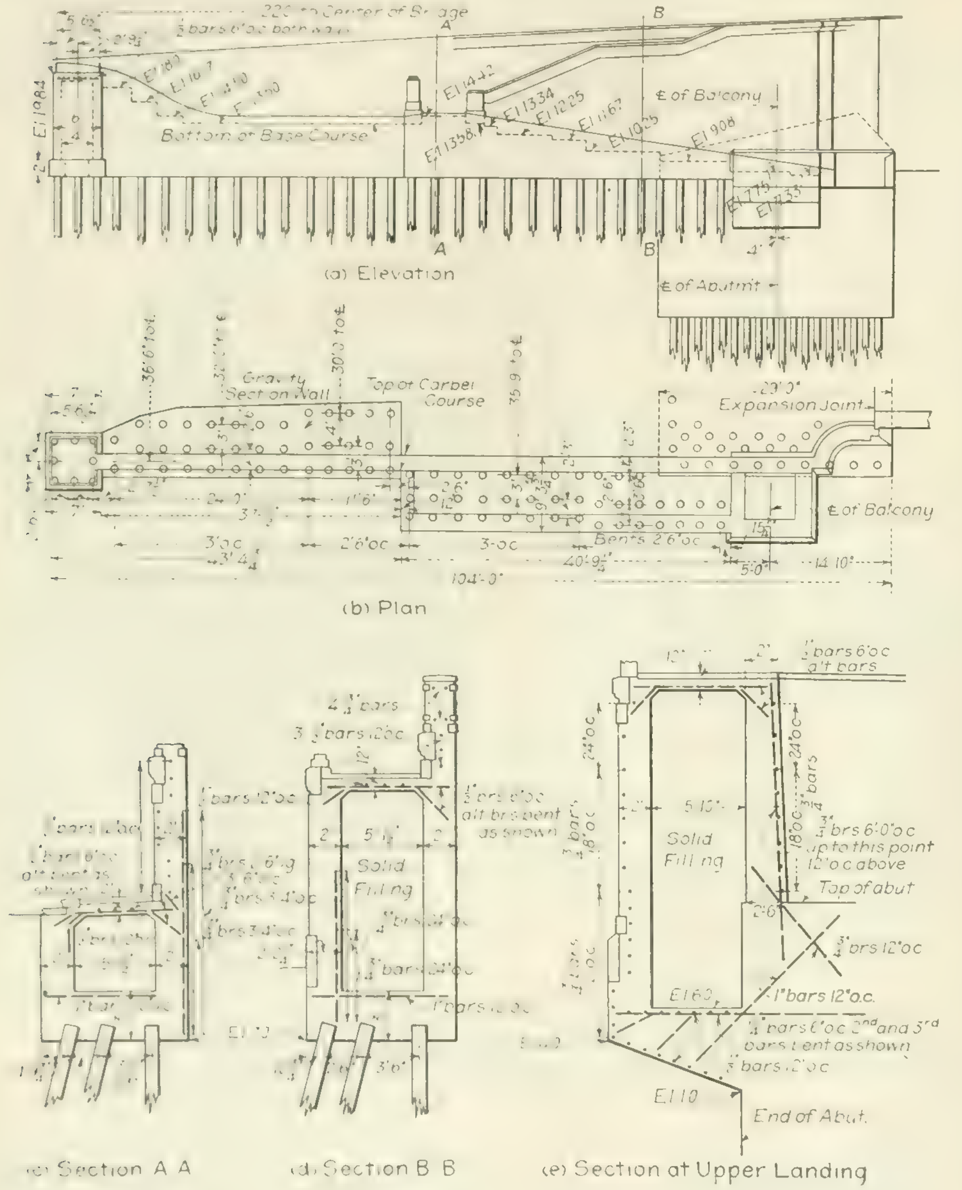


Fig. 6. Elevation, Plan and Cross Sections of West Wall and Stairway of Cambridge Approach of Larz Anderson Bridge.

The west wall of the approach at the Cambridge end of the bridge is partly of the gravity type and partly of the reinforced slab type, the change in construction being due to a sidewalk and a flight of stairs along the wall. This wall also rests on pile foundations. Figure 6 shows an elevation, a plan and three cross sections of the wall and stairs. It will be noted by referring to Fig. 6 (e) that, at the abutment end, the stairway is bracketed from the abutment to form a balcony. The remainder of the stair construction and of the retaining wall is indicated in Figs. 6 (c) and (d).

Spandrel Walls.—The spandrel walls are of the gravity type. They have a thickness at the sidewalk level of 2 ft. and a thickness at the arch ring of 2 ft. 7 ins. for the walls above the middle third of the arch and a thickness of one-half the height for the remaining portions. The spandrel walls rest partly on the brick facing ring of the arch and partly on the concrete arch. The upper surface of the brick ring and the concrete arch is waterproofed, and the spandrel wall is keyed into the concrete arch by means of a projection which fits into a 6x12-in. groove in the arch.

Approach at Boston End.—The east and west retaining walls of the approach at the Boston end of the bridge have the same length, and the construction is similar to that shown in Fig. 5, except that these walls do not have pile foundations and the footings are stepped until the bottom of the footing at the end of the approach is at elevation + 13.0 (instead of + 7.0). The tops of the walls at both ends of this approach have the same elevations as for the approach at the Cambridge end of the bridge.

CONSTRUCTION FEATURES.

Before construction was begun on the new bridge the contractors built a temporary wooden pile trestle across the Charles River. This foot bridge had a clear width of 8 ft. and a length of about 360 ft. It was torn down and removed at the completion of the permanent structure.

The cofferdams for the two piers consisted of a single line of 6-in. yellow pine, tongue-and-groove sheeting, braced by means of piles driven outside of the cofferdam and by a heavy system of timber bracing within the cofferdam. Figure 7 shows the construction features of one of the cofferdams. This view also shows the pumping facilities, the floating piledriver and the floating derrick used in excavating the cofferdam and in placing the concrete in the piers. The boulders shown suspended at the left is to be imbedded in the pier concrete. The temporary foot bridge is also shown in the background.

Figure 8 shows a view of the mixing plant

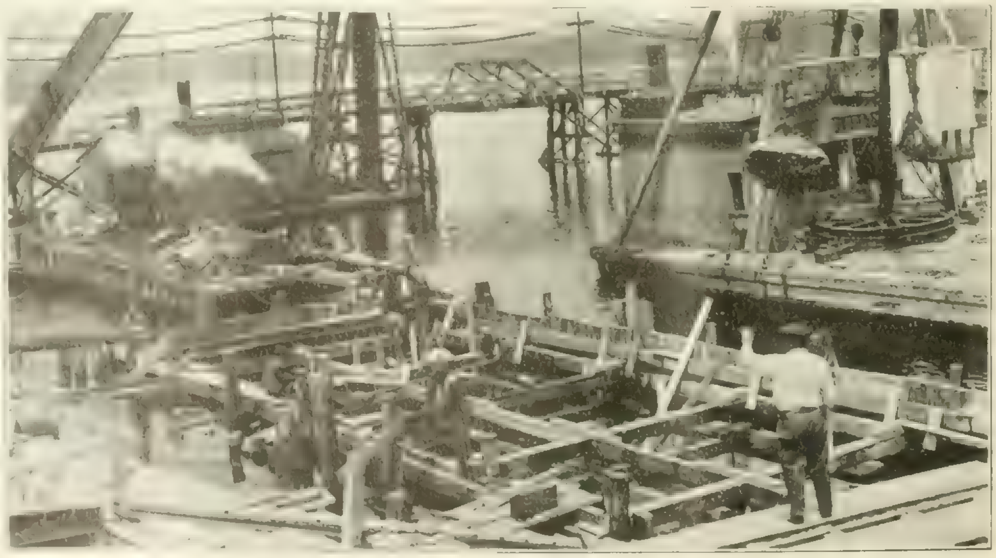


Fig. 7. View of Cofferdam for Pier and Contractor's Equipment Used During Placing of Concrete—Larz Anderson Bridge.

Fig. 8. View of Concrete Mixing Plant and Equipment Used in Mixing and Placing Concrete—Larz Anderson Bridge.

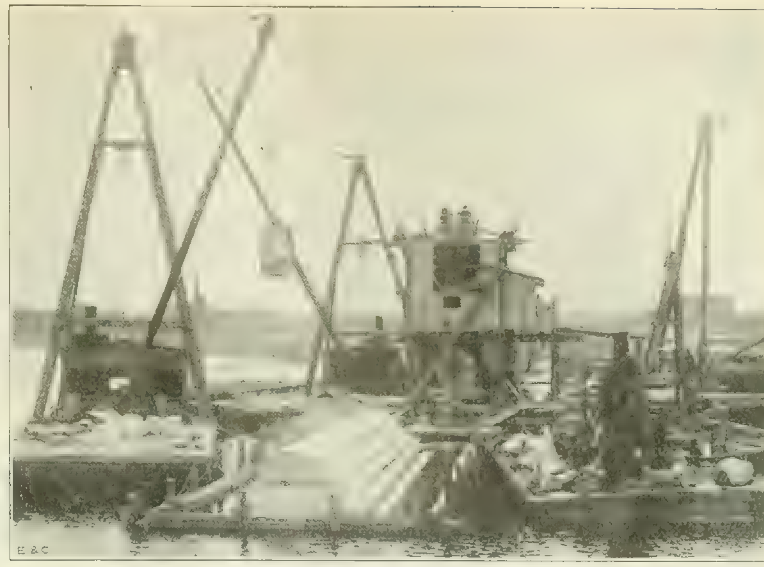


Fig. 9. View of Completed Pier of Larz Anderson Bridge and of Concrete Mixing and Placing Equipment.

and of the industrial track serving it. This view shows clearly the arrangement of the material bins. It also shows the derricks and the bucket used in placing the concrete, this bucket being shown under the spout leading from the concrete mixer.

Figure 9 shows another view of the concrete mixing and placing equipment. This view also shows a pier completed and ready for its adjacent arch spans. The granite belt course at the water line is shown in place.

In constructing the arches steel centering was used, the centering for an arch consisting of 13 three-hinged steel arches. In placing forms for the concrete arch ring planks were bolted to the upper chords of the steel arch centers, and the forms for the intrados were spiked to these planks. In erecting the steel centering a heavy timber bent was erected under the center hinge, which formed a working platform and gave additional support to the centering. This was advisable, as the crown was heavily loaded during the placing of the concrete near the spring line.

After the form for the arch ring was in place the brick rings, which face the concrete arches, were constructed. The contractors debated for some time as to the best procedure to follow in laying these brick arch rings to insure their being held firmly in place. It was finally decided first to lay the bricks in wooden forms in the shape of voussoirs of the depth and width of the completed ring and seven bricks wide. As soon as the mortar had hardened these voussoirs were taken out of the forms, and the projecting mortar was dressed from the faces so that the voussoirs would fit closely together when laid to form the arch ring. Figure 10 shows the manner of building up the voussoirs. The separate sections of the brick arch were then laid in the same manner as the voussoirs of a stone arch.

Fig. 10. View Showing Manner of Constructing Brick Voussoirs for Facing Ring of Arches of Larz Anderson Bridge.

The concrete arch rings were then poured, after the reinforcement had been placed and the bars wired together. Figure 11 shows an arch ready for concreting, the reinforcement and the brick rings being in place. The crown of the arch was loaded with blocks having a combined weight of about 25 tons, as shown in the view. Figure 12 shows one arch ring completed, the brick facing rings of another in place, and the men at work placing the reinforcement. This view also shows the position of various units of the contractor's equipment while the reinforcement was being placed.

Before the steel centers were removed the forms for the spandrel walls were constructed and the walls concreted up to the brick belt course shown directly under the cast stone belt course in Fig. 13. The outside forms were then removed, the brick and cast stone courses were set in place, and the concreting continued up to the top of the sloping rear face of the spandrel wall. The concrete railing, with its brick paneling and cast stone coping, was then constructed. All concrete surfaces were given a "picked" finish, the cast stone surfaces being left smooth to contrast with the poured concrete work.

Figure 13 shows the structure practically completed, looking toward the Cambridge end. This view shows the effective architectural treatment of the bridge, which was finished in the latter part of 1913.

Fig. 11. View During Concreting of Arch Ring of Larz Anderson Bridge—Load at Crown Is About 25 Tons.

COST OF BRIDGE.

The contract prices for various parts of this work were as follows:

Building 360-ft. temporary wooden trestle, 8 ft. clear width, and removing same..	$ 3,000
Constructing three reinforced concrete arch spans, including excavation, piers, abutments, cofferdams, etc., and removal of old bridge....	131,000
Constructing approach at Boston end....	7,500
Constructing approach at Cambridge end	9,000
Building stairway at Cambridge end.....	5,000
Furnishing and placing filling material, $0.65 per cu. yd.	
Additional concrete required in wing walls, if any, $7.50 per cu. yd.	
Any additional work which may be required, cost as determined by the engineer plus 15 per cent.	

Fig. 12. View Showing Men Placing Reinforcement in Arch at Cambridge Side and Completed Arch Ring at Boston Side—Larz Anderson Bridge.

PERSONNEL.

The bridge was designed under the direction of the Metropolitan Park Commission; Wheelwright, Haven & Hoyt, architects; John R. Rablin, engineer. It was constructed by the Holbrook Cabot Rollins Corporation, of Boston.

Some Data on the Use of Slings in Handling Loads.

In our issue of April 22, 1914, we gave data on the use and strength of hooks, cables, chains and other equipment for handling loads. The fact that erection foremen and even engineers in charge of handling loads do not always appreciate the limitations of hoisting equipment has resulted in many preventable accidents. As a rule, too little attention is given to slings, even when careful consideration is given to the main cables and other hoisting apparatus. The following data on the use of slings in handling loads are given to supplement those contained in our previous issue. They were abstracted from information compiled by the engineering and inspection division of The Travelers Insurance Co. of Hartford, Conn.:

The manner of attaching the slings to the load and to the hook of the hoisting cable is of great importance; and this part of the work should be entrusted only to experienced men. Loads may often be safely hoisted by the use of a single sling, but in other cases two or three slings are required—the number to be [illegible] the load but also upon its shape.

Wire cables, chains, and manila ropes are all used for slings, their relative strengths, for similar diameters, varying in the order in which they are here named. For many reasons aside from its strength a wire cable sling is [illegible] of any kind. Ordinarily, deterioration is easily detected in wire cables, as it is commonly indicated by broken strands that are readily discoverable by an experienced and qualified inspector. Chains, on the other hand, may sometimes be used almost up to the moment of failure with no manifest external evidence of weakness other than the existence of a few seemingly unimportant bruises, although a careful microscopic examination will often disclose a multitude of small cracks, showing that the metal has become "fatigued" by the heavy loads which have been lifted.

Wire cable slings, on account of their pliability, are often bent at very sharp angles, not only while being adjusted to their loads, but also when they are under stress. Sharp bends of this kind should always be carefully avoided, not only because they are immediately dangerous, but also because, when taken in connection with the twisting and untwisting to which the strands of the cable are subjected while in use, they cause rapid deterioration of the sling. The damage may be more marked in the inner wires than in those at the surface; and as the weakened condition of the inner wires is likely to pass unnoticed under ordinary inspection, unless the outer wires show serious wear or poor condition, the sling is often continued in use long after it should be discarded. In making a thorough inspection of a wire cable sling it is advisable to clamp the sling in two places and partially untwist the intervening section so that the interior wires can be examined.

In placing a sling about a load it is important to see that the turns of the sling do not lie one over another, because an excessive stress is likely to be thrown upon one of them unless careful attention is given to this point. A sling composed of a single length of wire cable, with spliced eyes, should never be used for hoisting a heavy load by hooking into only one of the eyes; because if this is done there will be a tendency for the load to revolve, thus unwinding the cable and permitting the splice to slip. On the other hand, when using a doubled sling with both ends engaged in the hoisting hook it is important to adjust the sling so as to equalize the stress throughout the sling as nearly as possible and to prevent it from becoming unduly localized or concentrated in certain parts.

When placing chain slings about loads, carefully avoid twisting the chains, because if they are twisted an excessive load may be thrown upon some of the links.

EFFECT OF OBLIQUITY OF THE PARTS OF A SLING.

The stresses that are thrown upon slings and ropes vary considerably with conditions, and they are often influenced to a marked degree by circumstances which the casual observer might consider unimportant. In particular, the inclination or obliquity of the sling, in those parts which lie between the supporting hook and the points at which the sling first touches the load, must be carefully considered, as it is a highly important feature in connection with safety. In order to fix the attention upon the effect the obliquity of the sling has upon the intensity of the stress, we shall assume that the sling is perfectly flexible in all the cases shown in Fig. 1, and also that the load is symmetrical in shape and symmetrically supported and that the branches of the sling (between the hook and the load) are equal in length and equally inclined. For simplicity we shall also assume that the total load that is to be supported is 2,000 lbs. in each case, except that shown in Fig. 1 (f).

Fig. 13. View of Larz Anderson Bridge Practically Completed—Note Effective Architectural Treatment.

Under these conditions, if the ends of the sling are exactly vertical (see Fig. 1 a) the stress on each one of them will evidently be 1,000 lbs. If the ends are inclined, however, as shown in Figs. 1 (b), (c), (d) and (e) the stress upon each of them will be *greater* than 1,000 lbs. by the amounts shown, and it will increase as the obliquity of the ends increases.

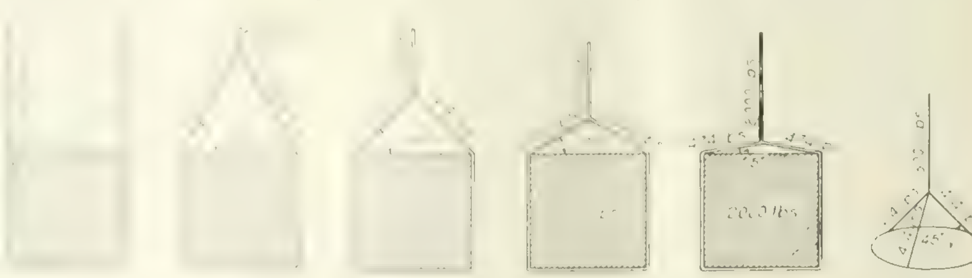


Fig. 1. Diagrams Showing Variation of Stress in Slings Due to Changes in Inclination of the Parts.

These drawings show very clearly the importance of giving careful attention to the inclination of the free ends of the sling. Men engaged in hoisting too often take it for granted that the tension on a sling is everywhere the same, and if the sling is strong enough to support the load in safety when the ends are vertical, they assume that it is safe to hook it around the load in any way whatever, which is far from true. The sling should always be long enough to allow the ends to be at least as steep as shown in Fig. 1 (c), i. e., the ends should never make an angle of less than 45° with the horizontal.

In lifting large plates of steel or heavy castings of a circular shape three-part slings are often used (see Fig. 1, f). In such a case the three parts of the sling should be substantially equal in length, and the points at which the sling grips the load should be selected so that the stem will be the same on each of the three branches, so far as this can be judged by the man in charge of the operation. If the load consists of a circular ring or plate of uniform section, this would correspond to making the points of contact equidistant around the edge; but if some part of the plate or casting has a heavy projection upon it, then two of the branches of the sling should be put nearer to this heavy region.

The steepness of the ends of three-part slings should conform with the same principles that have already been outlined above, in connection with slings of the two-part type.

Table I shows the manner in which the stress changes in two-part and three-part slings, for various inclinations of the ends to the horizontal.

PRECAUTIONS NECESSARY IN PLACING SLINGS.

When the load to be lifted has sharp corners or edges, as is often the case with castings and with structural steel and other similar objects, pads or wooden protective pieces should be applied at these corners, to prevent the slings from being abraded or otherwise damaged where they come in contact with the load. This is especially important when the slings consist of wire cable or fiber rope, although it should also be done even when they are made of chains.

Figure 2 (a) illustrates the importance of protecting sharp corners where slings run over them. The total load is here assumed to be 2,000 lbs., and as the ends of the two-part sling make an angle of 30° with the horizontal, each of these ends is subjected to a tension of 2,000 lbs. This tension is exerted partly to hold up the load, but it also tends to draw the sling horizontally against the load, at the points indicated by the arrows; and if the sling is pliable and is placed about the load symmetrically, as shown in the illustration, it will press against the load, under

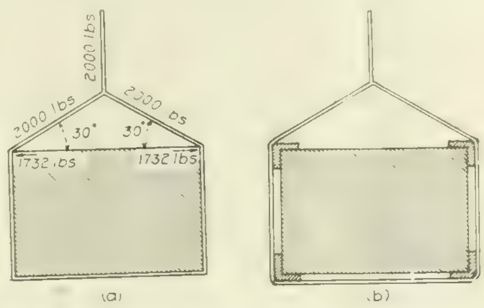


Fig. 2. Diagrams Illustrating Pressure of Sling Against Sharp-Cornered Load and Protection for Same.

the assumed conditions, with a horizontal force of 1,732 lbs. at each of the upper corners. This pressure may damage either the sling or the load, or both of them; and it is to prevent damage from this source, as well as from the direct weight, that the pads are recommended. Wooden corner-pieces are often provided, as shown in Fig. 2 (b), for use in hoisting loads with sharp angles. If pads of burlap or other soft material are used, they should be thick and heavy enough to sustain the pressure and to distribute it over a considerable area, instead of allowing the pressure to be concentrated directly at the edges of the object to be lifted.

CARE OF SLINGS.

All slings should be kept in good condition, and special attention should be paid to those that are used out-of-doors. Chain slings should be oiled to prevent rusting, and slings made of wire rope should be treated with oil, or preferably with special dressings prepared for

TABLE I.—EFFECT OF THE OBLIQUITY OF THE PARTS OF A SLING.

Angle between sling and horizontal.	Stress, in lbs., on sling, per lb. of total load — Two-part sling.	Three-part sling.
5°	5.737	3.825
10°	2.879	1.920
15°	1.932	1.288
20°	1.462	0.975
45°	0.7071	0.4714
60°	0.5774	0.3849
90°	0.5000	0.3333

this purpose, to prevent them from rusting. The inner wires often become corroded through exposure to the weather, even when the outer ones remain in comparatively good condition.

When not in use chains should be stored in a place specially provided for them, and locked up. They should be in charge of an experienced man, who should be held responsible for their condition. The man charged with the care of the slings should give them out as they are needed, and always with due regard to the use to which they are to be put. In this way it is possible to guard, effectively, against the workmen using slings of inadequate strength. All slings should be promptly returned to the official custodian when they are no longer needed for the work for which they were given out. As an additional precaution every sling should be provided with a small metal identification tag, which should be firmly fastened to it. The tag should give the maximum stress that the sling can safely withstand in use, and in the case of a chain sling it should also give the date of the last annealing.

Small cards giving the weights of the materials handled, per cubic foot, will often assist the workmen in estimating the weights of the loads, and will thereby be conducive to safety.

SEWERAGE

Experience in Germany with Combined Sedimentation and Digestion Tanks and Separate Sludge Digestion Tanks.

In the present article, which is a translation from the German, attention is called to some of the operating difficulties encountered by German engineers in the use of combined sedimentation and digestion tanks and of separate sludge digestion tanks. Among the former (two-story type) the Travis and Imhoff tanks are examples. The Foerster and Neustadter are examples of the latter. While a number of Imhoff tanks have been installed in this country, the operating experience with this type of tank is here still somewhat limited. As is well known this tank originated in Germany and it has been well tried out in that country. Some of its shortcomings and abuses have been discovered and are now being rectified. Many interesting points are brought out in the following article, notably that Imhoff tanks for all localities should not be built by one design, that tanks of this type operate satisfactorily only on sewage with a distinct alkaline reaction, that the storage of sludge for from four to six months is improper operation, etc.

As stated above the following article is a translation from communications of the Royal National Institute for Water Hygiene in Berlin-Dahlen, volume 18, 1914, article 3, p. 48. It was written by Professors Dr. K. Thum and Dr. Eng. C. Reichle, Division Superintendents of the Royal National Institute for Water Hygiene. The formal name of the communication from which we quote is: Preliminary Report on Facts About and Experiences with Imhoff Tanks and Kindred Sewage Clarification Processes.

Imhoff tank treatment and kindred processes in which special value is given to the fact that the sewage is kept fresh while more or less undissolved matter is deposited, have lately come into considerable use; sometimes by themselves, sometimes in connection with separate sludge digesting chambers. The advantages of such a process could clearly be seen, the simple manner of separating the sludge from the sewage, the avoidance of septicization in the sedimentation chamber, the delivery of a well digested, inoffensive and easily drained sludge in connection with a simple appearing operation seemed to make possible the construction of a mechanical plant that was practically odorless. The effluent being fresh even if still putrescible, was less liable than septic effluent to cause a nuisance, when discharged into rivers, and if subjected to further treatment in biological plants, would not give rise to bad odors, as it was free from evil smelling gases. The only disadvantage was the high first cost which was offset by the above described advantages, and especially on account of the simple operation of the plants.

The great vogue of these fresh water clarification plants with separate sludge treatment for the handling of domestic and municipal sewage, the building of them after one pattern without paying sufficient attention to the character of the sewage and sludge, and the supposition that this new process was nearly automatic, was bound to cause difficulties, which found expression in numerous inquires to the "National Institute for Water Hygiene." The correspondence and consultations in regard to this matter were considerable, and in view of the importance of the case it was decided to issue, in the communications of this institute, an exhaustive treatise on these questions, combined with the results of our own experiments. But to comply with the momentary need, we will consider some of the more important phases at once. We refer to our "Handbook on Hygiene" and state that some matters will require further investigation, but we have no doubt that the following conclusions will be of use in many cases.

Three points are discussed in the present preliminary communication, namely: (1) The formation of acid sludge in the sludge digesting chamber; (2) The formation of a floating cover; and (3) The frothing and spitting of a fresh water clarification plant.

THE FORMATION OF ACID SLUDGE IN THE SLUDGE DIGESTING CHAMBER.

As is well known, good Emscher tank sludge has a deep black color given to it by ferrous sulphide and will naturally re-act alkaline, as it is impossible for ferrous sulphide to exist with free organic acids such as occur in sludge. It has a peculiar, not disagreeable odor, somewhat like sealing wax, and is easily drained. The water contained in this sludge, which can be easily filtered off through filter paper, is more or less clear and colorless, does not smell putrid, contains ammonia and free carbonic acid and reacts alkaline, as all normal, clarified or raw sewage does.

But occasionally the sludge from Emscher and similar clarification plants is of an altogether different quality, especially if it has been left in the sludge chamber for a few months. It is not black, but yellow, like milk-faeces, grayish yellow or gray, has an evil smell, is very difficult to drain, has a distinct acid reaction, containing none or very little ferrous sulphide. The water contained in such a sludge has some ammonia, but has like the sludge a more or less but always distinct acid reaction, and finally the water as well as the sludge in the whole chamber will show this acid reaction.

Imhoff tank sludge drawn from tanks that are in the process of ripening has, like the above described acid sludge, a gray to gray-black color, but the reaction of this sludge and the water in it is alkaline. The water is easily filtered off, the nearer ripe the tank is the easier the sludge can be drained.

The appearance of from black to yellow, or alkaline to acid sludge, is nothing new or surprising. The human faeces, e. g., which form the largest part of the sludge, react as is well known, sometimes acid, sometimes alkaline, according to their composition. The color is either yellow, brown, or sometimes even black. In case of a largely vegetable diet, the faeces will have a strong acid reaction on account of the brisk fermentation of the carbon hydrates. Increased decay of albumen, consequently a larger formation of ammonia, will cause a stronger alkaline reaction. On a mixed diet they will be either neutral, slightly acid or slightly alkaline.

Acidulation and decay, i. e., fermentation and decay, caused by the same organic substance, is found also in so-called agricultural sewage, which is rich in carbohydrates. Starch factory sewage which is put into starch sludge basins, reacts acid and becomes more acid from day to day, and the starch sludge also shows an acid reaction. If all the acid present is neutralized by a base (ammonia, soda, lime, etc.) and an overdose is given (Zahn, test in regard to the possibility of purifying starch factory sewage by the biological process), typical putrification (with sulphureted hydrogen ferrous sulphide) as well known from municipal sewage, will take the place of fermentation.

Fermentation and putrification phenomena are found side by side in domestic and municipal sewage. The floating scum cover of the septic tank, which is the result of an accumulation of carbohydrates, is of a gray brown color and re-acts distinctly acid, because the sewage which has the abilty to neutralize these acids cannot come in contact with the scum cover, which often reaches a considerable thickness and the nitrogen, i. e., ammonia in the scum is not sufficient to neutralize the acids. The remainder of the tank reacts alkaline, the sludge is also alkaline, is colored black by the ferrous sulphide, because if any acid should form, it would be immediately neutralized by the free ammonia.

With the preponderance of carbon hydrates and a small amount of nitrogen combinations, such as ammonia and other bases, acid sludge must be expected with every method of separate sludge digestion and later on, as the acid reaction progresses also acid water. A preponderance of nitrogen combinations or other acid neutralizing matter assures the formation of alkaline sludge and alkaline water in the sludge digesting chamber. From sludge and water, which have only small acid neutralizing ability, an acid sludge is to be expected. Great acid neutralizing ability will guarantee good Emscher tank sludge having an alkaline reaction.

Fresh sewage sludge has, as a rule, a very small acid neutralizing power, the exception being a sludge rich in lime. Sludge, as is well known, is poor in nitrogen, and so must be poor in ammonia, which is formed from the nitrogen and which as a rule represents the acid neutralizing ability of the sludge. Sludge a few days old, drawn from a fresh water clarification plant, on testing often did not react alkaline, but distinctly acid. Young sludge from the bottom of a sludge digesting cham- [illegible] acid. Thick scum covers, strong suspended layers and sludge digesting chambers more or less filled with sludge are always prone to show an acid reaction. Normal municipal sewage compared to the sludge possesses high acid neutralizing ability. The proportion is about the same as in faeces and urine. The formation of acid sludge can, as a rule, only be prevented if the sludge is not stored by itself, but is brought into intimate contact with a sufficient amount of sewage or other acid neutralizing bodies, to make all acids harmless that are formed. The important points to be considered in the obtaining of a good sludge are the sewage, the thorough mixing of sludge and sewage and the right proportion between sludge and sewage.

We have already pointed out the necessity of the right mixture of sludge and sewage to obtain a good sludge, in a report, which we made in regard to the efficiency of the Kremer Apparatus in 1908. This report was the first short communication of institute tests, made on our west end test plant with a modified Imhoff tank, a so-called Kremer-Imhoff tank. The agitator, mentioned in this report, was ready for operation in the summer of 1909. In the meantime numerous troubles occurred, bad sludge, spitting and frothing and the formation of scum. The installation of the agitator and its daily use brought an alkaline and better sludge. We tried to overcome the frothing and the scum formation by a timely withdrawal of sludge, but in spite of this sometimes the whole contents of the sludge digesting chamber threatened to boil over. After long consideration it was decided to drain off the water in the sludge digesting chamber. This was done in December, 1910. The sludge digesting chamber was refilled from the top with clean water from the municipal water system and the sedimentation chamber refilled with sewage; the usual operation was then resumed. A week later we had for the first time a fully satisfactory typical Imhoff tank sludge and it continued to be of the same good quality, until the plant was discontinued during the summer of 1913. The operation of the sludge digesting chamber, to which no more clean water was added, was as follows:

The agitator was run for a short time every day, and on every Saturday, an exact predetermined amount of sludge was withdrawn from the sludge digestion chamber. Care was taken that after the withdrawal of the sludge, the proportion of sludge and water in the sludge digesting chamber remained in the raito of 1:2. Frothing and the formation of injurious scum did not occur again.

Through the mixing of the clean water with the sludge, the continued stirring and the regular withdrawal of the sludge, by which an amount of fresh sewage, equal to the amount of the withdrawn sludge, was introduced into the sludge digesting chamber, the clean supernatant water soon became septic, having a high acid neutralizing ability, and this produced good results. Continued clean water treatment would remove the injurious products of fermentation, which as Dunbar (Dunbar-Principle of the sewage cleaning question, 2d Edition, Muncheon and Berlin, 1912, page 198) has proved, sometimes lead to dead tanks; at the same time the useful ammonia, which prevents the acidulation of sludge, would be washed out and a later acidulation of the sludge would be sure to follow. A limited addition of clean water may be of some use, especially if the water has a high acid neutralizing power, i. e., considerable temporary hardness. Continued addition of clean water to the sludge, especially if this cannot be withdrawn immediately, does not seem to be advisable, unless by an addition of suitable bases, the acid neutralizing qualities of the water are increased.

Sludge, in Nordhausen, occasionally treated with clean water, reacted alkaline; the supernatant sewage also reacted alkaline, and the sludge was beginning to be satisfactory. But the sludge of another sludge digesting chamber, which was continuously treated with clean water from below, was acid; all the supernatant water was also acid, the sludge was poor. A good sludge could only be expected if the acids are neutralized by the addition of lime, chalk, soda, ammonia or by sludge containing sewage, or raw sewage, the whole contents being thoroughly mixed. Not so much clean water should be used after this and alkalies should be added as necessary.

Sludge digesting chambers which show acid decomposition or fermentation instead of putrefactive decomposition can and should be treated in the same manner as acid bottoms, acid compost heaps, etc., i. e., the acid must be neutralized by a base. The main conditions with such a treatment are, to obtain a good moisture of the sludge with the added material, give an over-dose and make sure of a sufficient supply of acid neutralizing water, which can be obtained in Imhoff tank installation by the regular withdrawal of sludge.

The proceedings will vary for different cases and attention must always be given to local conditions. If the sludge reacts acid and the supernatant sewage—in case of sour scum covers—the water below it, alkaline, one or several thorough mixings of the sludge with the sewage and regular withdrawal of the sludge (see above) may bring about the desired results. If both the sludge and the supernatant water in sludge digesting tank show an acid reaction, it is best to drain the supernatant acid water, and some acid neutralizing agent, mix it thoroughly with the sludge and fill again with sewage. The sludge should be drawn off regularly after that and acid neutralizing agents added if necessary. If there is much yellow sludge present, it would be best to wash the sludge with sewage, while thoroughly stirring it, either by means of an agitator or by compressed air.

Great attention should be given to the thorough mixing of the sludge with the bases, as it is absolutely necessary if good results are to be expected. Especial care ought to be taken if bases are used that are only partially soluble in water, such as lime, etc. If the lime is added before sedimentation, a very good mixture of the sludge with the lime is obtained and the sludge will have acid neutralizing properties. With such treatment the supernated sewage also possesses more than ordinary acid neutralizing ability and the proportion of 1:2 between sludge and water need not be so strictly adhered to. (It is always desirable to keep the sludge down to these proportions, even with a ripe chamber, as there exists always the possibility of an unexpected precipitation of sludge especially on the combined system). Due regard must be given to the fact that sewage already rich in lime exists in some places especially where the system receives the sewage from coke ovens.

During a test, which was made with sludge containing lime, we learned how desirable such an addition to the sewage was, giving the sludge an acid neutralizing property without cost. We were testing a method of separate sludge digestion in a lignite installation near Berlin. Although the sludge was treated daily with a large amount of fresh water, we never obtained an acid sludge as we did in Nordhausen, but always good alkaline sludge, and the water in the sludge digestion room also reacted alkaline. The fresh sludge was obtained not by precipitation with sulphate of alumina only, as usual, but with an addition of lime.

The principal conditions, which lead to the formation of good sludge, are in our opinon quite clear. The proportion between water and sludge, and between acid neutralizing agents and sludge must be right, the mixing of the parts, by agitators, if necessary, must be thorough, if the plants are expected to ripen and the formation of acid sludge is to be avoided. Regular withdrawal of sludge, of proper amounts, will assure good operating results. The sludge drying beds should be so arranged and partitioned that the sludge can be easily measured. The operation should include frequent tests as to the reaction of the sludge and the supernatant water; it should be taken into consideration that not only organic acids but also carbon dioxide, mineral acids and sometimes acid salts are able to cause an acid reaction.

THE FORMATION OF A SCUM COVER AND THE SO-CALLED FROTHING OR SPITTING OF A SLUDGE DIGESTION TANK.

The causes and the detrimental influence of the following phenomena have to be judged differently, depending on whether the sludge considered is already ripe or still in the ripening period. The difficulties on account of the formation of a heavy scum cover are greatest in the latter case.

Experiences with the action of covered septic tanks form the foundation of our conclusions. Although the workings of the septic tank have never been fully explained, enough is known about them which can be applied to the process in sludge digesting chambers. A skin of micro-organisms and particles of fat are formed on an ordinary covered septic tank, rising sludge sticks to this, and the cover gradually increases in thickness. With purely domestic sewage only a floating cover is formed with practically no layers in suspension. In the same proportion, as storm water carrying sand, and industrial sewage carrying heavy matter is introduced into the tank, so the proportion between the floating and suspended layers changes, until finally, if storm water only is handled, suspended layers only are found. (What is said about floating layers holds good for suspended layers, one is as detrimental as the other, but as these are not of a very frequent occurrence, they do not come within the scope of this article).

Sludge in the digestion chamber of Imhoff tanks and kindred installations is originally settled sludge from the sedimentation chamber. Sludge that forms the sludge cover, obtained its floating properties only in the digestion chamber, a property due to its entrained gases of decomposition. The rising to the surface, and remaining there, of single sludge particles or whole sludge cakes, is doubtless dependent on the composition of the raw sludge in the sludge chamber and the intensity of the gas formation (proportional to the unit area of chamber). Fresh sludge will act differently than putrifying sludge, the latter causing less floating scum. A sludge with a larger proportion of organic matter will, under the same circumstances, be less apt to form scum than a highly organic sludge containing a great amount of cellulose. The size of the deposited sludge particles also plays an important part in this matter. Disintegrated sludge particles, if not too fine, will gasify much quicker than coarse ones. It is important whether faeces reach the sedimentation tank well broken up or in large pieces, as it is known that faeces retain their floating ability for a long time, apparently because their compact structure offers a considerable resistance to the liberation of the enclosed gases. Finally, the manner in which the sludge particles settle, their sticking together through fat, hairs, etc., have great influence on the formation of sludge cakes, which to a great extent form the sludge cover. The influence of the intensity of the gas formation will be greater the deeper the sludge layer is. With a small free surface, where it can accumulate, the forming scum cover will be thicker. If the surface is too small for the scum, too many gas laden particles will rise, so that the top layers cannot get rid of their gas quickly enough to sink, it will be carried by the sludge underneath which cannot get rid of its gases, new gases will be formed in the scum cover itself, which will increase the volume of it and its thickness will naturally increase steadily.

A disadvantage of such a cover is that after it reaches a certain size, the rising sludge cakes seek a way out through the open slots to the sedimentation chamber. The principal disadvantage is that the sludge cannot get rid of its bad qualities, because it does not get mixed with the sewage, and the sludge digesting chamber will very slowly or never ripen. Different methods in practice have been tried to destroy and sink the scum covers: such as stirring it with poles or breaking it up with high pressure water. None of these have been a permanent success, as they do not accomplish the necessary continued mixing of the newly rising sludge. This can only be done by the installation of a mechanical agitator, or apparatus which not only stirs the sludge but brings about a thorough mixture of the sludge with the sewage. The most practical way in our estimation is to work against the formation of a too strong sludge cover from the beginning; it is generally to be expected with a sludge rich in organic matter (faeces). The sludge entering the sludge chamber should be well disintegrated, any scum cover in formation should be broken up by a stream of sewage under high pressure, mechanical agitators, or lowering of the water level in the sedimentation chamber. The plant always ought to be designed to have a sufficiently large surface for the escapement of the gases. The appearance of frothing (spitting) in an unripe sludge digesting chamber is a mere secondary phenomenon. It is similar to the so-called "hut" formation in yeast fermentation and is mostly caused by large amounts of gas rising to a small surface and carrying the sewage and finely divided sludge with it in the form of bubbles.

In ripe sludge digesting chambers, the scum formation is much lighter, and not liable to cause trouble, as the rising sludge is well mixed with sewage and the sludge at the surface loses its gas more readily than fresh sludge, not forming as much gas. But it must be removed from time to time. The spitting and frothing and entrance of sludge into the sedimentation chamber are generally caused by a too heavy floating layer of a too large accumulation of undissolved matter in the sludge digestion chamber, in which case the gas will carry up froth if the free surface is too small. This can be helped by simply drawing off some of the sludge or permanently prevented by keeping the sludge at a permanent level. (Special investigations, which we do not care to anticipate are being made on these points by the Emscher Genossenschaft).

CONCLUSION: THE DIFFERENT TYPES OF FRESH WATER CLARIFICATION PLANTS WITH SEPARATE SLUDGE DIGESTION.

Injurious formation of scum covers, frothing or spitting of the sludge digesting chamber, acid sludge and finally acidulation of the supernatant water are the operating difficulties, which are liable to appear in sludge digesting plants, if there is not sufficient room provided to accommodate the floating sludge, a too heavy formation of scum prevented and the sludge withdrawn at the right time. The foregoing statements also indicate that the sludge will have to be withdrawn properly if after being in operation for several months, it should not be good, but a large amount of sludge has been accumulated and the above described results, such as the entrance of sludge into the sedimentation chamber, begin to appear. Sludge accumulation above a certain volume make the withdrawal of part of the sludge an absolute necessity under all circumstances. Waiting will not better the condition of the plant, it will only make matters worse, as the continual deposition of sludge in the sludge digesting room will only change the proportion of water to sludge to the disadvantage of the latter.

It is quite possible for good Imhoff sludge to become acid, if it is not withdrawn at the proper time and fresh acid neutralizing material, i. e., sewage, etc., is unable to enter. The ferrous sulphide will be decomposed, sulphureted hydrogen will be formed and the sludge will lose its deep black color.

Many Imhoff tanks deliver a good sludge, as is well known and the frothing and scum formation does not give any considerable amount of trouble. The intensive gas formation, obtained by the depth of the tanks, causes a thorough mixing of the sludge with sewage in a sludge digestion chamber of generous proportions and so prevents the formation of sour sludge, the timely and well regulated withdrawal of sludge will take care of the supply of fresh acid binding or neutralizing material, and keep the frothing, which is caused by undissolved material in the sludge digesting chamber under control. The timely removal or breaking up of the floating scum cover will prevent the sludge particles, driven to the top by the gases, from sticking to it and causing trouble. What difficulties have sometimes been overcome in a purely empirical manner, very often without knowing anything about them, show the above discussed operating phenomena, which have been observed in practice. These are by no means confined to Imhoff tanks, but will occur in any plant having a separate sludge digestion system, if proper care is not taken. For practical reasons we like to discuss those, although we are well aware of the difficulty of doing so in a short article, without going into details of special cases.

The sludge digesting plants under discussion can be divided into two groups. The first group includes those plants, in which the sludge digesting chamber is below the sedimentation chamber, and is connected to it by open slots, through which the sludge drops automatically. Under this head come the Imhoff tanks, the Travis tanks, Kremer septic tank (Kremer-Imhoff tank), Stiag tanks, Spree tanks, Bus tanks, etc. To the second group belong those, in which the sludge digesting tank is a separate unit and is located beside the sedimentation chamber. The fresh sludge is periodically pumped or drained from the sedimentation tank to the sludge digesting chamber.

First Group: Sludge Digesting Chamber Under Sedimentation Tank.—These plants have the great advantage of an automatic sludge separation. Some of the disadvantages are: The poor operation of the sludge digesting chamber; if the sludge is not removed at the right time, rising sludge will enter the sedimentation tank, which is a common occurrence in spite of the overlapping edges of the slots; difficulty is found in giving the right dimensions to the sludge digesting chamber in proportion to the sedimentation tank.

All installations, coming under this group, require expert supervision while ripening. To avoid acid sludge or a heavy sludge cover, which almost always gives an acid reaction, it is recommended to fill the digesting chamber only partly with sludge during the ripening period and to always keep the sludge at the same level by periodically removing a like amount of sludge. As putrefactive decomposition can only take place while the sludge is alkaline, it has to be kept so at all times, if necessary by the addition of chemicals. For protection against floating sludge covers, it is to be recommended from a technical point to make the free surface of the sludge digesting chamber not too small. The partition walls between sedimentation and digesting chamber should be so arranged that no sludge can collect under them and that the sludge carried up by the gases will be deflected either to the open surface at one side or to the gas vents directly over the digestion chamber, these should be of sufficient size. Wide longitudinal openings as are used on the Travis tank seem to be the best. The long sludge chamber of these and similar plants should be divided into two parts, as in ordinary septic tanks, a large chamber in which the sludge is digested, and a second smaller chamber in which the sludge particles carried up by the gases, are separated. To obtain the largest possible surface and save construction costs, this institute has sometimes recommended the use of earth ponds with large free surfaces as sludge digesting chambers.

Second Group: Separate Sludge Digestion Chambers Beside the Sedimentation Tank.—Under the head of separate sludge chambers, located beside the sedimentation tank, come the designs of Foerster, Mondrion, the Neustadter sludge tank, etc. Opposed to the disadvantage of the cost of pumping, is the not to be lightly valued advantage of the complete separation of sedimentation tank from the sludge tank, so that the latter can never have any detrimental influence on the former. Besides the possibility of unlimited extension separate sludge tanks have the further advantage that they are very accessible and easily inspected. The decomposition of the sludge can be followed in all its stages and can be governed as necessary or desirable. With high ground water the first cost of the shallow tanks will be considerably less.

Separate sludge digesting tanks can be built as single units which are intermittently filled (Foerster) or as a series of tanks which make a continual operation possible (the Neustadter tank). A good sludge digestion is obtained by both types. The operation of single units requires individual expert operation. The continual (daily) operation of tanks in series can be made almost entirely automatic. But the first mentioned construction can be easily arranged so that the tanks can be operated either as units or in series. In arranging the tanks in series, the different processes can be separated, the brisk decomposition of the sludge and sewage will take place in the first tank and in the last tank the sludge can be "washed" by the introduction of clean water before it is drawn off. The advantage of washing the sludge does not in any way interfere with the addition of the sewage necessary for a brisk decomposition. Sewage should always be introduced into the first tank if not enough is contained in the withdrawn fresh sludge. The following procedure can be recommended: The clean water should be introduced into the tank that contains the best digested sludge, from there to the tank that contains the second best sludge, etc., until it is let off, so that the sludge in a certain stage of digestion never comes into contact with sewage from a sludge in a less advanced state of decomposition. The capacity of such tanks ought to be about the same as those that prove [illegible] for Imhoff tanks.

SUMMARY.

Fresh water clarification plants with a separate sludge digestion considered as a whole, are a distinct advance in the field of sewage and sludge treatment; they simplify the mechanical clarification of the sewage, lessen the sludge nuisance and have advanced the solution of the sludge question. The obtaining of a good sludge did not prove to be such a simple matter as was originally supposed. Communities which have their plants built by contractors, will do well not to accept them until the sludge digestion chamber delivers a good sludge. Very careful preliminary investigation should be made in each case to find out whether such a process is suitable to local conditions. Due consideration should be given to the difficulties of construction and operation. Other points to which attention should be paid are: Whether the sludge digesting chamber should be located below or beside the sedimentation tank, or whether both types should be used; also whether it would be best to install an agitator at once or only make provision for a future installation.

With expert design and expert supervision it is possible to avoid the above discussed operating troubles, which often make an otherwise good plant unsatisfactory. It will be absolutely necessary to stop the now prevalent [illegible] all [illegible] pattern, also when considering the efficiency of a plant a sharp line should be drawn between the two functions, sedimentation and sludge [illegible]

Methods of Obtaining Samples of Sewage and Sewagic Liquids for Testing.

[illegible] kindred liquids is to test the samples by chemical or bacteriological methods to determine [illegible] taken and tested with a view to finding out whether the quality of the liquid produced complies with a particular standard. Obvi- [illegible] from analytical results only when the samples are truly representative of the quality of the liquid from which they are taken. Much, then, depends on the skill and intelligence with which samples for analysis are taken. The present article describes the approved methods in vogue in England, for sampling sewage and similar liquids, and is from a paper by G. Bertram Kershaw before the recent annual congress of the Royal Sanitary Institute of Great Britain.

In order to be in a position to judge of the efficiency of a plant it is necessary to know the strength and character of the raw sewage as well as the quality of the effluent. In comparing with a standard this is unnecessary, the sample either passing or failing to pass a given standard.

It is of the highest importance that the sample analyzed shall be truly representative of the bulk of liquid from which it is taken, otherwise erroneous deductions may be drawn. Inasmuch as quantitative analyses are necessarily somewhat costly, and gaging arrangements often involve a good deal of trouble and expense, it is very advisable, on these grounds alone, that the sample or samples drawn shall be as nearly representative as possible

Nearly all waste liquids, especially those resulting from industrial processes, vary, not only in character but also in volume, from minute to minute, from hour to hour, and from day to day. Sewages, moreover, may also vary seasonally by reason of the influx of subsoil water and other factors; they vary markedly when heavy rainfall occurs.

If it is desired to ascertain the quality of a particular liquid at any given time, a single sample (termed a "chance" sample) will often suffice for the special or casual purpose for which it is taken. If, on the other hand, the character and strength of the liquid as a whole is what is to be determined average samples will be essential—that is to say, samples drawn according to the rate of flow of the liquid over at least four days.

SEWAGE LIQUORS.

It will be obvious in the case of sewage liquors, which in dry weather vary in strength roughly according to the rate of flow, that if the hourly proportional flows for five consecutive hours are 10,000, 30,000, 50,000, 60,000 and 20,000 gals., then samples merely drawn in equal quantities every hour will not represent the average strength of the liquid for the five-hour period unless the sewage is hourly of constant strength; whereas, if the samples are drawn hourly according to the rate of flow in quantities of, say, 100 c.c., 500 c.c., 600 c.c., and 200 c.c., and mixed together, an average sample closely approximating to the truth will result. It follows, therefore, that in sampling for average strength the hourly rate of flow of the sewage must be ascertained; in other words, the flow must be gaged continuously while sampling is in progress.

Average samples of sewage liquors were taken as long ago as 1872 by the committee of the British Association at the Romford sewage farm, and probably long before that date, for liquors other than sewage. Average samples of sewage liquors have also systematically been taken by the Sewage Disposal Commission appointed in 1898, and it may be of interest to describe the method of taking average samples of sewage from start to finish, since sewages and sewage liquor are more likely to prove of general interest than the sampling of waste liquors from manufacturing processes.

The first preliminary to be decided upon is the duration of the period over which the samples are to be taken. In the majority of cases, four sets of 24 hour average samples, commencing Monday morning and ending Friday morning, will give a good idea of the character and strength of the sewage. In the case, however, of a large town, sets of average samples extending over seven days, repeated at different seasons of the year, may be necessary. In the case of domestic sewages from small communities, the first four or five days of the week usually constitute a sufficient period to provide data upon which, say, works could be designed; the omission of sampling the Saturday and Sunday sewage, which is often weaker than normal on these particular days, provides a certain factor of safety.

It is always desirable to draw average samples of sewage in dry weather, and hence it is necessary to determine what shall be regarded as dry-weather sewage flow.

The extent to which rainfall will affect the sewage flow will depend very largely upon local conditions, such as the nature of the sewerage system, the character and gradients of the surfaces of the drainage area contributing to the sewers, together with many other factors; but it depends, other things being equal, chiefly upon the intensity of the rainfall. For example, a rainfall of .05 in., or even more, following dry weather, and spread evenly over several hours, may produce little or no appreciable effect upon the flow of sewage, certainly not sufficient to put a stop to the taking of samples; but the question of continuing or abandoning drawing of samples must be left to the discretion of the sampler, who will generally be able to gain some idea from the sewage works manager as to whether or not the sewage flow is being materially affected by rainfall.

If two or three dry days precede the sampling operations, so much the better, but it is impossible to predict seven to ten consecutive days of dry weather in this country.

Since an automatic height recorder in connection with the gaging operations is advisable whenever it can be obtained, it is a good plan to fix a weir and run the recorder for seven to ten days before actually commencing sampling, thus obtaining a series of charts showing the general run of the sewage flow; comparison with these charts when rainfall occurs will generally show when it is inadvisable to continue sampling for the time being.

With regard to rainfall records at most sewage works of any size, a rain gage is kept on the works, but if there is not one in use, a 5-in. gage is easily erected, and records should be taken at 9 a. m. daily during the sampling. If a rain gage is to be found in the town itself, daily records should be obtained from this source also, as it by no means follows that the rainfall in the town and on the works will be identical.

APPARATUS.

As regards the apparatus required for taking 24 hour average samples of crude sewage, about 30, or, better still, 36 wide-mouthed glass-stoppered bottles will be needed for chemical analysis; the use of the extra bottles (apart from chance of breakage) will be apparent later.

If samples for bacterial analysis are needed, a few small glass-stoppered sterilized bottles will also be wanted; these will generally be sent ready sterilized direct from the laboratory, and one or two of such bottles can be filled from the large mixing bottle when the 24 hours' average sample is made up. The stoppers of all the bottles, both chemical and bacterial, should be carefully ground in with fine emery, and the foot of each stopper rounded to avoid shutting in air bubbles.

As regards capacity, the larger bottles should hold about 600-700 c.c. each, and the sterilized bottles about 50 c.c. each. A large glass mixing bottle, holding about 1½ gals., will be wanted for making up the average sample from the 24 separate hourly samples, and also a couple of thick glass stirring rods, and a measuring glass graduated in cubic centimeters; two thermometers, grease chalks, two or three drying cloths, and some adhesive labels complete the outfit. The bottles used for holding the mixed 24 hours' average sample are generally half-winchesters, holding about 1,300-1,400 c.c. each; but for special purposes a winchester is sometimes required.

Turning to the gaging outfit required: The method of gaging employed will depend upon local conditions. Whenever practicable, a wooden dam, with a brass or gunmetal rectangular weir plate screwed to the face of it is the simplest method, the whole of the flow to be gaged passing over the weir, and the depth of liquid over the weir sill being registered continuously by an automatic height recorder. "Plastic," or a mixture of clay and boiled linseed oil, is useful for making watertight joints between the margins of the wooden dam and the channel walls.

It may be well to point out here that to avoid complications in the formula for discharge, owing to the existence of material velocity of approach, one or two points need to be kept in mind.

(1) The width of the weir should not exceed about one-third the width of the channel in which it is placed.

(2) The transverse sectional area of the liquid falling over the weir should not exceed about one-fifth of the transverse sectional area of the liquid in the channel immediately above the weir.

(3) The weir and dam board should be perfectly upright, and have a free overall and air space underneath the falling sheet of water, while the float of the automatic recorder should be placed sufficiently far back from the weir to avoid curvature of surface.

As to the type of recorder, I have always used a small portable Glenfield and Kennedy machine, the drum being driven by a seven-day clock, and the pen actuated by the up and down movement of the float. There are, however, several automatic height recorders on the market, some of them with pen or pencil cams cut to give the discharge direct upon the chart. It is not, however, a difficult matter for any engineer to construct a simple and reliable machine for comparatively little outlay. Care is required in adjusting the pen to zero on the chart, which should be carefully pressed down to the rim at the foot of the drum.

Most recorder pens have an up and down range of 6 ins., so that the depth over the weir must be limited to about 6 ins., unless the minimum early morning flow exceeds, say, 1½ in. depth over the weir, in which case the recorder can be made to register up to 7½ ins. by setting the pen at zero on the chart when exactly 1½ in. of liquid is passing over the weir, the chart base line thus reading 1½ in. above sill level. The recorder should be set up perfectly level, and the float, if in an exposed situation, should be surrounded by a cylinder of fine perforated zinc, to eliminate wind action, otherwise oscillation of the float will soon exhaust the ink in the pen, besides blurring the chart. It may be mentioned that a mixture of glycerine and charcoal, or any aniline dye, makes a good ink for a recorder pen.

Having erected the weir and recorder, the sample bottles should be prepared and the time for commencing settled. Assuming 9 a. m. to be the hour determined upon, and that average samples of crude sewage are to be taken, the recorder should be started at 9 a. m., the second at 10:30 a. m., and so on hourly until 8:30 a. m. the following morning, when the first set of 24 hourly samples will be complete. The filling of each bottle is usually most conveniently done at the weir outfall.

It will be found convenient to write on each bottle the nature of the liquid, the date, and the hour of drawing; this can be done with grease chalks upon the glass, direct, or adhesive labels can be used.

As each hourly bottle is filled the stopper should be allowed to sink down gently into place in the bottle neck; if this is not carefully attended to there is considerable risk of casualties when an attempt is made forcibly to unstopper the bottle. After stoppering the bottles should be set in a row in the order of the times at which they were filled. It will be found, when the 24 bottles are ranged in a row, that a wonderfully good idea can be obtained visually of the variations in the character of the sewage throughout the 24 hours.

PROCEDURE.

A point now arises as to the inclusion of fragments of faeces in a sample. Although they are usually omitted when taking chance samples direct with half-winchester bottles, this would appear to be a faulty method of sampling in certain cases, especially where the crude sewage subsequently undergoes treatment in septic tanks. If they are included they will need to be broken up in the hourly sample bottles before the proportionate amounts are drawn off.

After the last of the 24 hour samples has been taken, and the recorder pen has reached the 9 a. m. ordinate on the chart, the chart can be removed and replaced by a fresh one, and the hourly rates of flow worked out and entered up on the finished chart. The date, rainfall (if any) and other notes of interest should be recorded, such as times of weakest and times of strongest sewage, flushes of trade wastes, etc. For working out the hourly flows on recorder charts I use a special slide rule, which has the advantage of shortening the labor considerably. In the case of a recorder provided with a cam, the discharge can, of course, be taken direct from the chart; but with a recorder that only registers the height, height diagrams have to be translated into discharge diagrams.

The next step is to take from each bottle, commencing with the 9:30 sample, an amount proportionate to the rate of flow for each hour, and to transfer it to the large mixing bottle. Assuming the flow for the first three hours to have been severally 5,000, 6,500, and 8,500 gals. per hour, then if 50 c.c., 65 c.c., and 85 c.c. are taken from the first, second, and third bottles, respectively, these amounts will be proportionate to the rates of flow for those hours, and the same procedure must be followed with the 21 remaining hourly samples.

The measurement of the liquid from the hourly sample bottle is best effected by means of a tall glass cylinder graduate in cubic centimetres. Before the sample bottle is unstopped it should be turned over once or twice to mix the contents, and large solids should, if present, be broken up with a glass rod. In many cases the hourly amounts to be taken from each bottle can be obtained by knocking off two or more figures from the right of the hourly rates of flow, and if the figures thus obtained are inconvenient, the whole of these can be either divided or multiplied by a common factor. Care should be taken that the aggregate of the figures adopted are sufficient to yield enough liquid for the final sample; and it is better, if this can be done, to take sufficient liquid to fill two half-winchesters in case of breakage of one of them.

The working out of the chart and the sub-sampling, together with the entering up in the proper hour spaces of the chart of the times of strongest and weakest sewage, presence of trade wastes, etc., take some considerable time, and while this record is in progress the reserve bottles previously mentioned will be needed to take the first few hours of the second set of samples.

When the proportionate contents of the 24th hourly sample bottle has been transferred to the mixing bottle, the whole of the contents of this large mixing bottle should be gently agitated while the half-winchester bottle or bottles are being filled from it, in order to keep the suspended solids evenly distributed throughout the liquid. After the half-winchesters are thus filled the hourly sample bottles can be emptied, washed out with clean water, and set to drain. In cold weather, half-winchesters, or winchesters filled with sewage, tank liquor, or effluent, should be filled brimful, and the stoppers gently worked in, and tied down with a piece of calico or string; in hot weather, and also in keen frosty weather, a small air space may be left at the top of the bottle in the case of sewages or tank liquors. The samples should be despatched to the laboratory carefully packed in wicker cases or hampers, nothing being better than old crumpled newspaper for packing the bottles in. It is often useful to have a special sample of the weakest early morning sewage, taken in a half-winchester, to be examined for nitrates. These, if present, indicate the presence of subsoil water in the sewage.

It may be noted that in the case of very small sewage works, where an automatic height recorder cannot be obtained, depths over weir can be taken with a thin steel rule, one edge of which is ground thin. Depths should be taken sufficiently far from the weir sill to avoid curvature, and it will be found that if the rule is rubbed lightly against a block of wet clay and allowed to dry before each reading, the exact depth can easily be seen at a glance. The gaging of sewage flows in this way is very tedious, since readings should be taken every ten minutes, or at even shorter intervals, should the flow of sewage fluctuate much, as will almost certainly be the case with a small sewage works. At least three observers are needed for the 24 hours for taking sill depths and drawing samples.

There is another method of taking average samples, in which samples are taken at regular intervals throughout the 24 hours, each of these samples being analyzed separately; a "weighted sewage" can be obtained by multiplying each separate analysis by the rate of flow at the time the sample was taken, the results are then added up and divided by the total flow.

Some years ago the writer, having occasion to take a large number of average samples, designed an instrument for taking samples directly proportionate to the flow in one operation. Briefly described, the instrument consisted of a paraboloid copper tube or container, about 12 ins. long, ranging from about ⅝ in. at the bottom to about 2¼ ins. at the top, the several diameters being calculated from the gaging formula. This tube was attached to a sliding metal stem working up and down in sockets screwed on to a rod provided with a thin sole plate, upon which the tube rested when the sample was taken. When in use the tube was raised clear of the sewage tank liquor or effluent, and the sole plate placed on the measuring stud, which was about ⅛ in.—i. e., the thickness of the sole plate—below the level of the weir sill. The tube was then dropped through the liquid on to the sole plate, and the intercepted liquid poured off direct into the mixing bottle; a thin rubber pad lying in a recess in the foot or sole plate prevented any escape of liquid, while a strong clip at the head of the instrument kept the container tube well home against the rubber. In this way, at each sampling, a portion of liquid was withdrawn proportionate to the flow of liquid, in other words, proportionate to the height of liquid flowing over the sill. Like most automatic contrivances, however, while possessing many marked advantages with certain liquids, containing suspended matter of moderate size, it had also several drawbacks and the longer and more tedious method already detailed is to be preferred in most cases.

There are home-made automatic samplers to be seen in use at several sewage works, most of them actuated by a small water-wheel driven by a flow of sewage, tank liquor, or effluent.

TANK LIQUORS AND EFFLUENTS

The sampling of tank liquors and effluents is effected in a similar manner to that already described for sewages, and it is easier, because less complicated by solids; and when both sewage, tank liquor, and effluent are sampled at the same time, it is often convenient to draw what are known as "corresponding samples." By corresponding samples is meant the taking of two or more samples of the same liquid in its different stages, a certain interval of time or "lag" elapsing between the drawing of each of the said samples. For example: assuming a sewage to be flowing at a uniform rate into a tank having a six hours' capacity, and that a sample of the sewage is drawn as it enters the tank, a sample of the tank liquor taken six hours later will be a sample—other things being equal—corresponding to the sample of sewage.

It is often necessary to use dyes, such as fluoresin or eosin, in order to follow the progress of a particular liquid through several processes. For example, if the temperature of a liquid entering a settling tank is much higher than that of the liquid in the body of the tank, short cuts through the tank at the surface are almost certain to occur, and a dye, such as fluoresin, shows this at once. Samples should be taken just before the dye reaches the sampling point to avoid including the dye in the sample.

Corresponding samples are also exceedingly useful in showing the relative amount of work brought about by the several stages of a purification plant; the only objection to them is the time occupied in obtaining a complete set. It may be noted that the chlorine figures on analysis will usually but not invariably, show whether the samples of sewage liquor correspond properly.

Effluent from Contact Beds—With regard to the taking of samples of effluent from contact beds, it is well known that the quality of a contact bed effluent varies considerably as the bed is being run off, being poorest when the outlet valves are first opened, afterwards gradually improving until the bed is practically empty. It has also been found to be the case that samples taken at the period of midflow from the bed approximate closely to the average quality of the whole of the effluent discharge, and it is therefore sufficient in the majority of cases to take a sample at midflow from each particular bed discharge, and to mix the whole of them together to make up the 24 hours' average sample.

Storm-water samples can be taken automatically, if desired, by fixing a vessel on the side of the storm overflow manhole at the level at which the required dilution occurs.

Care should be taken in sampling all sewage liquors that no sediment is stirred up in any channel or manhole and mixed with the sample, otherwise the sample becomes worthless. This is of especial importance in taking samples of effluent.

It is, perhaps, not generally realized how much effluents vary in composition in the course of 24 hours, even in the case of land effluents, and when it has been customary to take a sample of effluent regularly at the same hour daily, it is often surprising to see the difference shown by average samples taken over the whole 24 hours.

TRADE WASTES.

The sampling of trade wastes stands on a different footing to the sampling of sewage liquors. A thorough working knowledge of the particular trade in question is needed if the sampler is to take representative samples. Trade wastes vary in character in short intervals of time, owing to the different operations which are frequently in progress at the same time at the one manufactory, as, for example, where bleaching, printing and dyeing are carried on simultaneously at the same works, as is often the case.

In the case of many trade wastes, when taking average samples it is advisable to take a sample every five or ten minutes owing to their rapidly varying character, and each separate sample should be tested with litmus for acidity or alkanitity, and the temperature noted. It is frequently desirable to take samples of each of the various liquors from a manufactory as well as of the general or mixed waste; in such circumstances, difficulties in arranging the flow are apt to arise, the contents of vats being let off with a rush in the space of a few minutes, the chart of the recorder consequently showing an abrupt vertical line which cannot be integrated for discharge. In such cases a number of dip samples may be taken, and the capacity of the vat or tank taken, and noted in pencil on the chart at the proper point.

At some manufactories with which the writer is acquainted, a variety of gaging expedients have been necessary in order to arrive at accurate information as to the volume of discharges.

Most trade-waste investigations demand a rather elaborate scheme to be laid down before satisfactory sampling can go forward, and since the analysis of many of these wastes is laborious and intricate, and consequently costly, it is very advisable that, as in the case of sewage liquors, the samples shall be as far as possible representative of the liquids from which they are drawn. Many trade wastes, it may be mentioned, do not, owing to their nature and color, admit of the use of eosin or fluoresin for tracing the various liquids through their several processes.

RIVER SAMPLES.

River samples require considerable care in taking, especially where large rivers are concerned, and where bacterial samples are in question; the boat from which the sampling is done often itself carries contamination on its sides at the water line. All river samples should, wherever practicable, be cross-sectioned ones, that is, a number of samples should be taken on a line normal to the axis of the river, and these should be mixed together for the final samples.

For the taking of surface samples in large rivers, I have found two glass tubes of ⅜ in. internal diameter wired together, and about 18 ins. long, exceedingly useful. As a boat is rowed very slowly along the section line, dip samples are rapidly taken every 6 ft. or so, alternately 6 ins. and 2 ins. deep, and the fingers or palm of the hand closed over the top apertures of the tubes, which are then quickly emptied into a glazed earthenware pail. If corresponding samples are needed, the river water should be colored with fluoresin when the first set of samples are taken, and practically the same body of water can then be detected and resampled lower down. Two or more samplers are needed for taking the second sample as quickly as possible just before the dye reaches the cross-section. To those who have been accustomed to judge the average velocity alone, the slow average rate of flow shown by the fluoresin will come as a revelation.

There are many other ways of taking special samples, such as deep-well samples, many of them requiring specially designed and expensive apparatus, but they are perhaps of special and not general interest, and for that reason they are not included in this paper. In conclusion, it may be observed that, although the procedure involved in obtaining true average samples may appear at first sight to be too refined for general use, it should be carefully borne in mind that a substantial error in the strength and character of a sewage—such as may very easily arise if a few casual samples taken at haphazard are relied upon—may be the cause of sewage purification works being constructed not competent to bring about a proper degree of purification of the sewage they are called upon to treat.

Repairing Cracked Sewage Tanks by Grouting at Belfast, Ireland.—The sewage tanks at Belfast have a capacity of 5,500,000 gals. They are located in a formation known locally as "sleetch." This material is made land; when dry it crumbles like fine sand, with a little moisture it swells and becomes like a soapy clay, and with a large quantity of water it has the consistency of soft dough. These tanks were leaking and cracked in the bottom and sides and were repaired by grouting under pressure. The cost of rebuilding them would have been in the neighborhood of $200,000, and, in addition, the sewer service would have been more or less interrupted in the event of reconstruction. The city surveyor took one tank at a time and forced cement grout into the crevices and under the floor; altogether 1,120 tons of cement were used. It was mixed with an equal quantity of sand, and was put in under air pressure varying from 7 lbs. to 15 lbs. per square inch. The reservoir is now quite watertight. Excavations recently made alongside it show that the grout found all the crevices in the foundation and followed them through the sleetch, even 4 ft. beyond the outer walls.

ROADS AND STREETS

Paving Methods in Baltimore, Maryland.

Contributed by Harry D. Williar, Jr., C. E., Assistant Engineer, Baltimore Paving Commission.

Baltimore, Md., has, up to the past few years, been generally known as "The City of Cobblestone Streets," a unique distinction not undeserved. Since the first cobblestone was laid in 1781, the mileage of this class of paving has increased until on January 1, 1912, out of a possible 590 miles of streets 408 were paved with cobbles. The 182 miles of streets without cobbles were paved in a large measure [illegible] sisted of the main thoroughfares of the city.

[illegible] temporarily improved streets in Baltimore was due to the fact that the city was without a comprehensive sewerage system, and it was [illegible] streets before such a system—which was inevit[illegible] well under way one of the most efficient and modern storm-water and sanitary sewerage systems in the country. It was at that time that a Paving Commission was formed, that since its inception has transformed about 67 miles of cobbled streets into streets of smooth paving. At this present rate of progress in eight years the city will be entirely free of cobblestones.

CONSIDERATIONS INVOLVED IN THE SELECTION OF [illegible]

In selecting the materials to be used on the different streets the city has considered closely the following important factors:

(1) The location of the street (business or residential district). (2) The character of traffic to which the street is to be subjected (light, heavy, "standing," or through traffic). (3) The grade of the street. (4) The width of the street. (5) The existence or non-existence of street railway tracks.

There are very few cities where one class of paving can be used on any one street throughout its entire length. Due to the heavy grades and general topography this feature is particularly noticeable in Baltimore, and a great variety of paving has been used. Regardless of the class of material, the following general qualities enter into the construction of all modern pavements:

(1) It should be impervious; (2) afford a good foothold for horses; (3) be hard and durable, to resist wear and disintegration; (4) be adapted to the grade; (5) suit traffic conditions; (6) offer the minimum resistance to traction; (7) be as noiseless as possible; (8) yield neither dust nor mud; (9) be easily cleaned; and (10) it should be economical.

TYPES OF PAVEMENTS ADOPTED.

The following are the classes of pavements being used in Baltimore with the nature of streets on which they are generally adaptable:

Granite Block.—On streets of excessive grades and in business districts where the traffic is either very heavy or standing. Where traffic demands the immediate use of the thoroughfare, bituminous filler is used; otherwise cement grout.

Sheet Asphalt.—In residential sections, and business districts where there is through but not not exceptionally heavy traffic. On grades up to 5 per cent it gives excellent results; on steeper grades it becomes slippery and unsafe.

Vitrified Block.—On semi-residential thoroughfares of average traffic up to 7 per cent grades. On steeper grades in place of granite blocks, "Hillside" vitrified block, in a number of cases, is being used and is giving excellent results.

Wood Block (Properly Treated).—On streets in the vicinity of hospitals. Owing to its being practically noiseless, it makes an excellent pavement for such locations. On grades over 2 per cent it becomes dangerous and unsafe for horse traffic, and should not be used.

Bituminous Concrete.—On outlying streets carrying light traffic.

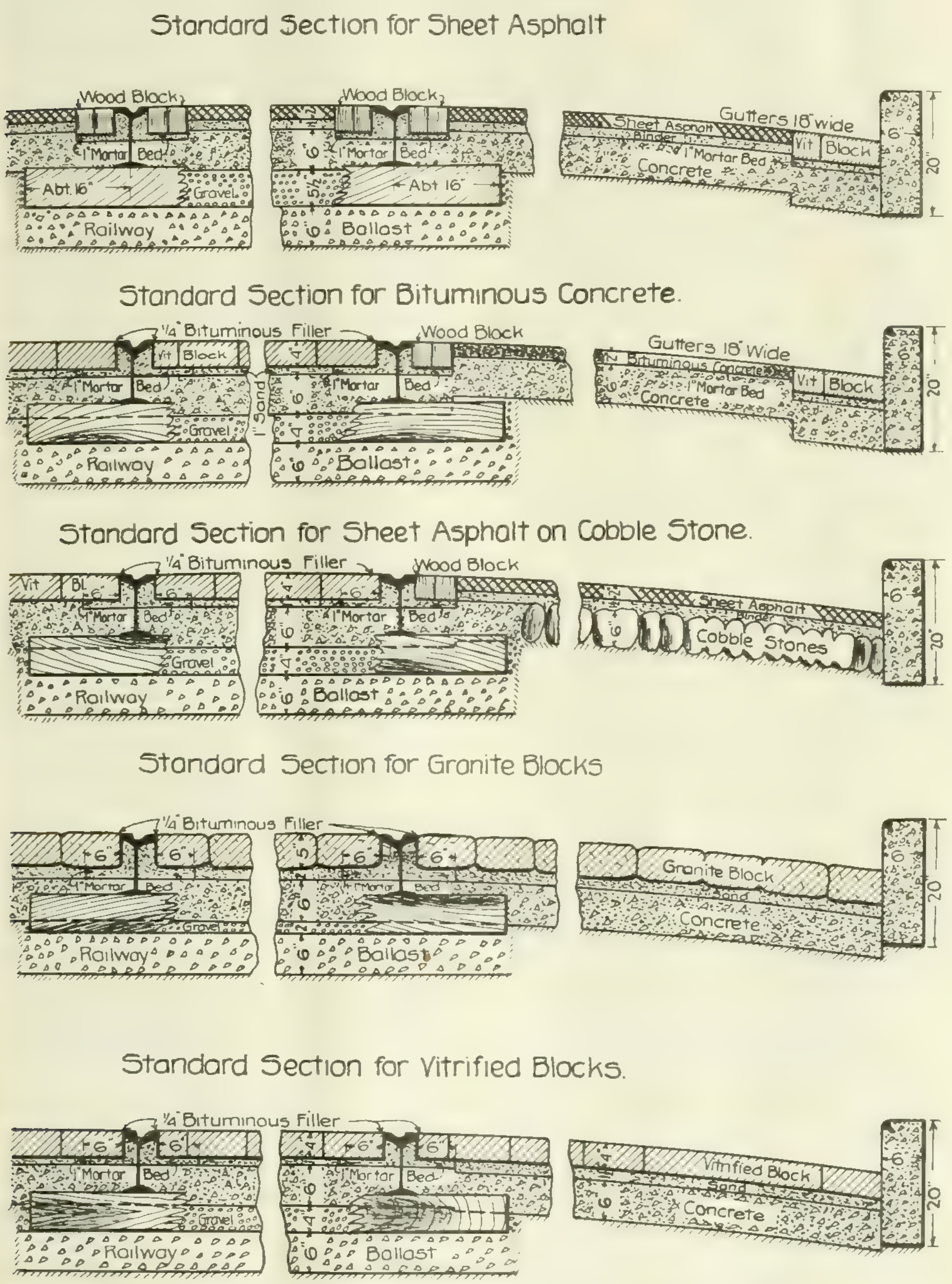


Fig. 1. Part Cross Sections of Pavements Constructed by the Baltimore Paving Commissioners.

The following table shows the mileage of the pavements laid in Baltimore since 1912, with the average price of each for two years:

Class of pavement.	Mileage laid since 1912.	Average price per sq. yd. 1912.	Average price per sq. yd. 1913.
Sheet asphalt—			
1½-in. topping, 1½-in. binder, 6-in. base	31.50	$1.67	$1.66
Vitrified block—			
Cement filler, 6-in. base	18.02	2.09	2.24
Hillside vitrified block—			
Cement filler, 6-in. base	.92	....	2.28
Granite block—			
Cement filler, 6-in. base	5.66	3.32	3.39
Granite block—			
Bituminous filler, 6-in. base	4.42	3.32	3.62
Bituminous concrete—			
2-in. wearing surface, 4-in. base	5.24	1.34	1.30
Wood Block—			
Sand filler, 6-in. base	0.63	3.18	3.25

The above prices apply to work of the Paving Commission only, and not to that done by the Commissioners for Opening Streets or the State Roads Commission, both of which commissions do paving within the city limits.

CONSTRUCTION METHODS.

The paving of Baltimore, not unlike that of other large cities, consists in converting previously laid out unimproved streets into streets of smooth and modern paving. Owing to the proximity of buildings, the grades and lines are changed but little.

Curbing.—Armored concrete curb, Fig. 2, is used to a considerable extent and is constructed 6 ins. wide and 20 ins. deep in 10-ft. lengths. The concrete, thoroughly mixed in the proportion of 1 cement, 2½ sand, and 5 stone or gravel, is placed in wooden forms and thoroughly rammed and spaded in 4 or 5-in. layers until within 2 ins. of the top. This 2 ins. of top surfacing is composed of cement and sand, or fine granolithic mixed in proportion of 1 to 2. The upper edge of the face of the curb is protected and reinforced by a galvanized iron or steel bar (Wainwright No. 1) securely anchored to every 10 ft. of curb by four cast iron frogs. It is important that a clean expansion joint running the entire depth of the curb be allowed between every 10-ft. section. The setting of forms accurately to grade and line, substantially bracing them to prevent bulging when the concrete is put in, and a good solid foundation, are the fundamental requisites for good concrete curb. The average cost of this class of curb is 62 cts. per lineal foot.

On some streets where stone curb is already in place and is found to be of sound granite, gneiss or sandstone, it is being thrown out, redressed, and reset to the new lines and grades at an average cost of 31 cts. per lineal foot.

Concrete Foundation.—The thickness of the concrete foundation for any street is determined only after a careful study of the thoroughfare in question has been made. Present and future traffic conditions and the composition of the subsoil are the principal governing factors. On streets or alleys of light traffic, with a good solid subgrade, a 4-in. base is used. Where the traffic is heavy or congested this depth is increased to 6 ins. Although there are some engineers who recommend as high as an 8-in. or 12-in. base in cases where the subgrade is bad, it is, in the opinion of the writer, a decided waste. If the subgrade is soft, spongy or such that a 6-in. foundation will not hold, then it becomes necessary not to increase the foundation, but to increase the carrying capacity of the subgrade by means of artificial drainage. In the last analysis, the stability of all pavements must depend not upon the surfacing or the base, but upon the stability of the subsoil to support the base. A concrete properly mixed in the proportion of 1:3½:7 will give excellent results.

Sheet Asphalt.—Baltimore's specifications for sheet asphalt pavements permit of all the brands of asphalt that will pass the requirements of the specifications of the American Society of Municipal Improvements. To date about 75 per cent Mexican asphalt has been used, the rest being Bermudez and California. It is laid in two courses, 1½ ins. of binder and 1½ ins. of topping, except on a few streets where the topping, or wearing surface, is increased to 2 ins. thick. An effort is made to have the concrete foundation for bituminous pavements somewhat rough without having loose stone on the surface. This is particularly desirable on streets having grades of over 2 per cent, to prevent the surfacing from creeping. The writer recommends the use of 1-ft. or 1½-ft. vitrified block gutters, laid at right angles to the curb, in connection with all asphalt pavements. If gutters are not needed on account of a daily flow of water on the street, they permit the proper rolling of the surface, which otherwise is impossible to obtain on account of the rollers being unable to get close to the curb. Care should be taken to keep the asphalt extra high along the edge of the gutter, otherwise in a year's time it will be found low at that point and give an ugly appearance to the street.

Some experiments have been made with sheet asphalt or bituminous concrete laid directly on the old cobblestones as a foundation. The results have not been satisfactory. It is the general opinion that there is no saving or economy, even on streets of light traffic, in this form of construction. The risk of the settlement of ditches, probably unforeseen at the time of construction, is great, and the necessary filling of low spots with concrete gives unsatisfactory results. Although about 6 miles of bituminous surfacing has been laid on a cobblestone base, the method is not considered a good one, and this form of construction has been discontinued. It is recommended, therefore, that in place of using the cobblestones as a foundation, they be sold the contractor at a price named by him, removed, crushed to the required size, and used in a concrete foundation for the same street. A price per square yard for these old cobbles is now named by the bidding contractor on each contract, and the amount deducted from the total tabulation of the bid. The average bid price for this old material is 1/10 ct. per square foot.

Vitrified Block.—The Paving Commission is laying about 10 miles of vitrified block annually of which, in 1913-1914, 75 per cent was composed of wire-cut-lug block. The abrasion test required is 22 per cent for streets and 24 per cent for gutters. In the construction of a block pavement every effort is made to have the surface of the concrete foundation perfectly smooth since rough places are likely to project through the sand cushion and cause the block to "ride." The grout will soon work

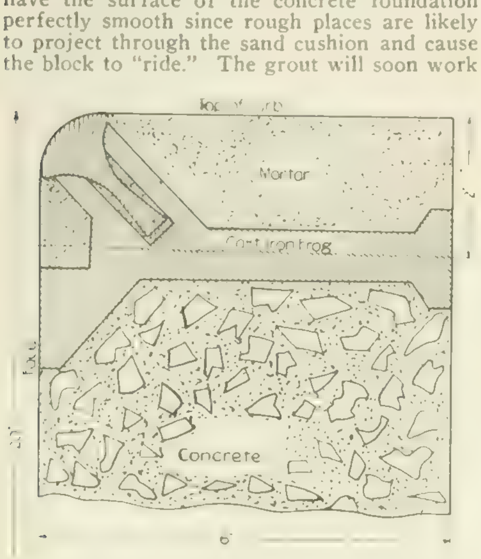


Fig. 2. Curb Protected by Wainwright Bar.

loose where such a condition exists, and in a short while this failure will spread over a large area. A smooth condition of the base is an essential requisite for good results which is too often overlooked. Several thicknesses of sand cushion were tried to determine which would give the best results. A cushion 1½ ins. thick, after thorough compacting, was adopted. This cushion should be struck off to the proper cross section, thoroughly rolled with a hand roller weighing about 250 lbs., and again struck off. This process should be repeated until the sand is so firm that it will not work up in the joints when the block sur- [illegible]

No block pavement should be grouted if the sand has worked up in the joints more than ¾ in. from the bottom. If the sand works up more than this, the joints will not receive their full amount of cement grout, and failure is inevitable. The writer has seen squares at a time torn out and relaid because of this condition. The cause of sand in the joints is due to an improperly prepared sand cushion, or to the block being laid too loose. Sprinkling water on the cushion just prior to laying the block will frequently eliminate this trouble.

The next most important step in the construction of a vitrified block pavement is the preparing and applying of the cement filler. Baltimore is following in this detail the speci- [illegible] Manufacturers' Association. Good sharp clean sand and tested cement are mixed dry in proportions of 1 to 1 and dumped in a specially constructed box having one corner lower than the other. Water is then added until the mixture is like thin cream. The surface on which the grout is to be applied is thoroughly sprinkled with water, and by means of scoop shovels the grout is then thoroughly spread over the street. Brooms work the grout into the joints until the last course is applied, when a squegee is used to give a finish.

[illegible] traffic for 14 days after its completion, and during warm weather, for the first week, sprinkled daily with water.

Clean cut longitudinal expansion joints are very important in a cement filled block pavement. Too large a joint is better than too small. Tar or asphalt should be poured in the joints, the width of which depends on the width of the street. The following if carried out will give good results:

On streets 30 ft. wide and over between curbs, 1¼-in. joint next to each curb. On streets between 20 ft. and 30 ft. wide, 1-in. [illegible] ft. wide, ¾-in. joint next to each curb. On streets under 12 ft., ¾-in. joint next to each curb.

Close inspection is absolutely necessary during construction. There are more small points and matters of detail that play an important part in the proper construction of this class of pavement than in any other. If they are overlooked for the big points, which usually take care of themselves, the pavement will be a failure; if followed out, the pavement will be a permanent success.

Treatment Along Street Car Tracks.—In the railway area Baltimore is using, for the most part, vitrified block; some granite block and a small amount of scoria block and sheet asphalt are also being placed. Where vitrified block and granite block with cement filler are used, a bituminous strip is laid between the rail and the block. This strip forms an expansion joint, and also acts as a shock absorber, relieving the block of any motion or vibration of the rail. This strip has eliminated a number of failures in pavements at points where they are most frequently found, i. e., along street car tracks.

Where sheet asphalt is laid on a car-track street from curb to curb, either wood or scoria block liners laid on a mortar bed are used along the rails. Where possiblbe the writer advocates the embedding of the railway ties in concrete. This is being done on the main thoroughfares in Baltimore.

Notes on the Width, Alignment, Grade and Drainage Features of the Designing of Country Roads.

Important preliminary features of the designing of country roads are the consideration of width, alignment, grade and drainage. These constitute the permanent portion of the road. A brief summary of current practice with regard to such details in New Jersey is given by Robert A. Meeker, State Highway Engineer of New Jersey, in a paper before the Canadian Road Congress.

WIDTH.

The first point to be considered in designing a road is its width. It may be generally stated as axiomatic that the width of roads should be in multiples of eight, this being the width that should be allowed for each vehicle using the road. A road 8 ft. wide might more properly be termed a lane leading to one building or a small group.

The next in importance should be 16 ft. in width, in order that two vehicles might have sufficient width in which to pass.

The third width, or 24 ft., would permit of two vehicles passing while the third was standing along the side of the road—or two loads of hay or other bulky material to pass.

The fourth width, or 32 ft., permits two vehicles to stand along the sides and leaves sufficient space for two other vehicles, moving in opposite directions, to pass each other in safety. These widths refer to the traveled carriageway alone, no allowance whatever being made for the accommodation of pedestrians, nor for any drainage structures.

In order to obtain a roadway of sufficient width to accommodate travel passing in both directions, 24 ft. may well be taken as the minimum allowable, and if there is the prospect of an increase of traffic in the near future, a proper addition to the width of the surface, necessary for the accommodation of the traffic, should be provided for in the original design. It is almost impossible to properly grade and drain a road of less than 24 ft., and the wider the roadway the more easily it is drained, and also maintained, due to the fact that the traffic is distributed over a greater area, and that the surface is more freely exposed to the drying action of the wind and sun, thereby preventing the formation of mud and ruts.

ALIGNMENT.

The second problem is that of the location of the line. On a new road this is determined by certain well-defined principles. First, the beginning and ending points should be connected by the most direct line; second, the grades should be kept as low as possible; third, for economy's sake, the line should be so located as to reduce the amount of grading to the minimum, likewise the number and size of the bridges.

The factors governing the departure from a straight line are many. In crossing a ridge we seek the lowest point in the summit, in order to avoid expensive cutting or the alternative of steep grades; in following a valley we keep well up on the hillside, to avoid bridging ravines and small water courses; if we encounter a swamp or pond we can frequently, by swinging the line, save the expense of a heavy fill; a stream may be avoided by diverting the line, thus saving the cost of bridges.

On an old road another set of problems has to be solved; these are chiefly those of expediency. Though a straight line between the termini may not only be the best but also the cheapest, the claims of intermediate communities may be so strong that the line must be diverted from its best course to satisfy the wants of the communities to be served. But through it all, in spite of all of these warring factors, the engineer must never lose sight of the straight level line between two points as his ideal. By keeping this constantly before his mind's eye the results that may be achieved will often surprise even the author.

GRADE.

The grade, or the angle which the axis of the road makes with a horizontal line, is the most important economic feature in road design, for upon it depends the amount of material a horse can draw over the road. The results of experiments made both in England and France prove that a horse can haul twice as heavy a load up a 2 per cent grade as he can up a 6 per cent grade. That being so, the value of a road for heavy traffic, having a maximum grade of 6 per cent, is only one-half of that having a maximum of 2 per cent. This fact is often lost sight of in designing new grades, the object of many road officials being to build as many miles of road as possible for a given amount of money, the first cost, and not the ultimate value of the road

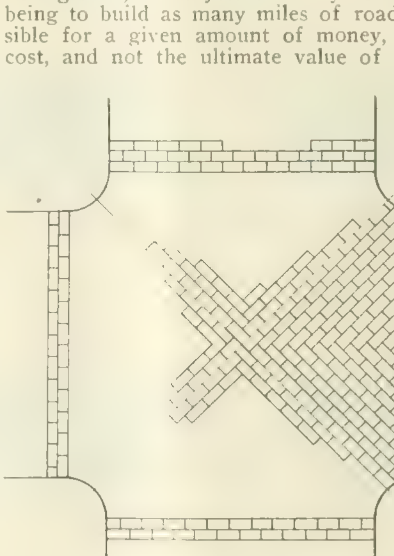


Fig. 3. Double Diagonal Plan for Paving Street Intersections.

to the community, being the basis upon which the improvement is made.

DRAINAGE.

Having laid out your road as straight as possible, and having reduced your grades as much as your funds will permit, the next important problem is that of drainage. This is of two kinds—surface and subsurface. Surface drainage is both transverse and longitudinal. Every road must be so planned that the water which falls upon its surface will not remain upon or along it. The first object is attained by giving the road a proper crown or cross section, so that the water may be conveyed quickly to the gutters on the sides. This crown should have the form of the arc of a circle, drawn through three points—the center of the road and the gutter on either side. The elevation of these points should be in the following ratio: For earth roads a fall of 1 in. per foot from the center to the gutters; for waterbound macadam, ¾ in. per foot, and for bituminous concrete ½ in. per foot. This form of cross section permits of the fullest use of the road, and at the same time conveys the water to the gutters without washing the sides or shoulders of the road. The longitudinal surface drainage is taken care of by the gutters, which must be carefully trimmed to conform to the grade of road, all holes being carefully filled and all humps cut off. In fact, the gutters must be as carefully graded as the center of the road. Proper inlets to bridges crossing the road should always be constructed if the bridge is as wide as the carriageway. In some soils these precautions are not sufficient, and we are then compelled to lay underdrains. These should be placed about 3 ft. inside of the gutter line, for two reasons: First, to intercept the subsurface water before it reaches the middle of the road, and second, to prevent erosion in case the gutters are gullied. The object of underdrains is to cut off the subsurface water before it can get beneath the traveled road; therefore their place is on one or both sides of the paved way.

SURFACING.

The last thing to be considered in the designing of a road is the pavement. This will depend upon the kind and amount of traffic. For country roads gravel or plain macadam is sufficient. For suburban and small town roads a surface treatment, either with bituminous or lignin binders, will suffice; if the traffic is great, bituminous concrete or brick, and in and near large cities, where the traffic is very heavy, stone blocks.

The Control of Openings in Street Pavements and the Cost of Restoration in Cincinnati, O.

The proper paving of a street is simple compared to the difficulty of keeping it paved. No sooner is the pavement completed than public utility corporations and property owners seem to unite in tearing it up.

Interesting data on this subject, collected from a number of cities, is given in a recent report on the pavements of Akron, Ohio, prepared by the Municipal University of that city.* Parts of this report are included here.

Akron issued 1,490 permits for street openings in 1913. If the average length of these openings is assumed to be the same as those of Boston in 1910, their total length is nearly 17 miles. If the average width of the openings is taken as 3 ft., the total area equals that of a street 30 ft. wide and 1.7 miles long, or over 10 per cent of the total new pavement laid in 1913.

Complete prevention is at present impossible, but much can be done to reduce the number of these openings to a minimum. Cincinnati will not lay a pavement until water and sewer mains are provided for and their house connections to each lot made. It also has an ordinance prohibiting the opening of streets for a period of three years after they have been completed. Macon, Ga., passed an ordinance on April 7, 1914, prohibiting street openings in new pavements for a period of five years from the time the paving is commenced. Akron has no ordinance restricting excavation in streets newly paved.

The repaving of such openings is done by the Street Repair Department and the permittee is charged the actual cost of the labor and materials. In Cincinnati a careful record of cost was kept by F. S. Krug, chief engineer, on the repair of openings in different kinds of pavements and a schedule of prices which include an overhead charge has been issued, based on this data. This enables the property owner or corporation to find out in advance how much his excavation will cost him. These costs are shown in Table I.

TABLE I.—COST OF REPAIRING OPENINGS MADE IN THE PAVEMENTS OF CINCINNATI, OHIO, CLASSIFIED BY THE SIZE OF OPENING AND TYPE OF PAVEMENT.

Size of opening, sq. yds.	Surface, sq. yds.	Concrete, cu. yds.	Total cost.	(1) Extra material.	(2) Average cost per sq. yd.	(3) Avg. cost per sq. yd. overhead included.
			ASPHALT.			
1	304.8	63.95	928.51	3.67	1.83	2.29
2	502.4	116.69	1,396.22	....	1.68	2.10
4	805.9	176.74	1,530.07	...	.67	.84
8	354.5	67.83	623.90	..	.69	.86
12	732.1	65.50	885.72	....	.74	.93
50	3,784.1	131.15	3,385.88	....	.89	1.11
	6,483.8	621.86	8,750.30	3.67	.85	1.06
			BOWLDERS.			
1	133.1	...	167.71	3.85	1.23	1.54
2	531.4	....	528.67	14.81	.95	1.21
4	279.2	....	809.17	20.78	.81	1.01
8	392.3	....	193.95	7.52	.67	.84
12	621.8	...	533.95	27.20	.82	1.03
50	2,297.3	..	1,506.69	52.51	.63	.79
	4,834.2	.....	3,740.14	126.67	.75	.94
			BRICK.			
1	304.9	41.78	786.81	15.43	1.76	2.20
2	816.5	117.29	1,691.26	17.63	1.24	1.55
4	1,066.8	149.17	1,360.54	45.51	.92	1.15
8	393.2	55.66	612.79	15.13	.73	.91
12	700.3	97.38	1,078.14	25.26	.78	.98
50	5,720.3	96.59	5,224.20	228.43	.82	1.03
	9,001.1	557.87	11,253.74	347.39	.89	1.11
			GRANITE.			
1	683.0	55 67	1,236 13	18.82	1.38	1.66
2	1.071.1	146.03	2,120.21	33.99	1.20	1.50
4	1,832.8	238.13	3,352.84	51.77	1.07	1.34
8	1,189.7	135.82	1,947.15	43.08	.96	1.10
12	3,193.3	208.56	3,839.75	90.16	.84	1.05
50	29,202.4	727.53	20,547.63	569.95	.56	.70
	37,172.3	1,511.74	32,953.71	802.27	.65	.81
			MACADAM.			
1	344.5	....	331.65	108.02	.65	.81
2	2,080.6	.....	1,412.61	472.43	.45	.56
4	2,926.9	....	1,450.38	474.21	.33	.41
8	619.9	......	341.66	76.55	.43	.54
12	1,127.1	.....	515.33	190.63	.29	.36
50	14,237.6	......	3,809.83	1,110.65	.19	.24
	21,336.6	......	7,861.46	2,432.49	.25	.31
			WOOD BLOCK.			
1	33.5	8.21	92.58	.....	1.39	1.74
2	80.6	13.18	223.18	.....	1.85	2.31
4	123 1	24.93	317.20	8.75	1.06	1.33
8	77.1	16.79	137.34	.....	.56	.70
12	219.1	22.09	377.24	15.05	1.12	1.40
50	1,419.0	.....	1,632.88	44.00	1.12	1.40
	1,952.4	85.20	2,780 42	67.80	1.18	1.48
			BITUMINOUS MACADAM. (Penetration.)			
1	54.0	.....	88.96	18.76	1.30	1.63
2	106.7	.....	133.22	29.51	.97	1.21
4	209.0	.....	237.39	52.00	.89	1.11
8	73.5	.....	77.23	15.25	.84	1.05
12	207.1	.....	244.28	46.75	.95	1.19
50	140.2	.....	111.72	38.00	.53	.66
	790.5	.....	893.80	200.27	.88	1.10

Notes:—(1) Material chargeable to the party to whom permit is issued through material being destroyed or misplaced when cut is made in pavement.

(2) Includes only direct cost of labor and material actually applied on the job, as shown by the foremen's reports, which are made out daily and forwarded to the office.

During the year 1913 all new material was used in mixing concrete, none of the old material being used. The price of this concrete varied from $7 in the small cuts to $6 per cubic yard in the larger size The prices given were arrived at by deducting the c ost of the concrete at the above mentioned prices from the total cost, which remainder represented the actual direct cost of restoring pavement surface.

(3) The same as the preceding column, with the addition of 25 per cent overhead, which represents the difference between the total expenditures of the department and the amount expended directly for labor and material, as shown by the reports of foremen doing the work.

Sub-Surface Maps.—Destruction of pavements can be further reduced by preparing sub-surface maps showing the precise location and character of all structures under the city streets. This is quite a task even now, but every year it grows larger and it is only a question of time when it will have to be done. Brooklyn, New York, and Philadelphia, Pennsylvania, have such maps; Los Angeles, California, is now preparing one, and Cincinnati, Ohio, has taken the first step by compiling sewer record plats of the city. Such a map will enable the proper department to issue excavation permits intelligently and the data recorded on such maps will not only prevent destruction of property but may avoid loss of life.

The Differentiation of Natural and Oil Asphalts.

It is difficult to distinguish chemically, by tests prior to construction, the difference between oil and natural asphalts. Mixtures of the two types cannot be easily detected. Without reference to the relative qualities of the two materials as paving cements it is desirable from an engineering standpoint that some method of determination be developed. A paper by E. C. Pailler of the laboratory of the bureau of highways of Manhattan, N. Y., presented in the "Journal of Industrial and Engineering Chemistry" gives a method of making this determination.

MINERAL MATTER AND FIXED CARBON.

It is well known how great a variation of mineral matter exists in all asphalts. The same is true of the fixed carbon content. Table I shows the average, minimum and maximum amounts found.

Table I shows that if we take a mixture of, say, 50 per cent Bermudez and 50 per cent California, it will, under conditions, be almost impossible to detect the presence of the California, for, assuming the fixed carbon content to be 12.50 per cent for Bermudez and 14.50 per cent for California, the 1:1 mixture will have a fixed carbon content of 13.50 per cent.

Much has been said for and against the fixed carbon determination. Lester Kirschbaum (ENGINEERING & CONTRACTING, Feb. 12, 1913) states that the fixed carbon content is a means of identification of native bitumens. It is true that the fixed carbon content in natural bitumens is clearly defined, the range being less than 2 per cent, and therefore it is valuable as an indication as to the purity of the natural asphalts, even if they are fluxed. Should a Bermudez asphalt show a fixed carbon of 14 per cent or more, I would expect that the sample in question is not straight

Bermudez, and other tests described later would be made. If, however, a petroleum residuum is under consideration, the fixed carbon content as a means of identification is of not much value, although an important significance may be attached to it. Kirschbaum, in his article, treated this subject in an elaborate manner, and it is quite logical to look upon the fixed carbon in oil residuum as an index of the severity of the heat treatment it has received. Unfortunately the fixed carbon in California, Mexican and Texas oil residuums may be the same for all, and it is seen that it would be impossible to draw conclusions from this determination. The method employed in the fixed carbon determination is described in the *Journal, American Chemical Society*, 21 (1899), 1116. To test the accuracy of the method a sample was given to four different chemists, who were instructed to use the same method. The results obtained were as follows:

	Percentages of— Volatile combustible matter.	Fixed carbon.	Mineral matter.
1	83.03	16.77	0.20
2	82.93	16.67	0.20
3	82.80	17.02	0.18
4	82.76	17.08	0.16

L. M. Law (*The Journal of Industrial and Engineering Chemistry*, 5, 1021) quite recently had fixed carbon determinations made on two samples at six laboratories and the results obtained differed 6 per cent on one and 9 per cent on the other sample. However, three results in each sample check very well, and I am safe in saying that these three laboratories evidently employed the same method. The fixed carbon determination is an arbitrary one and has to be carried out to the smallest detail. The burners, crucible, size of flame, etc., have to be the same in all determinations to obtain the right results. Could any one expect to get the same results if, in the sulphonation test, he would use dilute sulphuric acid, and in the other concentrated, or in a moisture determination if he would heat the substance to only 50 degrees C.? The pressure and the composition of the gas has very little effect on the result. This is shown on the three following tests: No. 1 was run with gas at full pres- [illegible] pressure and No. 3 with different gas and pressure at the author's private laboratory.

PERCENTAGES FIXED CARBON.

No. 1	[illegible]
No. 2	17.45
No. 3	17.08

My contention is that a fixed carbon determination is of much value in asphalt analysis in native bitumens as a means of identification, and in petroleum residues as [illegible] the petroleum was subjected to—the former, however, only in conjunction with test described later.

Very little is known of the chemical composition of the bitumens contained in asphalts. They are hydrocarbons of complex structure, largely cyclic and bridge compounds, together with a small amount of their sulphur and nitrogen derivatives, and are closely related to the hydrocarbons occurring in petroleum; as a matter of fact, some writers have advanced the theory that asphalts are the residue of petroleum, the [illegible] natural way evaporated.

SULPHUR CONTENT

The process to convert oil into asphalt, which took ages in Nature, is used in the manufacture of oil asphalts, and is accomplished in a few days. The difficulty in distinguishing between the natural and artificial asphalts, or in proving their presence in a mixture, is due to this fact. Malencovic (*Baumaterialenkunde*, 1906, p. 29) states that oil asphalts and natural asphalts differ in that the former do not react with mercuric bromide solution; the latter, however, owing to their sulphur contents, give a precipitate. This test, however, is of no use, as oil asphalts are very often artificially sulphonated and consequently give the reaction. The sulphur content in oil asphalts and natural asphalts is, if the oil asphalt is not sulphonated, quite different, as the following table shows:

TABLE 1. MINERAL MATTER AND FIXED CARBON CONTAINED IN ASPHALT.

Asphalt.	Percentages of— Fixed carbon— Av.	Max.	Min.	Mineral matter— Av.	Max.	Min.
Trinidad	10.20	10.60	9.80	35.00	38.00	30.00?
Bermudez	12.90	13.50	12.50	4.00	5.00	3.00
California	14.50	16.00	13.80	0.15	0.20	Trace
Mexican	15.50	18.00	[illegible]	0.20	0.30	Trace

PERCENTAGES SULPHUR.

Trinidad	4.35
Bermudez	5-6
California less than	2.0
Mexican	4.0

The sulphur content is determined by igniting the asphalt in an atmosphere of oxygen, precipitating the SO_3 with $BaSO_4$. The ignition of asphalt in oxygen consequently oxidizing the sulphur to SO_3 is more accurate and easier to perform than the oxidation of the sulphur by the Carius method. A number of experiments have shown that it is quite difficult to oxidize all of the asphalt in some cases. In others it was found that the results were too high by the Carius method, to say nothing of the danger of exploding the glass tube. It is not at all unusual to have three or four tubes blown up in a single determination. The method, using Graef's apparatus, is easily performed, requires but little time and the apparatus used is moderate in cost.

As stated before, it is doubtful whether the determination of sulphur is of any practical value as an identification, as many oil asphalts are artificially sulphonated and some oil asphalts contain naturally a considerable amount of sulphur. Kast and Logan (*Journal Society Chemical Industries*, 19 (1900), 505) state that all petroleum, and therefore petroleum residue, except that from Legernsee, contain sulphur (from 0.14 — 1.90 per cent). This sulphur may be present as free sulphur, sulphides or complex sulphur compounds. Maybery and Quayle succeeded in isolating a series of alkyl sulphur compounds by distilling crude Canadian oils at a pressure of 55 mm. Richardson and Wallace have found free sulphur to be the form in which the larger part if not nearly all of the sulphur in Texas oil exists. Some hydrogen sulphide is dissolved in the oil and a large amount is formed if the oil is heated as in distillation. The sulphur content in Mexican oil asphalt is as high as 6.20 per cent. Considering this fact, a sulphur determination alone would not amount to much as an identification of asphalts, especially mixtures or asphaltic cements which are fluxed with oil residuum.

I. Marcusson (*Chemical Zietung*, 36, 84, 801) states that in all natural asphalts there are present a number of organic acids or acid salts, which on distilling are collected in the first few cc. of the distillate and this distillate also contains a certain amount of saponifiable oils. It was this statement which was the basis of this investigation. There was, however, no method given as to the manner of distillation. The collecting of the distillate, and accordingly the results obtained in a rough-and-tumble way did, by no means, give satisfactory results, and the first step was to devise a method which would, under the same conditions, give uniform results. The acids present are organic and are all insoluble in water. They are also not decomposed by heat and the test can safely be carried on at a temperature of 375 degrees C.

The apparatus used is a small retort of 150 cc. capacity, two weighing bottles of 10 cc. capacity and two 8-oz. Erlenmeyer flasks. The reagents are N/10 NaOH, ethyl ether, neutral alcohol of 95 per cent strength and a 1 per cent alcoholic solution of alkaline blue.

Method.—Thirty grams of bitumen or an amount of asphalt which will yield 30 grams bitumen are placed in the retort and heated until it melts. The flame is now raised until the first drop distills and so adjusted that about one drop in two seconds is collected. The distillate is collected in the weighing tubes (the weight of which were previously obtained) in 5 cc. portions and kept separate. After weighing the tubes with the oil, each of the distillates is dissolved in 10 cc. portions of ether. The ethereal solution is then washed with water in a separatory funnel until free from mineral acids which may be present, transferred to the Erlenmeyer flask, diluted with 75 cc. neutral alcohol, 5 cc. of the alkaline blue solution are added and titrated until the red color changes to blue. The acid number is expressed in milligrams of KOH used to neutralize the acid in 1 gram of oil distillate. The following table shows some of the results obtained:

Asphalt.	Acid value. Dist. 1.	Dist. 2.
1 Pure Trinidad	16.20	8.40
2 Pure Trinidad	15.70	7.90
3 Pure Bermudez	9.80	3.20
4 Pure Bermudez	9.60	3.60
5 Pure California	0.11	0.07
6 Pure California	0.19	0.04
7 Maltha flux	0.12	0.25
8 Maltha flux	0.16	0.19
9 Bermudez maltha	9.80	2.30
10 Mexican	0.24	0.05
11 Mexican	0.22	0.09
12 Bermudez + California	4.90	1.20
13 Bermudez + Mexican	5.40	2.00
14 Bermudez + California + maltha	5.50	1.70
15 Bermudez + Mexican + maltha	5.15	1.22

The above table shows clearly that the acid values of natural asphalts are considerably higher than those of oil asphalts. The second distillate in both Trinidad and Bermudez asphalts shows still some acid value, while in California the second distillate is almost free from acids. Therefore, if the acid value falls below 1.0, we can safely say that we are dealing with an oil asphalt or a mixture. If we deal with an asphaltic cement, which will be shown by taking the penetration, ductility and other physical tests, we should expect an acid value of not less than what is given under No. 9. If it is lower, we can expect to deal with a mixture.

TABLE II.—VARIATIONS DETERMINING TYPE OF ASPHALT.

No.	Fixed carbon, per cent.	Mineral matter, per cent.	Acid value. First dist.	Second dist.	Sapon. value.	Conclusion as to nature of asphalt.
1..	12.30	4.90	9.6	3.6	29.8	Bermudez
2..	9.95	35.60	15.9	8.1	36 0	Trinidad
3..	15.85	Trace	0.16	0.08	11.2	California
4..	10.10	Trace	0.12	0.18	7.4	Maltha California
5..	14.60	2.21	4.9	1.2	19.3	Bermudez, 1 : 1
6..	14.15	2.94	4.7	0.9	22.0	California and Bermudez, 3 : 7
7..	[illegible]	0.20	0.24	0.05	9.8	Mexican
8..	[illegible]	0.22	0.22	0.69	10.7	Mexican

The acid number is of great value as a preliminary test, to determine whether we deal with natural or oil asphalts or a mixture. It was also noted that the behavior of the asphalts in the retort, while distilling, indicates the nature of the asphalt. Dealing with natural asphalts we have, as soon as distillation starts, dense yellow fumes, emitting a strong sulphide odor, which are entirely absent in California and other oil asphalts. Oil asphalts, too, distill much easier than natural, but at a higher temperature.

SAPONIFICATION TEST.

Having ascertained the acid value of the asphalt, the next step was to determine the saponifiable quantity. Five grams of ash-free asphalt are dissolved in 30 cc. benzol. This is boiled with 50 cc. *N* alcoholic KOH under a reflux condenser. After cooling to room temperature, 250 cc. 95 per cent alcohol are added and the excess of alkali titrated back with *N* H_2SO_4 using alkaline blues as an indicator. The end reaction is, in spite of the dark color of the solution, very distinct. The flask is now heated again on the water bath and more acid added if the red color reappears. This is to be kept up until the color remains blue. A blank determination is run alongside on the reagents used, to make allowance for the action of the alkali on the glass.

The following table shows the results obtained by the above method:

	Sapon. No
Trinidad	40.0
Bermudez	28.0
California	12.0
Mexican	10.5
Maltha flux	8.0
Bermudez and California, 1 : 1	2[illegible].5
Trinidad and California, 1 : 1	26.0
Bermudez and Mexican, 1 : 1	19.7
Trinidad and Mexican, 1 : 1	24.9

The above shows very clearly the great difference between natural and oil asphalts. The difference of this constant enables one to tell not only the nature of the asphalt, but also gives fairly accurate quantitative results.

It is interesting to note that in asphalts there is such a great difference in these two constants—the acid and saponification value and undoubtedly a further investigation will reveal more distinguishing means between the various brands of asphalts.

The foregoing proves that there are a number of tests which are of great value as single tests. In conjunction with other tests they show conclusively the nature of the asphalt or mixture. Furthermore, if the flux is known a mixture of oil and natural asphalts can readily be recognized, as Table II shows.

Maintenance on New York Roads.—There are now 8,380 miles on the county systems and 3,700 on the state routes in New York state. Annually the state expends an average of $2,000,000 additional to aid towns in improving pieces of 68,000 miles of town highway. Seven-tenths of the agricultural products of the state are moved over these town highways. Two batallions of patrolmen, about 800 men, are employed during the open season of travel, making minor repairs to highways. For the up-keep of state and county highways over $4,750,000 is being expended in 1914. In work by maintenance forces, the department is using approximately 1,200,000 gals. of asphaltic products, 800,000 gals. of tar and tar products, and 200,000 cu. yds. of stone, iron ore tailings, gravel and sand. This is exclusive of 200 repair contracts awarded after competitive bidding.

Bituminous Concrete Using Oyster Shell Aggregate.

(Staff Article.)

Near the Atlantic and Gulf seaboards where broken stone for road metal is scarce and expensive, oyster shells can be obtained at a low cost and have been used to a large extent in the construction of roads. They have proven reasonably durable, although producing a good deal of dust in dry weather, owing to the comparatively rapid grinding up of the shells under traffic. In order to remedy this, various bituminous binders have been employed with the shells as they have been with other aggregates.

An interesting example of the use of a bituminous surface using oyster shell aggregate may be seen on the Beckwith Neck Road connecting Cambridge and Hillspoint in Dorchester County, Maryland. The stretch of road improved, located about one mile beyond the town of Cambridge, was laid in five continuous sections, each section containing a different mixture, as follows: (1) Whole shells, crushed shells and bitumen. (2) Whole shells, crushed shells, sand and bitumen. (3) Whole shells, sand and bitumen. (4) Crushed shells, sand and bitumen. (5) Beach gravel, screened and graded with coarse sand, fine sand and bitumen (a check section for use in determining the relative wear). A total length of 506 ft. was surfaced, the width surfaced being 14 ft.

Fig. 1. Shell Surface Prepared for Surfacing.

Fig. 2. Laying and Rolling Warrenite Surface.

The type of bituminous concrete used is known as Warrenite (an improved type of bituminous concrete which is patented) and the surface was mixed and laid by the Warren Brothers Company. The mixing and heating plant used was built by the same company, being especially designed for the laying of bituminous concrete in places at a considerable distance from railroads.

The surfacing was placed on an old oyster shell road. On one section a depth of 3 ins. was loosened by means of picks in the wheels of a steam roller. This loosened material was removed, screened into graded sizes, mixed in the desired proportions, heated and mixed with sand and bituminous cement. It was then laid while hot and rolled. On the other sections new shell aggregate was used, the mixed surfacing being laid directly on the old roadbed. The methods of mixing and laying were uniform throughout the work.

This surface was laid in the early summer of 1911. When inspected in October, 1913, the general condition of the surface was found to be good. In the sections where whole shells were used some of the whole shells which were near the surface with their flat sides up were cracked. The most satisfactory section was the one in which shell crushed to pass a 1-in. ring, sand and bitumen were used.

Hints to Road Superintendents on Preparing Bridge Plans.—Hints for the preparation of bridge plans by Clifford Older, bridge engineer for the Illinois Highway Commission, are given in a recent publication of that commission: Be sure to send the plans in duplicate or the checking will be delayed until duplicates are sent. Be sure that a title indicating the name and location of the work is shown on the plans. Be sure that the plans indicate the depth the foundations are to go below bed of stream, and show reference to a bench mark, so there may be no question as to the correct elevation of all parts of the situation. It is suggested that, as far as possible, drawings be made on sheets measuring between trimming lines 22x28 ins. and between margin lines spaced 21x27 ins. Sheets of this size when folded twice each way are about the size of standard letter paper (8½ ins. by 11 ins.). The specifications and proposal sheets are also of this size. Place the title of the drawing near the lower right-hand corner of the sheet and fold the prints so that the title shows. This avoids the necessity of writing the title on the back of the print or spreading the print out to discover the name and location of the work represented. On tracings of plans for a bridge which may be suitable for some other structure in the future, it is often convenient to omit the title from the tracing. The title may be lettered on another piece of tracing cloth and laid over the detail tracing when the blue prints are made. The prints will then show the title. Special notes or dimensions may be printed in the same way, and the detail tracing or "standard" may be used for any number of different bridges.

Concrete Cube Tests for Concrete Roads in New York.—One of the most important changes of type of road called for by the standard specification of the New York Highway Department is that of the first class concrete pavement where a mixture of 1 part of Portland cement, 1½ parts of sand and 3 parts of stone is called for. In these requirements and in the other directions of this specification the type of concrete road which has proved generally to be a success as built in other parts of the country is followed. In this connection it may be noted that the breaking tests on concrete cubes, formed from the material as delivered from the mixer, have given some very satisfactory results. Six-inch cubes, 28 days old, have been broken under pressure in the testing machine, generally at from 3,000 to 4,000 pounds per square inch, and in some instances have run as high as 4,500 to 5,000 pounds, the higher figure being the limit of pressure exerted by the machine. In crushing, these cubes have generally shown rupture of the stone aggregate, thereby indicating the great strength of the mortar. Since uniformity of strength in the concrete is most desirable in these pavements, the fact that the cubes on individual roads show in the test fair uniformity in the breaking stress indicates success on this line.

BOOK REVIEWS

Scales for Ascertaining the Dimensions of Pipes, Drains and Sewers.—By C. E. Housden. Longmans, Green & Co., New York City. Cloth, 4½x7 ins.; 16 pp.; illustrated; 60 cts.

The scales shown in this work are taken from the author's work entitled "Water Supply and Drainage Systematized and Simplified." The use of these scales will give, for any coefficient selected, the dimensions to the nearest inch of pipes or half-pipes, and of any design of drain or sewer in which a circle or semicircle can be inscribed. The author illustrates six types of drain sections, each made up of straight lines or arcs of circles, or both. The computation by means of the scales of the dimensions of these sections for any desired carrying capacity and with the selected coefficient is explained. Typical problems are solved, all steps in the solution being shown.

This is a handy little book for the engineer who has numerous hydraulic computations to make.

Influence Diagrams.—By Malverd A. Howe. John Wiley & Sons, New York. Cloth, 6x9 ins.; 65 pp.; illustrated; $1.25.

The book treats of influence diagrams for the determination of maximum moments in trusses and beams. The author states that its object is to bring attention to the fact that for loads on all ordinary trusses the influence diagrams for bending moments are drawn by following a single simple rule, and that the diagrams so constructed require no computations for their direct application. The influence diagrams for loads on continuous trusses, cantilever trusses and arches are shown to be based upon the one general diagram for simple trusses. The use of influence diagrams in determining the position of wheel loads which produce maximum moments is also explained. The author's treatment of influence lines is both direct and simple.

Chapter I treats of the use of influence diagrams in determining the maximum moments in simple trusses of various types; Chapter II considers double intersection trusses; Chapter III considers continuous trusses; Chapter IV treats of arches with open and solid webs; and Chapter V shows the application of influence diagram to the determination of the bending moments in beams [illegible]

Mechanical Properties of Wood.—By Samuel J. Record. John Wiley & Sons, New York. Cloth, 6x9 ins.; 160 pp.; illustrated; $1.75.

The book was written primarily as a text for students of forestry. The mechanics involved are reduced to the simplest terms, without reference to higher mathematics. The author has attempted to avoid technical language and descriptions in order to make the subject matter readily available to anyone interested in wood. The book also contains data which should prove of value to those engaged in designing wooden structures.

The text is divided into three parts: Part I discusses the mechanical properties of wood the relation of wood material to stresses and strains. Much of the material in this section is merely elementary mechanics of materials, although the tabular data are of considerable value.

[illegible] the mechanical properties of wood. The author attempts to answer questions concerning the effect of the rate of growth on the [illegible] cutting, of locality of growth, weight, water [illegible]

Part III describes methods of timber testing, the methods described being for the most part those used by the U. S. Forest Service.

In the appendix there is reprinted the working plan followed by the U. S. Forest Service in the extensive investigations covering the mechanical properties of woods grown in the United States. The text closes with a bibliography of important publications on the mechanical properties of wood, and timber testing.

Modern Tunneling. With Special Reference to Mine and Water Supply Tunnels. By David W. Brunton and John A. Davis. New York. John Wiley & Sons. Cloth, 6x9 ins.; pp. 450; illustrated; [illegible]

This volume deals almost exclusively with mine tunnels and tunnels for other purposes which have essential features practically identical with mine tunnels. Soft ground and subaqueous tunneling and tunneling for railways are not considered except in the respect that certain tunneling problems are common to all kinds of tunnels and these of course are considered. The work is notable among books on tunneling in being modern. There is very little space given to the history of tunneling and still less space is given to descriptions of long ago tunneling operations. Besides being praiseworthy for these eliminations, the book deserves commendation for its collection and tabulation of tunneling data. There are tabulations of drilling speed reported at various tunnels; cost of drill repairs; data concerning tunnel cars; operating cost of gasoline haulage; means of illuminating various tunnels; number and depths of holes used in driving tunnel headings; dynamite used at various tunnels, and costs of perhaps a score of tunnels. This book within the limits of the field covered is of noteworthy practical value. The schedule of chapters is: History of Tunneling, 5-34; Modern Mining and Water Tunnels, 35-52; Choice of Power for Tunnel Work, 53-79; Air Compressors, 80-110; Ventilation, 111-124; Incidental Surface Equipment, 125-129; Rock Drilling Machines, 130-162; Haulage, 163-180; Incidental Underground Equipment, 181-209; Drilling Methods, 202-225; Blasting, 236-259; Methods of Mucking, 260-269; Timbering, 270-288; Safety, 289-327; Cost of Tunnel Work, 328-359; Bibliography, 421-426.

Symmetrical Masonry Arches.—By Malverd A. Howe. John Wiley & Sons, New York. Cloth, 6x9 ins.; 241 pp.; illustrated; $2.50.

This is the second edition of Professor Howe's book, the first edition appearing in 1906. Much of the text has been rewritten and considerable new matter has been added. In this edition an effort has been made to simplify the demonstrations of the formulas. The book considers the design of stone, plain concrete and reinforced concrete arches, the solutions being based on the elastic theory. In general the unit load method has been employed, although a method is given by which the effect of certain fields of loading can be found directly.

Chapter I gives a development of the fundamental formula used in the solution of arches; Chapter II gives the formulas applicable to symmetrical arches fixed at the ends and loaded with vertical loads; and Chapter III treats of symmetrical arches fixed at the ends and loaded with horizontal loads. Chapter IV gives the application of the formulas to the solution of both a stone arch and a reinforced concrete arch. Chapter V illustrates and describes a few typical arches. These five chapters cover the first 144 pages of the text. The remainder of the book consists of five appendixes which give data to facilitate the design of arches. Appendix A gives some data on the physical properties of stone and also gives tabular data for about 600 masonry arch bridges arranged according to span; Appendix B discusses arch coefficients and gives tables of coefficients for 20 arches of different proportions; Appendix C gives formulas for symmetrical fixed arches when the origin of coordinates is taken at the crown; Appendix D gives an exceedingly brief treatment of unsymmetrical fixed arches; and Appendix E records some data on the internal temperature range of concrete bridges.

Engineering as a Profession.—By A. P. M. Fleming and R. W. Bailey. John Long, London. Cloth, 5x7 ins.; 280 pp.

This very interesting British book was prepared primarily for the purpose of setting forth the facilities that actually exist for obtaining the most satisfactory engineering training and subsequent engineering employment in Great Britain. Since it is designed largely for the guidance of parents and guardians who are investigating, on behalf of sons or wards, opportunities in the practice of engineering, the book first gives a broad general outline of the field of engineering activity. Comparisons are drawn between engineering and the older professions with respect to qualifications essential to success, cost of preparation, probable income, etc. The authors' conception of the personal characteristics of importance in engineering makes interesting reading. They say: "He should have an aptitude for mathematics and natural sciences, as well as a taste for mechanical contrivances, and, in addition to the qualities already referred to as being essential to success in any profession (sound moral character, strength and honesty of purpose, sound judgment, perseverance, and ability to co-operate with fellow-workers), the engineer should have the keen business instinct, the ability to argue logically, and to be able to set forth clearly and concisely his views on any problem he may have to consider. He should have a good physique. Robust health is probably more essential in engineering than in most other professions."

Modern methods and facilities for obtaining an engineering training are very fully discussed and information is given in sufficient detail to form a dependable guide to the student in the selection of a course of training, either practical or academic.

The authors apparently believe that the student at the outset of his studies can have a very definite conception of the position he hopes ultimately to secure, for the book endeavors to outline the most direct course from the novice to his goal.

The book also gives information for the guidance of foreign youths who go to England for their engineering training. Engineering opportunities in foreign countries are discussed. There is also a comparison of the methods of training engineers in vogue in England, Germany, the United States and other countries.

The American engineer who reads this book with particular attention to what it states and infers with reference to the material status of engineers abroad is quite likely to conclude that he is most fortunately situated for engaging in the practice of engineering. Thus we read that in Great Britain unless an engineer can afford to buy a partnership in a firm of consulting engineers he must expect to support himself without return for from five to seven years.

On the whole the book is exceedingly interesting to students of engineering and of the engineer. Obviously its greatest value to the former will be to those who live or expect to practice in England.

Mileage of Different Types of Roads Under Contract in New York.—Since May, 1913, the New York State Highway Department has placed under contract a total of 1,003.12 miles of highways. These contracts are made up of the following mileage under different types of construction: Waterbound macadam, 286.90; waterbound macadam, hot oil, 10.87; waterbound macadam, cold oil, 10.06; bituminous macadam, 352.31; concrete, 217.13; concrete, bituminous top, 8.34; brick, 90.53; asphalt block, 6.83; stone block, 1.93; other types, 18.22.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

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An Improvement in the Methods Commonly Used in Constructing Retaining Walls for Buildings.

The construction of retaining walls for the deep basements of large buildings, always an expensive and hazardous operation, is especially troublesome in soils such as are encountered in Chicago. Various methods have been developed to cheapen and expedite retaining wall construction, but the methods commonly used are tedious and slow. The danger of constructing such walls without providing adequate foundations for them was strikingly illustrated in a recent failure in Chicago. The enormous pressures exerted at the depths to which the sub-basements are often carried requires a heavy bracing of the trench, which greatly interferes with the construction operations. The necessity of preventing large settlements of the floating foundations of adjacent buildings, occasioned by the flow of material into the excavation, requires that the bracing of the trenches be such as to preclude any possibility of displacement. Due to the type of bracing and the construction methods commonly used for deep retaining walls it has been necessary to concrete the walls in comparatively thin horizontal layers and to allow each layer to harden before the next one is concreted. This has made it difficult to reinforce the walls against horizontal shear, and has made the use of rods for reinforcement generally unsatisfactory.

To overcome some of the difficulties encountered in retaining wall design and construction, the architects and engineers for the Lumber Exchange Building, Chicago, Ill., have developed a construction method which permits the entire section of the retaining wall between columns to be concreted in one operation, thus insuring monolithic construction and rendering the problem of reinforcing the wall a comparatively simple one. The method also permits the drums and other bracing to be left in place until the entire wall has hardened sufficiently to resist the earth pressure. This greatly lessens the possibility of failure of the wall and of danger to adjacent structures. The article appearing in the "Buildings" section of this issue explains and illustrates the details of the new method, which was successfully used in constructing the 50-ft. retaining walls of the Lumber Exchange Building. In this building the construction work was especially difficult due to the fact that the buildings on three sides have floating foundations, the spread footings of one of which—a 16 story structure—project a considerable distance into the site of the new building.

Why Was "Interest During Construction" on the Panama Canal Omitted From All Cost Estimates?

Now that the Panama Canal is finished we are able to calculate the item of interest on the annual expenditures. In round numbers this "interest during construction" amounts to $50,000,000, assuming the interest rate to be 3 per cent. The total expenditures charged against the canal amount to $325,000,000 up to May 31, 1914, exclusive of interest during construction, but inclusive of the $40,000,000 paid to the French company and the $10,000,000 paid to the Panama Republic.

Not in any of the estimates of cost of the canal, nor in any of the published statements of its actual cost, is there a cent of "interest during construction." Hence a huge item of construction cost—$50,000,000—has received no mention thus far in the reports relating to the canal.

According to the accounting requirements of the Interstate Commerce Commission, railway companies must charge "interest during construction" to a construction account designated for this purpose. It needs but scant consideration to see the reason why such interest payments are properly part of the construction cost of a private enterprise. The very same reasoning justifies charging public works with interest during construction if the full cost is to be known.

When public works are built with money secured from bond issues it is more apparent that the bond interest is part of the construction cost than is the case when the money is secured by direct appropriations from funds raised by taxation. Nevertheless there is no inherent difference. The loss of income on an investment during a construction period is an element of construction cost, irrespective of the source of the invested capital.

For one reason or another, engineers have not always included interest charges in their estimates of construction cost, but no engineer attempts to justify such an omission on the ground that "interest during construction" is not a cost incident to the building of the property. It is somewhat astonishing, therefore, to see no mention at all of an item that totals fully 16 per cent of the cost of the plant and other property relating to the Panama Canal.

The cost of the Panama Canal has not yet ended, nor will it end for several years. Following the 10-year period of construction will come the "development period" which may greatly exceed 10 years. The "development period" will last until the annual net earnings on the canal equal the interest on the investment, which investment must include the accumulated annual deficits. The annual deficit is here used to mean the amount by which the annual interest exceeds the annual net earnings.

It is not easy to predict what these deficits of "development cost," will total, but that they will exceed $50,000,000 is quite certain. Much will depend on the toll rates.

Sewage Disposal in the Next Decade.

Within the next decade many millions of dollars will be expended for the interception and treatment of the sewage of large American cities. The article on Milwaukee sewerage, which is published this week, points out, incidentally, that only a very few of our principal cities have squarely faced the sewage disposal problem in the light of modern science and of community responsibility. Among the larger cities only Boston, Providence, Columbus, Baltimore, New Orleans and Atlanta have works which square with modern ideas relative to the proper disposal of sewage. The problem is still a live one in Chicago and sewage treatment for this city is sure to come. New York, Philadelphia, Cleveland, Detroit, Cincinnati, Rochester, Milwaukee, Indianapolis and Dayton are all studying the treatment problem. Great works for the collection and disposal of the sewage of these cities will doubtless be constructed within the next ten years. The same may be said with reference to numerous secondary cities in all parts of the country.

While improvements in the art are to be expected and doubtless will be realized, experts believe that such advances will occur chiefly in details of design and as a result of more faithful and competent supervision of plant operation. Some experts have directly committed themselves upon this point. Others have given evidence of holding these views in recommending the construction of works, costing millions of dollars, based upon the methods now in vogue. Moreover, many of the larger undertakings now under serious consideration will require so long in building that developments in the art occurring within the next four or five years will be available for use before the actual construction of many of the larger treatment works is reached. In a word, the art has progressed to a point where great expenditures may safely be made in its application without fear of the early obsolescence of works.

Two considerations which have grown steadily in favor among engineers, sanitarians and laymen, during the past two or three years, will henceforth supply a great stimulus to sewage treatment activities. The one is that even when a public water supply is filtered, the filter should not be overloaded, its limitations should be kept in mind and, consequently, it must have a raw water of fair quality to work upon. The other consideration to which we refer is the growing conviction that a self-respecting community cannot afford to pollute natural bodies of water with its untreated sewage.

The Re-Surfacing of Worn or Obsolete Macadam Pavements on Country Roads.

A worn or obsolete macadam pavement, if honestly constructed, is an asset and has a considerable value as a base for a type of surface better suited to traffic markedly destructive to macadam. It has been frequently pointed out in this journal that the economic life of a macadam road is by no means limited to the time it serves as a wearing surface. After the period of usefulness as a wearing surface has passed the pavement still retains a large portion of its value. This value, however, is as a foundation or base, not as a wearing surface.

A finished macadam road surface costs about one-fourth more than a crushed stone base of equal thickness for a brick pavement or any type of bituminous pavement. The value of the old macadam should be fully equal to that of a newly laid base. As a matter of fact it is worth more. The wholesale ripping up and discarding of the material contained in old gravel or crushed stone macadam roads simply because it is desired to use a different type of wearing surface is folly and, in many cases, pure extravagance.

A relation undoubtedly exists between the type and amount of traffic a road bears and the proper surfacing material to use. The folly of constructing a brick or concrete surface upon a road on which the traffic will always be light is as great as it would be in building a double-track railroad to some isolated Smithville that ships but a few cars of freight a year. The excellence of the road is unquestioned, the expediency of its construction at all is the matter in point. Many earth roads at present in use should properly remain earth roads for a long time. With the demand for improved service on heavily traveled roads it simply will not pay to improve them.

A type of construction economical in that it makes use of the road already built is described on page 357 of this issue. Asphaltic concrete has been successfully laid on a macadam base for many years, notably by the engineering department of the District of Columbia and also by the Warren Brothers paving company. There is no question of the success of this type of construction. The attractive feature, however, is its low cost. The cost of an asphaltic concrete surface 2 ins. thick should not exceed 70 cts. a square yard.

A condition that has, heretofore, made difficult the adoption of this type of surface for rural roads has been the expense of long hauls and the cost of the plant necessary to properly prepare the asphaltic concrete. The increased efficiency of motor trucks and their proved utility for hauling over country roads have, however, materially changed this condition. By using motor trucks for hauling the effective radius of operation from one location of the mixing plant is increased from about 4 miles, the distance economical with teams, to about 10 miles, length of economical haul with motor trucks. Moreover, the time of travel is much reduced, permitting the hot material to be delivered at the temperature favorable to successful manipulation in placing.

In some sections brick and stone block pavements are laid on an old macadam base. Many miles of brick pavement have been so laid. While on new construction a concrete base is distinctly preferable, if a well compacted macadam road already exists it is, undoubtedly, better economy and good engineering under many conditions of light traffic to use the old pavement as a base for the new.

A commendable tendency to make more and better use of old pavements is exhibited in the taking up and re-dressing of granite blocks, the development of macadam resurfacing methods, the turning of brick pavements at a moderate cost, in one case, 35 cts. a square yard, and in the standardization of concrete base construction. The tendency to increase the size of the scrap heap should be discouraged. Pavement scrap heaps already in existence contain too much material that has not outlived its useful life. Because a small portion of the thickness of a pavement has been worn away there is no excuse for the wholesale scrapping of the pavement.

WATER WORKS

Construction of Water Works Tunnels in the Metropolitan Water District of Massachusetts.

III.

Cost of Extending, by Day Labor, the 24-in. Water Pipe Tunnel Under North Channel of Mystic River at Chelsea Bridge.

Contributed by William E. Foss, Assistant to the Chief Engineer, Metropolitan Water and Sewerage Board, Boston, Mass.

THE OLD TUNNEL.

The work of widening and deepening the draw at the Mystic River bridge between Charlestown and Chelsea, in 1900, made necessary changes in the pipes which were laid at that point as inverted siphons under the channel. The great depth and width of the draw made the use of a siphon impracticable, and a tunnel 6 ft. in diameter and 145 ft. long was constructed, in 1900 and 1901, in which a line of 24-in. pipes were laid. A brief description of the construction of this original tunnel is here given as introductory to the description of its extension in 1912.

About 45 ft. of each of the two shafts of the original tunnel was composed of steel cylinders 8 ft. in diameter, made of ½-in. plates. Three 9-ft. lengths of cylinders were set up on the deck of a lighter and partially lined with brick masonry. The 27-ft. section of the shaft, weighing about 40 tons, was then lowered into position and allowed to settle into the river bottom. Two additional sections of the cylinder were then bolted on and lined with brickwork. When this had been done, an air lock was bolted to the top of the shaft, the water forced out by compressed air, and the construction of the shaft and tunnel carried on under an air pressure of from 20 to 29 lbs. Below the bottom of the steel cylinders the shaft was lagged with circular wooden lagging 5 ins. in thickness; and when the required depth was reached, Portland cement concrete, about 3 ft. in thickness, was put in the bottom of the shaft, and the shaft was then lined with brickwork to the bottom of the lining already in place. A circular steel shield about 9 ft. in diameter and 4½ ft. in length was used in excavating the horizontal portion of the tunnel. The shield was forced ahead by means of hydraulic jacks as the material was excavated, and circular wooden lagging, 5 ins. in thickness, cut from 2-in. spruce planks and having an inside diameter of 8 ft. 2 ins., was placed in position as the shield advanced. When the lagging had been advanced from 8 to 10 ft. beyond the completed masonry, it was lined with brickwork and washed with cement.

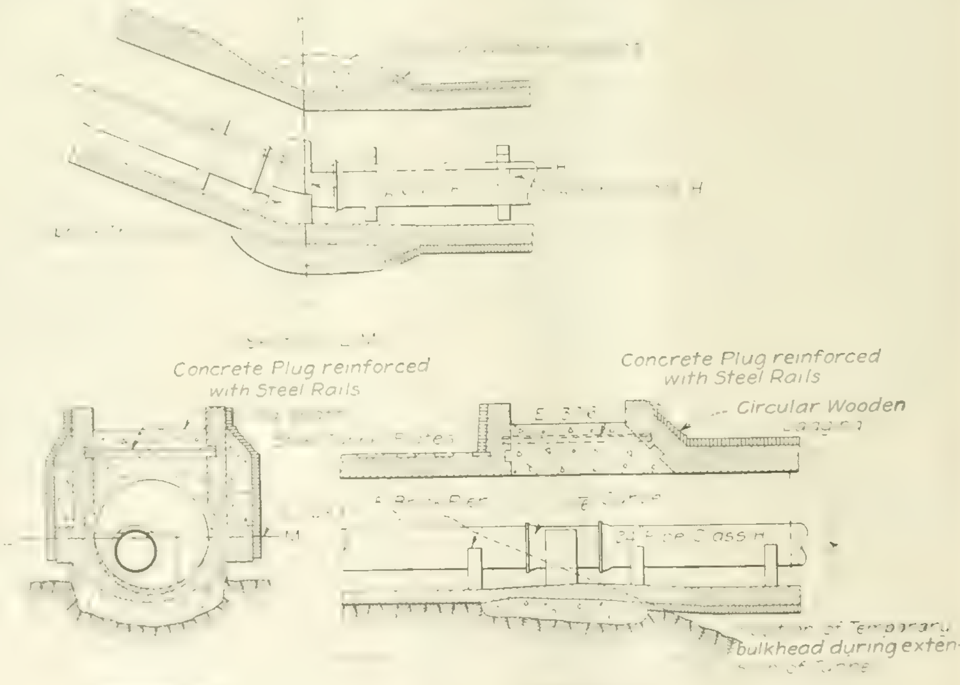


Fig. 2. Sections at Angle in Mystic River Water Tunnel at Old South Shaft.

Where the tunnel was built there is a depth of from 15 to 20 ft. of water at low tide. In sinking the shaft on the Chelsea side of the channel, the material excavated was mud for a depth of 3 ft., clay for a further depth of 11 ft., and then sand and gravel for the remaining distance of 21 ft. The horizontal portion of the tunnel was excavated entirely through sand and gravel, containing some small boulders and a few pockets of clay. Much difficulty was experienced in preventing the escape of air and consequent flooding of the work; and when the drift was about 26 ft. from the shaft a blow-out occurred, and work was delayed for a week. The shafts were surrounded by guards of oak piles securely fastened together with hard pine timber and bolts at their tops and at a point a few feet above the low-water mark. Both shafts and tunnel were 6 ft. internal diameter, with walls of 12-in. brick masonry laid in Portland cement. The shafts were 62 ft. in depth and 139½ ft. from center to center. The leakage into the tunnel after the work was finished averaged about ½ gal.

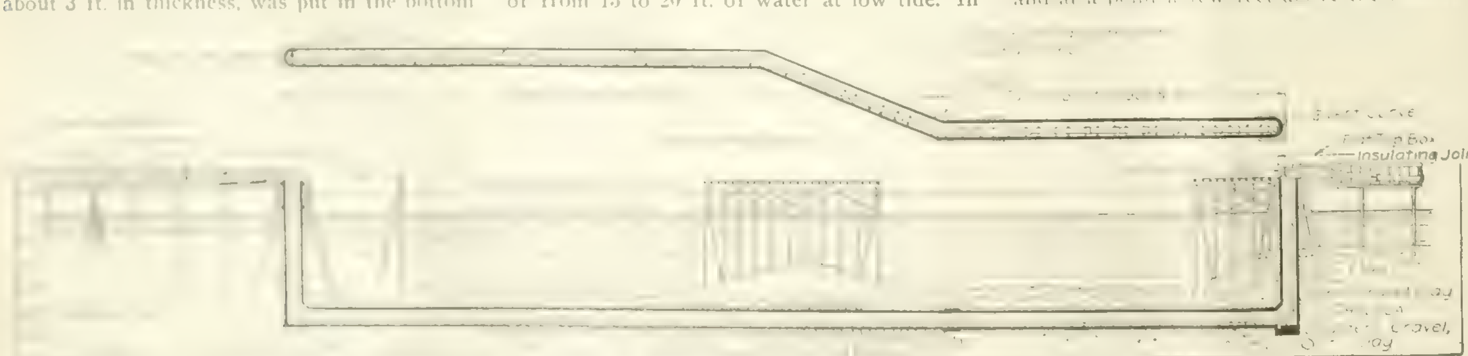


Fig. 1. Horizontal and Longitudinal Sections of Water Tunnel Under Mystic River, at Chelsea Bridge, Joining Charlestown and Chelsea, Mass.

per minute. The total cost of the work, including the reconstruction of the wooden boxing at each end and repairing fender guard, but not including the cost of laying the pipes, was $21,850.

EXTENSION TO MYSTIC RIVER TUNNEL IN 1912.

The tunnel was extended, in 1912, a distance of 273 ft. southerly from the old shaft on the river. The shaft is of the same interior diameter as the tunnel and is protected for 38 ft. on the upper end by a steel casing 8 ft. 2 ins. in internal diameter and ½-in. thick, which extends about 15 ft. below the river bed. The shaft is 57.7 ft. in length, with its top about 5 ft. above mean high water, and is protected by a circular fender guard composed of 42 oak piles securely framed and bolted together. A

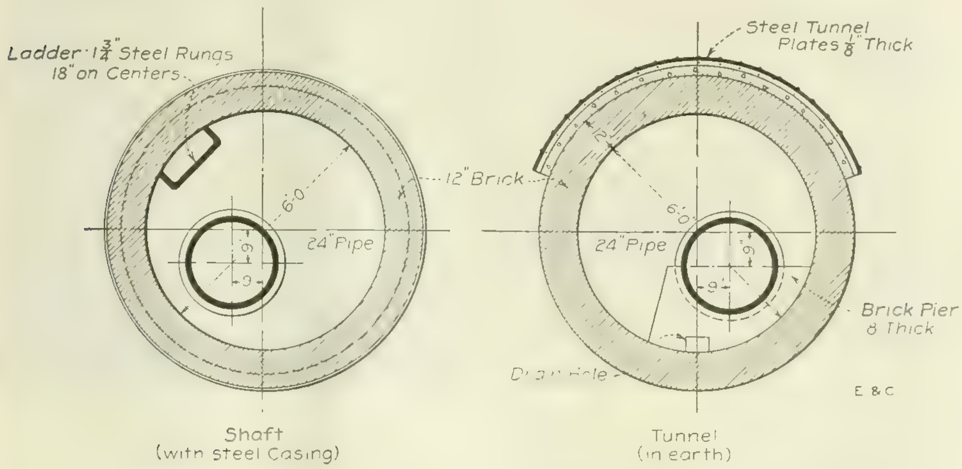


Fig. 3. Typical Sections of Shaft and Tunnel at Charlestown Shaft, Constructed in 1912.
Tunnel in rock constructed with 8 in. brick walls.

Charlestown side of the former channel, in connection with the widening of the channel and rebuilding of the bridge. The total length of the new tunnel and shaft is 331 ft.

The work was begun March 8 and was completed Sept. 11, 1912. It was done by the pneumatic process, the pressure varying from 19 to 27.5 lbs. per square inch. The work was carried on continuously with a day labor force working in three 8-hour shifts per 24 hours.

The old tunnel, which was 140 ft. in length between the shaft on the Chelsea side and the shaft on the Charlestown side of the ship channel, was located about 10 ft. from and parallel with the upstream side of the old bridge and, as above stated, was constructed in 1900 and 1901.

The extension began at the old Charlestown shaft, which was removed after the new tunnel was completed, and from this point the new tunnel deflects to the right from the line of the old tunnel through an angle of 22° 30′, and after extending for a distance of 75 ft. it deflects to the left through the same angle and runs parallel with and 30 ft. from the upstream side of the new bridge for a distance of 198 ft., to the new shaft. Horizontal and longitudinal sections of the tunnel are shown in Fig. 1. The sections at the angle, where the old South shaft was located, are shown in Fig. 2.

The tunnel is 6 ft. in interior diameter with a 12-in. wall of brick masonry, except at places in solid rock where the wall is 8 ins. thick. Typical sections of shaft and tunnel are shown in Fig. 3. The center line of the tunnel is about 43.5 ft. below mean low water in the plan and section of the pile guard are shown in Fig. 4. This fender guard, three temporary dolphins and a temporary platform at the shaft were constructed by the contractor who was rebuilding the bridge for the city of Boston, and the cost of this work is not included in the data here given.

Rock was encountered in the bottom of the excavation for a distance of about 200 ft., and for about 130 ft. the entire heading was in rock. The remainder of the excavation was in sand and gravel, with some boulders.

The 24-in. pipes in the old tunnel were found to be in poor condition below high water and have been entirely replaced. In relaying the pipes the curves at the bottom of the shafts were embedded in concrete and the pipes through the tunnel supported on brick piers 8 ins. in thickness, spaced about 6 ft. apart. The manhole curve at the top of each shaft is secured to the curve at the bottom of the shaft by two 1½-in. diameter vertical rods, and to the pile foundation by two 1½-in. rods carried back horizontally about 60 ft. For the purpose of preventing the deterioration of the pipes by the action of salt water and electrolysis they were covered with a ½-in. coating of slaked lime, which was in turn covered with a 1½-in. coating composed of equal parts of Portland cement and sand. These coatings are shown in Fig. 5. The section of the pipe box on the land side of the new shaft is shown in Fig. 6.

The work of setting up the boilers, air compressors, electric light plant, hoisting engines, pumps, etc., was begun on March 8, and during the week ending March 23 the water was pumped out of the old tunnel, the old pipes removed from the shaft and a brick bulkhead 24 ins. thick built into the tunnel about 12 ft. from the shaft. An air lock was then bolted to the top of the shaft and on April 1 the air pressure was applied. The brick lining was then removed at the bottom of the shaft and the work of driving the tunnel extension began on April 8. It is interesting to note that while a circular steel shield (see Fig. 7), and wooden logging were used in 1900, steel roof plates (see Fig. 8), have been used and the wooden logging has been omitted in all subaqueous tunnel work since that date.

Rock was encountered in the lower part of the heading and rose as the heading advanced until at a distance of 24 ft. from the center of the old shaft the tunnel was entirely in rock and so continued for a distance of 200 ft. The work of lining the tunnel with brick was commenced on April 13 and both excavation and lining were carried forward at the rate of about 2 ft. in 24 hours until July 17, when the brick lining had been advanced 206.5 ft. beyond the old shaft. A brick bulkhead was then built near the end of the finished brickwork and the lined portion of the tunnel cleaned, plastered with cement mortar and washed with cement grout. A concrete bulkhead reinforced with steel rails was then built into the old shaft, above the tunnel, as shown in Fig. 2, and on July 25 the removal of the brickwork of the old shaft was commenced. When this had been partially done an attempt was made to raise the steel casing of the shaft with its brick lining by means of a powerful lighter, aided by compressed air in the shaft. This proving unsuccessful the shaft was left in place until the piles surrounding it were removed, and on Nov. 30 it was raised by the lighter of the Merritt-Chapman Co. (see Fig. 9), and placed on the pier at the Naval Hospital near by. The brickwork was then removed from the casing and the steel sold for junk.

The circular guard of oak piles surrounding the new shaft was built by the George T. Rendle Co. between April 20 and May 18. The steel shaft casing, composed of four sections, each 9½ ft. long, was set up on the lighter and two of the sections were lined with brick. On May 20 the casing was lowered into position by the use of the lighter. The brick lining was then completed. On August 9 the hoisting engine and air lock were removed from the old shaft and set up at the new shaft. The air pressure was applied to the new shaft on August 10 and t' e brick lining of the shaft completed to the finished grade on

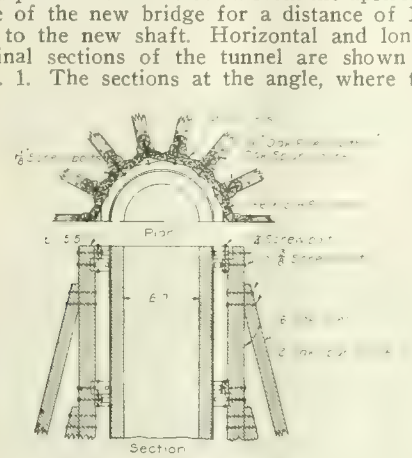


Fig. 4. Plan and Section of Pile Guard to Protect Shaft.

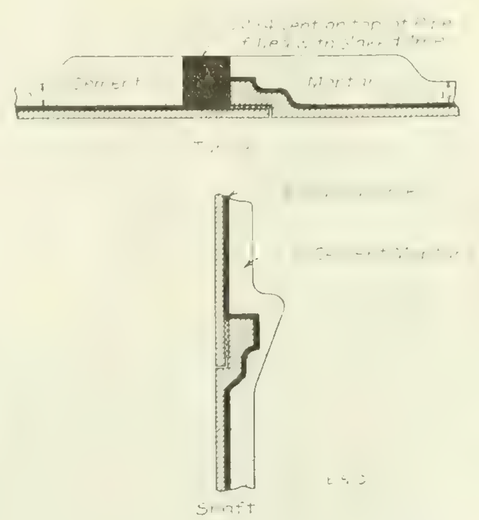


Fig. 5. Section of Lime and Mortar Coating Over All Pipes in Tunnel and Shaft.

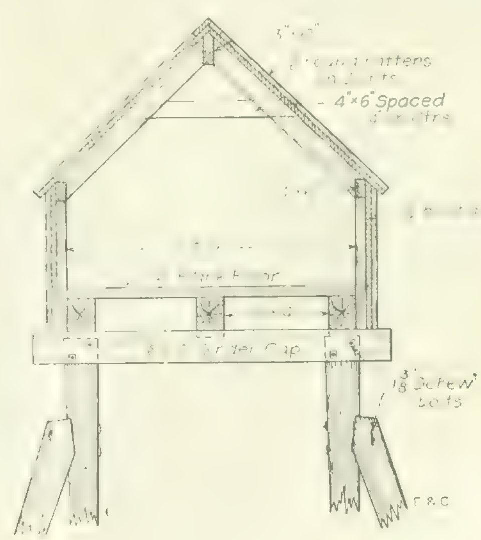


Fig. 6. Section of Pipe Box.

August 16. The plant used at the new shaft is shown in Fig. 10.

Quicksand was encountered at the bottom of the shaft. The bottom was covered with a flooring of 2-in. plank on which was placed Portland cement concrete 18 ins. in thickness. Tunnel excavation from the new

Fig. 7. View of Hydraulic Shield Used in Driving Original Mystic River Water Tunnel in 1900.

shaft was begun Aug. 16 and the tunnel lining was finished Sept. 1. The air pressure was removed on Sept. 3 for a short time for the purpose of testing the tightness of the masonry. On Sept. 6 the air pressure was finally taken off. The work was carried on continuously day and night, a force averaging 16 men being employed on each of the three shifts. In order to comply with the law limiting the hours of labor to 48 per week, and at the same time carry on the work continuously, additional laborers and machinists were employed. The pipe line was finished on Nov. 30 and placed in service on Dec. 7.

The following prices were paid for labor:

[illegible] per week [illegible] days	[illegible]
[illegible] per week [illegible] days	17.00
Head [illegible] per week [illegible]	[illegible]
Miner, per [illegible]	[illegible]
Miner, [illegible]	[illegible]
Lock tender, per [illegible]	[illegible]
Topman, per [illegible]	[illegible]
Laborer, per [illegible]	[illegible]
Laborer, per [illegible]	[illegible]
Blacksmith, per [illegible]	[illegible]
Mason [illegible] per [illegible] day	[illegible]
Mason (piece work, [illegible]), per linear foot of tunnel	2.60
Mason (piece work, shaft), per linear foot of shaft	[illegible]

The following prices were paid for materials:

[illegible] per M	[illegible]
Portland cement [illegible]	[illegible]
[illegible]	[illegible]
Dynamite, [illegible]	[illegible]
[illegible]	[illegible]

The [illegible] follows:

	value when new.
1 [illegible] 14x24-in.	
1 Air [illegible]	
1 75 HP. [illegible] ins. x 13 ft.	$2,100.00
1 Air receiver, 3 ft. x 7 ft	[illegible]
1 Shaft [illegible]	[illegible]
1 Knowles [illegible] pump	[illegible]
1 Worthington [illegible] duplex pump	[illegible]
1 Edson pump, [illegible]	[illegible]
1 Head house wheel, [illegible] ft. diameter	[illegible]
1 12-HP. [illegible] dynamo engine	[illegible]
1 Generator, [illegible] amperes, 110 volts	[illegible]
1 Two drum, [illegible] HP., double [illegible] hoisting engine and boiler	[illegible]
1 Ton steel rails	40.00
3 Car trucks with wheels, axles and boxes	105.00
3 Skips, [illegible]	90.00
6 Cylindrical buckets	180.00
5 3-ft. radius ribs for lagging. Pipe and fittings.	
36 12-in. bracing jacks.	
1 Portable forge	30.00
2 Wheelbarrows	6.00
1 150-lb. medium anvil.	

GENERAL EXPENSES.

The following general expenses were incurred:

Superintendence		$ 2,400.00
Rental of plant and services of assistant		5,520.00
Installing Plant:		
Labor	$690.03	
Teams and lighter	135.00	
Supplies	562.76	
		1,387.79
Hoisting Plant:		
Labor	$797.00	
Lumber	532.92	
Supplies	54.20	
		1,384.12
Operating Plant:		
Labor	$7,860.22	
Coal	2,889.60	
Miscellaneous supplies	208.52	
Miscellaneous teaming	48.00	
		11,006.34
Removing Plant:		
Labor	$421.25	
Teams and lighter	190.00	
		611.25
Preliminary and Incidental Expenses:		
Pumping water from old tunnel	$388.32	
Building and removing bulkheads	382.58	
Cutting brickwork in old tunnel	491.08	
Cleaning out finished tunnel, etc.	276.38	
		1,538.36
Total general expenses		$23,847.56

The plant was operated 4,248 hours, from 7:30 a. m., March 13, to 7:30 a. m., Sept. 6, 1912. Cost of general expenses per hour, $5.61.

EARTH EXCAVATION.

About 301 cu. yds. of earth was excavated under air pressure of 19 to 27½ lbs. per square inch. Work was continuous for three shifts per 24 hours. Material encountered in the shaft varied from fine silty sand at the bed of the river to sand and coarse gravel, with some boulders, at the elevation of the tunnel. In the horizontal portion of the tunnel the material was largely blue gravel.

		Per cent of total.
General expenses	$5,277.34	56.8
Steel casing for shaft	760.00	8.2
Roof plates for tunnel	510.85	5.5
Clay	96.99	1.1
Tools	21.58	0.2
Miscellaneous supplies	23.86	0.3
Labor	2,506.10	26.9
Use of [illegible]	93.33	1.0
Total	$9,290.05	
Cost per linear foot	$61.18	
Cost per cu. yd. of excavation	30.90	

ROCK EXCAVATION.

Approximately 302 cu. yds. of rock was excavated in the horizontal portion of the tunnel. The rock was of a hard, though seamy, texture and required blasting. The air pressure used was the same as when the heading was in earth. When the heading was in solid rock,

Fig. 8. View of Steel Roof Plates Used in Driving Extension to Mystic River Water Tunnel in 1912.

about 26 lbs. of dynamite was used per linear foot.

		Per cent of total.
General expenses	$12,025.43	59.1
Roof plates for tunnel	493.53	2.4
Blasting supplies	91.38	0.5
Clay	60.45	0.3
Tools	21.56	0.1
Miscellaneous supplies and expenses	47.22	0.2
Labor	7,594.73	37.4
Total	$20,334.30	
Cost per linear foot	$130.85	
Cost per cu. yd. of excavation	67.29	

BRICK LINING.

About 310 cu. yds. of brick masonry (including concrete base at foot of new shaft and concrete plug in old shaft) was built, with an air pressure of 19 to 27½ lbs. per square inch. Brick masons were paid largely by piece wor[illegible]

		Per cent of total
General expenses	$ 4,368.84	39.7
Brick	1,304.96	11.8
Cement	727.22	6.6
Sand	186.37	1.7
Lumber	53.48	0.5
Miscellaneous supplies	88.52	0.8
Use of lighter placing shaft	121.67	1.1
Brick mason	793.40	7.2
Labor	3,369.08	30.6
Total	$11,013.54	
Cost per linear foot	$33.18	
Cost per cubic yard	35.55	

Fig. 9. View Showing the Pulling Up of the Old South Shaft of the Mystic River Water Tunnel.

REMOVING OLD SHAFT.

About 29 cu. yds. of brickwork were removed from the old shaft with the use of dynamite and bull pointing. The upper 18 ft. of brickwork were left in the steel casing, which was withdrawn by lighters and transported to the Naval Hospital pier.

Fig. 10. View of Plant Used in Sinking Shaft, Mystic River Water Tunnel Extension.

General expenses	$2,176.25	
Blasting supplies	19.91	
Clay	30.05	
Miscellaneous	32.37	
Labor	1,155.36	
Use of lighter handling shaft	440.00	
		$3,853.94
Credit for steelwork sold as junk		72.00
Total for removing old shaft		$3,781.94
Cost per linear foot		$74.16

SUMMARY OF COSTS

Earth excavation	$ 9,290.05
Rock excavation	20,334.30
Brick lining	11,013.54
Removing old shaft	3,781.94
	$44,419.83
Engineering	2,200.63
Total cost	$46,620.46
Cost per linear foot of complete tunnel	140.85

Difficulties Encountered and Methods Employed in Putting Down Large Dug Well in Water Bearing Sand at Atlantic City, N. J.

To the Editors: I have read with interest the description by Mr. J. W. Ledoux in Engineering and Contracting, of September 23, of difficulties encountered in sinking a pump well in New Jersey, and venture to send a few notes concerning another troublesome piece of similar work in the same State in 1904 in connection with the Atlantic City water works, of which I was at the time engineer and superintendent.

It was desired to construct the well of concrete 25 ft. in diameter with the top of the concrete floor about the same distance below the ground surface. The soil was sandy but underlaid by clay at a depth of 25 ft. and with the ground water level but 7 ft. below the surface. A view of the superstructure is shown in Fig. 1.

The contractor elected to construct a circular wooden curbing that would serve as the outer form of the concrete wall. When at a depth of 15 ft. this curbing showed signs of failure and the contractor was ordered to strengthen the work. This he did, but in an inefficient manner, and when the bottom of the pit reached a depth of 24 ft. a strong flow broke out through an old test boring that had been made earlier on the same site. This was of such volume that it was barely handled by a 6-in. and a 2½-in. pulsometer.

Fig. 1. Completed Superstructure of Pump Well at Atlantic City. Ten-inch Tubular Well on the Left. Edge of Old Pump Well on Extreme Right.

The contractor attempted to control this flow by driving a 6-in. and then an 8-in. cast iron pipe about the hole, placing 50 bags of sand around the same and then ramming small bags of hair, oakum and sand down the 8-in. pipe into the cavity below. The latter were ejected by the flow of water (about 375 gals. per min. were being removed by the larger pulsometer) but this was partially checked by ramming in manure. Meanwhile the water continued to break out around the cast iron pipes, the curbing continued to fail and the surrounding material to settle and it became evident that the contractor was unable to cope with the situation. The Department then took control and decided to surround the entire work by an octagon, 40 ft. in the clear inside, of 6 in. tongued and grooved sheet piling driven into the clay to prevent lateral infiltration and to lower the ground water by sinking a 10-in. tubular well a few feet outside, operated by air lift, to reduce the upward flow through the old test hole. A view of the pile driver and the nature of the ground surface of the well site is shown in Fig. 2.

After completing these two preliminary pieces of work, the old curbing with its intricate mass of crossbracing was entirely removed, the ground water lowered some 4 ft. by pumping the 10 in. well at a rate of about 1,000,000 gals. per day, and excavation was carried down to the clay. Heavy round timbers cut on the ground were then placed as bracing just above the bottom. But before anything further could be accomplished the flow in the bottom and from the sides increased to an alarming extent, causing settlement on the side to within 5 ft. of an old pump well, through which the entire city received its supply. As this was built of dry masonry it would have taken very little displacement of the adjacent earth to cause its destruction and allow its contents to flow into the new excavation.

To control the upward flow through the old test hole a 20-in. pipe was driven surrounding the 8-in. pipe already placed. As this was ineffectual a 16 ft. section of 42 in. wood-stave pipe was assembled, the materials for which were at hand, and this was driven around the 20 in. pipe and the flow finally controlled.

To prevent failure from further settlement on the outside a row of 2-in. sheeting was driven parallel to and a few feet from the 6 in. sheet pile curbing, to which it was braced, and the hole filled with alternate layers of hay, manure and clay. This prevented further trouble at this point. Two inch sheeting was also driven into the clay on the interior of the curbing, where leakage was excessive, and small crevices were calked with oakum. A view of the cave-in towards the old pump well is shown in Fig. 3.

Fig. 2. View of Driving of 6 in. Sheet Piling in Sinking Large Dug Well in Water Bearing Sand at Atlantic City.

Fig. 3. View of Cave-in Towards Old Pump Well Being Cleaned Out. This Was Refilled with Manure and Clay. Bank on Right Supported by Sheeting Braced to Sheet Piling.

Having the inflow of water well in hand, bags of concrete were placed under water in a ring 3 or 4 ft. wide around the inside, and the rest of the base was loaded with bags of sand and concrete. The water could then be safely lowered, a 12 in. ring of concrete was placed inside the ring of bags, the concrete floor reinforced by railroad iron laid, and the outer walls carried up to the top, ready for the superstructure, without further difficulty.

The lessons derived from this piece of work were, first, the futility of entrusting construction of this character to an incompetent or inexperienced contractor, and, secondly, recognition of the fact, at the start, that in wet underground construction it is more important to "play safe" than to save a few dollars in preliminary preparations.

Very truly yours,

KENNETH ALLEN

Park Row Building, New York City.
October 6, 1914.

Permissible Pollution in Raw Water to Be Filtered for Domestic Consumption.

The modern water filter has been so markedly successful in its service of purifying water supplies for domestic purposes that it has sometimes been taken to be a cure-all for any ills or unfortunate conditions which might affect the raw water. A raw water may be turpid enough to make it almost a liquid mud; it may be so badly polluted by bacteria that it is not much better than a weak sewage effluent; it may have such a strong color that it could easily pass at a soda fountain, yet in all such cases it has often been taken as a matter of course that the addition of an ordinary water filter, either of the slow sand type or the mechanical type, would be sufficient to bring the water to a condition of perfect purity in all its properties and make it absolutely satisfactory and sufficient for all drinking purposes.

LIMITATIONS OF FILTERS

It is unfortunately true that such perfection of operation cannot at all times and places be depended upon, and that the water filter, while often doing what seems almost impossible, may not always do all that it has sometimes been called upon to do. In some particulars, such as the removal of turbidity and the removal of color, the eye can readily detect to what extent the water filter fails of giving perfect results. In other particulars, such as the removal of bacterial pollution, so as to make the water of satisfactory sanitary quality, it is much less easy to see what has been accomplished by the filter, and because of inadequate supervision and lack of adequate bacteriological testing a filtered water of less than a reasonable standard of purity has often been accepted. This is particularly likely with a small plant which does not have regular and dependable laboratory supervision. There seems no doubt that a water filter can only effect so much and no more in the way of bacterial purification. Having in mind limits such as these, it will be necessary to consider the raw water to be applied to a filter and to determine how much of a burden the condition of the raw water will impose on the filter, so as to permit this filter to do reasonably satisfactory work, and to apply to a filter only such water as with reasonable preparation can be properly treated to give a good drinking water supply. These matters are considered in the present article which is a reprint of a paper by Mr. George W. Fuller, of New York City, before the recent annual convention of the American Society of Municipal Improvements.

[illegible]

While for some years this question of the proper loading or burden on filters has been considered in many places and attempts have been made to fix some sort of a standard which would serve the purpose, these attempts have met with very little success. The difficulties of fixing [illegible] evident. It is rather obvious that no exact relationship exists between the sanitary quality of the water and the number of bacteria present, no matter how the bacteria are tested, whether in the form of total bacteria or some special form such as the colon bacillus. It is possible that some waters should have good sanitary properties and yet have a rather large number of bacteria. Other waters, again, which might be relatively free from bacteria, would show themselves to be unhealthful.

While it has been clearly recognized that some limits, definite if possible, should be placed on the proper burden for filter plants, no such limit has, until recent times, been established.

B. COLI AS A BASIS OF DETERMINATION.

The first question that arises for determination is, How shall the quality of a water be measured or determined? For bacterial purposes, bacteria have been taken in various ways, measuring the total number of bacteria either at 20° Centigrade or 37° Centigrade on various media. Then, again, special bacteria, such as the B. coli, may be considered a better indication of fecal pollution than total bacteria. In some extreme cases experiments have suggested the taking of the actual determination of disease bacteria, such as the typhoid bacillus or the cholera bacillus, as a guide to the true disease-carrying sources of pollution. Each of these methods has some advantages and some disadvantages.

Considering the last, the direct determination of disease-producing bacteria, this is practically ruled out by the difficulty of making such determinations. It has not yet been generally feasible to determine with precision and certainty whether bacteria such as the typhoid are present in any water, and while such determination is sometimes successful it has not been generally so. In addition, we are not quite sure as to the limitations of water-borne diseases to any specific small number of kinds, and it would be quite possible that even exact and positive exclusion of a certain small number of organisms would not be a guarantee that the water could not contain some other disease-producing bacillus which has not yet definitely been determined as such.

The determination of the total number of bacteria present has been the simplest and most common form of water analysis procedure, and has considerable value. The old standard of the German sanitarians was an allowance of 100 bacteria per cubic centimeter in filtered water. This standard is a simple and in some ways a useful one, and has been widely extended in this country. Assuming a bacterial efficiency of the filters as 98 per cent, the raw water to give such a filtered water should not exceed a bacterial content of 5,000 per cubic centimeter.

The objection may be raised that by far the greater number of bacteria are simply water bacteria or non-pathogenic, and are no indication either of pollution or of having any harmful effect. A water may have a very large number of bacteria and still be absolutely harmless. This objection has been strong enough largely to discredit the total number of bacteria as being a very good evidence of the wholesomeness or unwholesomeness of any particular water.

The third determination, that of B. coli, has come to be recognized to be the most satisfactory, all things considered, of the possible methods of determining the pollution of a water. B. coli of itself is not, of course, any direct indicator of disease-bearing bacilli. It is only a very indirect evidence of the possibility of recent fecal pollution. As is well known, B. coli originates from many other sources than human fecal excrement. It is found in large quantities in animals, particularly domestic animals. It is also found in considerable quantities on grain materials and tilled fields. Water running off from farms, pastures, etc., must necessarily contain considerable quantities of such coli, and under such circumstances B. coli in no way show the evidence of dangerous pollution. Nevertheless, taking everything into account and knowing the limitations of the B. coli as an indicator of fecal pollution, it still must be said that this is the single best means we have of determining whether a water is most probably wholesome or unwholesome according to the number of B. coli present in the water. We recognize that this basis of determination is not at all conclusive or positive, but we also recognize that it is the best we have, and as such it is proper to make use of it in gaging the sanitary qualities of the water.

INTERNATIONAL JOINT COMMISSION INVESTIGATION.

An International Joint Commission, composed in part of appointees of the United States and in part of appointees of Canada, has been investigating the question of pollution of the international boundary waters, with the idea of determining to what extent pollution must be limited in order not to endanger the health of the communities on the two sides of these boundary waters. As a great part, if not all, of the possible danger to health which must arise from drinking water, the question practically resolves itself down to this:

1. Is it possible to maintain the water in such shape that without treatment it shall be suitable for drinking water?

2. If it is not possible to maintain the water in such a condition of purity and it does need filtration before being suitable for drinking water, what is the limit of pollution allowable before the filtration plant will be overloaded, and safe drinking water cannot reasonably be obtained by ordinary filtration?

The first source of information is those cities which have been receiving a supply of unfiltered water which have sufficient data available to show how this water stands in rank on the basis of B. coli present in the water. An examination of these B. coli records and the records of the city, its typhoid data, to show whether this water can reasonably be classed as a fairly healthful and wholesome water or whether it should be classed as a water of rather deficient quality. A second source of information is the various filtered waters which are supplied to a number of communities and the records they have of coli content in this filtered water and the B. coli content in the raw water supplied to those filters. Most of our communities which do have filtered water show a satisfactory effluent and a satisfactory typhoid fever rate, which is some measure of the wholesomeness of the water supplied. An examination of the correspondingly raw waters applied to the filters gives some indication of where a reasonable standard limit should be placed. A third source of information is the consideration of a standard of efficiency of bacterial removal and the assumption of a reasonable number of B. coli in water supplied for drinking purposes and a corresponding calculation of what would be a proper natural limit of B. coli in the water supplied to the filters under these conditions.

On the basis of the information of these three classes must be fixed the required standard. This information, which is afforded, is by no means exact in character, and no exact reading can be made from it even in any particular case. Moreover, the data obtained from various cities are very conflicting, and even the data obtained at various times from the same cities under apparently similar contions are conflicting and cannot always be reconciled. Some cities are receiving a markedly poor water in B. coli content, yet have a very satisfactory water from a sanitary standpoint, and typhoid fever rates are relatively low. Others, again, receive a rather good water and deliver for drinking purposes a decidedly good water and yet show a much less satisfactory typhoid fever rate.

It is recognized that typhoid fever originates only in part in water supplies, and that B. coli in the water and typhoid fever rates need not be in any relation to each other.

A case in point is that of the Birmingham, Ala., water supply. This city receives its water from two sources, namely, the Five-Mile Creek and the Cahaba River. The water from the Five-Mile Creek is sterilized after being filtered and shows practically no B. coli as delivered to the consumer. The water from the Cahaba River is filtered but not sterilized, and shows an appreciable amount of B. coli, though not an extraordinary number. Yet the typhoid fever of the two districts supplied from the two plants is no better in the first case than in the second. In the entire city the typhoid death rate is very low during the winter months but is high during the warmer season of the year, when fly transmission becomes a factor of importance.

The best than can be done with the data at hand is to consider all these data, properly weigh them, and on the basis of a rather general judgment come to some conclusion on what is a proper limit of B. coli in water to be applied to the filters. Such a conclusion can by no means be considered an exact judgment, and should be considered rather an average than a rigid limit for any particular case. This standard specifies that water applied to filters should show by the presumptive test not more than 500 B. coli per 100 c. c. of water as a yearly average, meaning by this that B. coli should not be found more than 50 per cent of the time in 0.10 c. c. samples. For averages for shorter selected times, monthly, weekly, or daily, the allowable B. coli content may be considerably higher. It is believed that for this purpose the averages based on a year's readings are more useful than averages for a shorter time with a correspondingly different standard of allowable B. coli.

The adoption of a standard such as this means that, apart from pollution objectionable in other ways than bacterial, the allowable limit of bacterial pollution should be particularly measured by the condition of the water as it reaches the intake of some water supply where the water is purified. If this water shows no more than 50 per cent postive tests for B. coli in 0.10 c. c. samples as a yearly average, the amount of bacterial pollution arising from a sewer outlet may be considered not too high. If, on the other hand, the bacterial standard so set is exceeded in the water supply, the need is set for the polluting community to purify its sewage by sterilization or further to such an extent that the water will not be affected to a more objectionable degree than the one allowed.

For any particular case to be considered special consideration must be given to the local conditions. If the water is such that practically all the B. coli can be attributed to sewage pollution, a rigid interpretation of the standard would properly be in order. In the case of a stream passing through farming communities where a large part of the B. coli does not originate directly from fecal excrement, but from fields, under such conditions that disease pollution would not follow, this standard can be more liberally applied. Other factors to be considered are the distance of the water supply intake from the source of pollution, the degree of dilution of the sewage effluent, the nature of wind and water currents, which may objectionably or favorably affect the movement of water to a water supply intake, and many other factors of this nature. While it is undoubted that a good deal of judgment must be used in special cases in applying any special standard of purity, and while such a standard cannot be used in ignorant hands as a substitute for expert supervision and expert judgment, a basis such as this is well worth establishing as a starting point for further investigations.

The bacterial efficiency of a filter is not an invariable quantity. Roughly speaking, 98 to 99 per cent may be placed as a fair average under ordinary conditions. Percentages, however, are not always a good gage of efficiency, as a water with an initial bacterial content that is high will show a proportionately higher efficiency in a filter than a water with a low bacterial content, and yet the resultant filtered water may be much less satisfactory to the consumer from a healthful standpoint than would be the case with the other filter with a lower bacterial efficiency. In addition, sterilization is usually applied in modern filter plants as a reserve factor of safety, and should be available to be applied in all plants and with the use of effective sterilization added to properly filtered water an effluent can be obtained which is almost, if not entirely, sterile for ordinary use.

With these conditions in mind, it seems reasonably safe to say that a water having in its raw form a coli content not exceeding 500 per each 100 c. c., based on yearly averages, will show in the water supplied to the consumers something not more than 5 to 10 B. coli per 100 c. c. Such water is believed to be a fairly safe water, when properly sterilized, for drinking purposes and for all other domestic uses.

Method Employed in Making Watertight Joints in Vitrified Clay Pipe at Meridian, Miss.

To the Editors: The article entitled "Some Water Works Engineering Mistakes," recently contributed to ENGINEERING AND CONTRACTING by Mr. Dabney H. Maury, and the letter from Mr. J. W. Ledoux commenting upon that article, are very interesting to the water works managers and superintendents throughout the country. If followed by similar communications along the same lines a lasting benefit will be conferred upon us.

During the present summer the writer laid a line of 18 in. terra cotta pipe, about 500 ft. long, and used the process described by Mr. Ledoux, namely, that of making the joints watertight first by calking with tar hemp (though the line was not first tested, as I knew it would hold under the head, 8 ft.), and then by carefully making up the joints with a 1:2 cement mortar on top of the hemp, the mortar being carefully troweled, and the ditch left open until the test was made. Not a leak showed up, nor has one shown since.

Yours very truly,

M. L. WORRELL,

Manager, Meridian, Miss., Water Department.

Meridian, Miss., Sept. 24, 1914.

(We desire again to extend an invitation to readers in the water works field to write us letters for publication along the lines of the above mentioned communications. What mistakes have you made in the design, construction or operation of water works, and what lessons have you learned from them? What methods have you used in successfully solving problems of a difficult nature?—Editors.)

ROADS AND STREETS

Resurfacing Old Macadam With Bituminous Concrete in Chicago.

(Staff Article.)

The boundaries of Chicago extend far beyond the densly settled portions of the city and include many suburbs separated by areas of land used by truck gardeners and other small farmers. Near the outskirts of the city the roads are in many respects rural in character, although carrying a heavy motor traffic and other traffic consisting mainly of heavy loads of farm produce hauled into the city from outlying districts.

These roads are, for the most part, surfaced with limestone macadam, Fig. 1, averaging 18 ft. in width. Until recently this type of surface has been satisfactory but in the last few years the enormous increase in fast moving pleasure traffic and heavy motor truck hauling, especially the latter, has occasioned wear such that the maintenance cost averages 20 cts. a square yard per annum.

In view of the large mileage and the impossibility of completely replacing the pavements with a type of surface better adapted to the traffic passing over and at the same time maintaining other roads in a passable condition at a reasonable cost, and in view also of the fact that the grades on most of these roads must be revised when permanent street improvement is begun, it was decided by the Bureau of Streets, which under the Chicago charter assumes the responsibility for the repair and maintenance of streets after the first paving is completed, to surface the old macadam with bituminous concrete making as few changes as possible in existing grades.

Funds for this work are derived from a wheel tax upon all classes of vehicles, apparently a direct and fair method of securing funds for street maintenance and repair. In Chicago the revenue from this tax is kept in a separate fund and is used for repair and maintenance purposes only.

MIXING PLANT.

Within the city limits of Chicago there are approximately 500 miles of macadam streets and roads, a large proportion of which are of the type described. To accomplish the work of resurfacing the outlying roads a portable, one-car asphalt plant with a rated capacity of 2,500 sq. yds. of 2 in. asphaltic concrete in 9 hours was purchased from the Warren Brothers Co., Boston, Mass., at a cost of $13,000. This plant was installed at 95th St. and Vincennes Ave., near the southern limits of the city, adjoining a railroad right-of-way and from this point surfacing material used this season has been hauled.

The plant was put in operation in May, 1914, and since that time has operated continuously. The rated capacity has been exceeded upon several occasions, 3,240 sq. yds. of surfacing being the maximum output in one day. Several changes were made in the plant after its erection by W. H. Barton, the foreman, to better adapt it to the work in hand. The whole plant was changed so as to use fuel oil for heating purposes, avoiding the smoke and other inconveniences from burning coal. A system of lubrication was

Fig. 1. Typical Cross Section of Roads Improved. Note the Deep Ditches and Narrow Shoulders.

Fig. 2. Laying New 8-in. Macadam Base.

devised whereby all small lubricators were displaced by one large central steam pressure lubricator with lead pipes to the various points where lubrication was required. This central lubricator was a home-made affair constructed of a section of 3 in. pipe about 4 ft. long with a steam lead having a condensing coil at the top of the pipe and the oil leads at the bottom. The Kinney rotary pump used to pump the heated binder was equipped with a reversible engine permitting pumping in either direction without change of fittings or pipes.

The general arrangement of the plant is clearly shown in Figs. 6 and 7, one photograph being taken looking at the end of the plant and the other from the side. Figure 5 is a diagram showing the relative location of the plant units and the arrangement of stock piles. To drain the concrete lined pit in which the hauling wagons stand while loading, a steam siphon was rigged. The siphon simply consisted of a steam pipe connected by means of a tee, having one leg extending into a sump in the pit and the other leading into a drain pipe discharging at some distance from the plant.

Operation.—In operation, the sand and stone mixed in the proper proportions are run into the dryer where the mixture is heated to 300° F. From there it is conveyed to the storage bins and thence to the measuring bins, whence it is drawn off as required to the 15-cu. ft. mixing drum. Materials are proportioned by weight.

An interesting method of mixing the sand and stone at the dryer elevator is used. An opening cut in the base of the wooden bulkhead at the foot of the elevator permits the stone to flow into the pocket at the elevator base. A man with a nice judgment as to correct proportions is stationed here and controls the flow of stone and indicates to shoveler when sand is to be added. It may be readily seen that the judgment of the mixer is all important.

The force employed at the mixer consists ordinarily of 1 foreman and 34 men, as follows: 1 chief drum man, 1 drum man, 1 kettle man, 1 mixer man, 1 timekeeper, 1 material man, 25 laborers, 1 assistant chemist, [illegible] ployed at the mixer averages $90 a day.

To facilitate the quick delivery of material at the plant a car tracer is employed who locates and keeps the cars in transit. This tracer uses a motorcycle and usually covers a distance of 75 miles each day.

Mixture.—The mixture used averages approximately [illegible]

Item.	Percentage.
Bitumen	[illegible]
Sand	[illegible]
Stone	[illegible]
Filler	[illegible]
Total	[illegible]

The stone aggregate consists of clean crushed Wisconsin granite ranging from ¼ to 1 in. in diameter which cost delivered $2.25 per cubic yard. Torpedo sand is used. Universal Portland cement serving as filler to make up the deficiency in fine material. The Mexican liquid asphalt used for binder (penetration 60 to 70) is delivered in tank cars.

Hauling.—Ordinarily about 18 teams are employed in hauling. Recently, however, 5 ton Pierce-Arrow motor trucks have been used to advantage. Material hauled in motor trucks is handled more quickly and arrives at the point where it is to be laid in better condition than when hauled by teams. The newly laid road is, however, subjected to excessive loads due to their use, all materials being, as far as possible, hauled over the completed surface.

SURFACING.

The approximate cross section of the old roads is shown in Fig. 1. As a rule little grading is done. However, where the surfacing has been completely destroyed, or where short stretches of new macadam base are necessary, Fig. 2, an ordinary 8-in. macadam road is constructed and thoroughly compacted prior to surfacing with bituminous concrete.

Preparation of Old Surface.—In preparing the old macadam for resurfacing, where it is flat or depressed in the center, the sides are hand picked and the material moved to the center and thoroughly rolled, care being observed to leave at least 4 ins. of compacted stone at the sides. Another method used with success consists of filling these depressions, and such slight holes as found at intervals, with stone coated with asphaltic cement hauled from the mixing plant. When the latter method is used, the plant is operated for, say, a day, preparing this binder stone and all holes and depressions are filled for some distance ahead of the surfacing. In cases where additional material is necessary over the entire width of roadway, the surface is scarified a depth of 3 ins. and sufficient crushed stone added to make the center thickness about 10 ins. and the sides 6 ins.

After leveling up and trimming the existing roadway as described, using care to disturb the existing surface as little as possible, and thoroughly compacting new work, all loose material and dust is removed from the surface by hand sweeping, exposing, as far as possible, the coarse stone of the macadam. Then 2x6 in. planks, laid flat, are lined in by eye, parallel to and with the inner edge 9 ft. from the center line to serve as a shoulder to hold the asphaltic concrete and to facilitate rolling at the edges. These planks are bedded and backed with earth. The roadway is thus prepared for laying surfacing.

Laying Surfacing.—Surfacing material is delivered hot in tarpaulin covered wagons, or motor trucks, and dumped directly onto the prepared base. The material is raked, smoothed and tamped, Fig. 3, to a uniform surface 2 ins. thick and finished by rolling with a 6-ton tandem roller. No paint, or finish coat of bitumen is applied a slight roughness of surface being desired.

Force Employed.—The day labor force employed in preparing the old surface and laying the asphaltic concrete is ordinarily organized as follows: 1 asphalt foreman, 2 rakers, 2 smoothers, 2 tampers, 15 helpers, 2 watchmen, and 2 roller engineers. By far the larger part of the work consists of laying the surfacing.

Fig. 3. Smoothing Bituminous Concrete Surface Prior to Rolling. Note Guide Boards at Sides.

Fig. 4. Completed Road Surface. The Shoulders and Ditches of the Old Road Undisturbed.

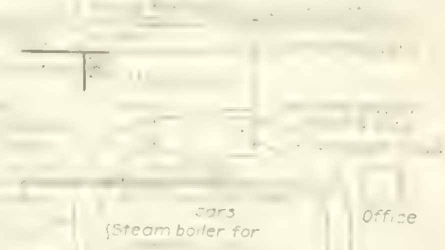


Fig. 5. Diagram Showing the Arrangement of Mixing Plant and Service Tracks.

OUTPUT AND COST

As a rule, in excess of 2,000 sq. yds. of 2-in. surfacing, or about 1,000 lin. ft. of roadway 18 ft. wide, has been covered each working day of 9 hours. The average cost of all work completed, including the preparation of the old roadway and laying the 2-in. asphaltic concrete surface, has been approximately 70 cts. per square yard. A total of about 7 miles of roadway has been resurfaced.

PERSONNEL

Walter G. Leininger, superintendent of streets for the city of Chicago, has the general supervision of this work, T. L. Mulroy, assistant to the superintendent, being in direct charge.

Reclipped Granite Block Pavement: A Use for Waste Material.

Granite block pavement is by many considered the most durable of modern pavements. A discussion of the use of blocks from old pavements of this type by Wm. A. Howell before the recent meeting of American Society of Municipal Improvements is given here:

Until recent years old granite blocks were not supposed to have any great commercial value. They were principally used for paving the yards of freight terminals and the tracks of street railway corporations, stock yards, etc. There was no regular demand for this form of paving material. In May, 1911, the city of Newark received bids for the repaving of a very important thoroughfare, calling for 23,500 sq. yds. of pavement. A smooth pavement was to be laid, and the contract provided that the old granite blocks taken from the old pavement shall become the property of the contractor, and that allowance for same should be made in his bid for the new pavement. The successful bidder was not particularly pleased with this provision of the contract and made the remark that he expected to have the old blocks on his hands about ten years. At this time we are replacing an old granite pavement on Broad Street, Newark, with a 4-in. wood block pavement. The yardage of the old pavement was 75,500 sq. yds., the number of paving blocks in the street aggregating 1,585,500 blocks, of which probably 85 per cent are usable. The city is selling these blocks to contractors on napped granite work at $30.00 per thousand. The blocks represent an asset to the city of about $40,000.00, a very tidy sum for the street repair fund.

The writer inspected a number of granite quarries in Maine and Massachusetts in the summer of 1909 and was accompanied throughout a portion of the trip by the owner of a large quarry in Maine, who was also a paving contractor in New York City and adjoining towns. The granite man was asked the question: "To what use can old granite blocks be put?" His answer was that "they could be napped and reclipped and would practically make a new pavement."

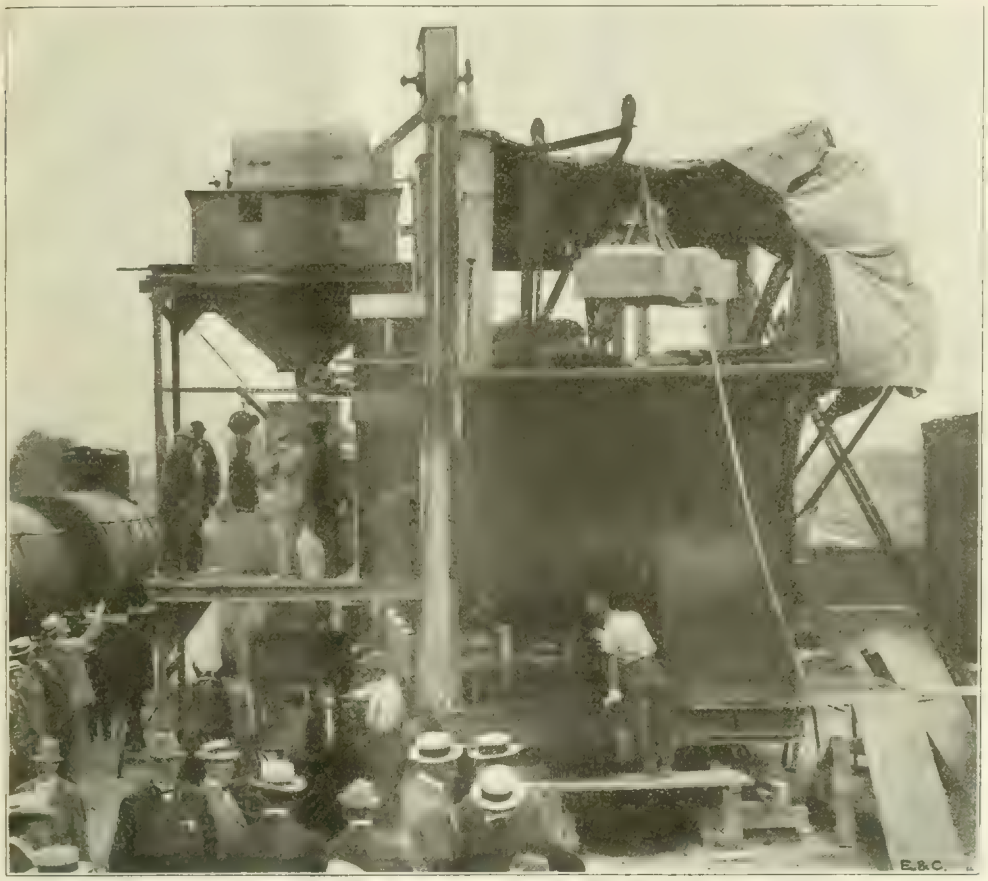


Fig. 6. Mixing Plant Showing Method of Loading Wagons.

FIRST USE OF RECUT BLOCKS

The first street within our knowledge where napped or recut granite blocks were used for paving material was on Webster Avenue in the Borough of the Bronx in 1909. The old blocks were 7 and 8 ins. in thickness and with few exceptions in fair condition, coming principally from Long Cove, Me., Cape Ann, Mass., and Somes Sound. The specifications called for the blocks to be 6 to 12 ins. long, 3½ to 4½ ins. wide and 5½ to 6½ ins. deep. Where a block ran up to 4¾ or even 5 ins. wide, the engineering department allowed it to be laid, provided the entire course was carried out with blocks having the same width. The price bid was $1.27 per sq. yd. for the paving and $4.00 per cu. yd. for the concrete, 5-inch concrete being used. The specifications originally provided for the joints to be filled with fine gravel and paving pitch. After the work had been started it was deemed advisable to substitute cement grout.

There may have been some small jobs of nappd. reclipped granite work done before the work referred to, but to the best of our knowledge this is the first contract work awarded by a municipality laid with this form of pavement. The street referred to is in first-class condition today. Since that date the Bronx authorities have laid thousands of yards of this form of pavement, mention of which will be made later on. "The old blocks (in the words of the contractor, Mr. William Booth) were simply napped and squared up to give a joint say not exceeding ¾ of an inch. The price paid for making these blocks was $10.00 per thousand."

USE IN VARIOUS CITIES.

Philadelphia.—Only one street in Philadelphia has been laid with napped, reclipped, grouted granite blocks, and that is Broad Street, between Cumberland Street and Silver street, a distance of about 1,450 ft. and a yardage of 10,546 sq. yds. This was a repaving job, the old blocks being taken up, reclipped on the job and replaced in the street. The cost was $1.43 per sq. yd., which included the cost of the sand cushion. The concrete base was in place. The specifications called for the blocks to be redressed to the following dimensions: 5 to 6 ins. deep, 5 to 7 ins. long, and 3½ to 5 ins. wide. "Of course, it is necessary (in the words of the specifications) that the blocks shall not be too badly worn to be clipped in such a manner that they will comply with the specifications and blocks differing in width more than one-quarter of an inch will not be allowed." This pavement was completed in June, 1913. Mr. Wm. H. Connell, chief of Bureau of Highways, states that in his judgment "the laying of this kind of pavement is very economical, as it equals in smoothness and appearance, the latest, and best type of construction, of granite block paving." The writer inspected this street during July of the present year, and was greatly pleased with its appearance. The heads of the blocks were remarkably smooth, and the joints appeared to be closer than could be obtained on the majority of streets laid with new granite.

Troy, N. Y.—Mr. A. E. Roche, city engineer of Troy, N. Y., states that during the past year the city of Troy has gone into the splitting and quartering of its old granite block pavements and relaying them, on quite an extensive basis.

The streets on which the blocks were halved and quartered, or napped and reclipped (as it is called in Newark) were paved some 30 or 40 years ago with Rockport Granite of a size 9 to 14 ins. long, 5 to 6 ins. in width, and 7 to 8 ins. in depth. These granite blocks naturally became rounded on the surface and very uneven. The streets particularly referred to are adjacent to the Boston and Maine Freight Terminal, and to the New York Central yards.

The successful bidder was required to take up the blocks, remove them from the site of the street to any plot of ground he could hire, for the purpose of napping and reclipping, to excavate the gravel foundation on which they had been originally placed, to a depth of approximately 4 ins., and to lay a concrete foundation of an aggregate of 1 part of cement, 3 parts of sand and 6 parts of stone. The blocks were then halved and quartered, so that they were from 7 to 9 ins. long, 4 to 5 ins. in width and 4 to 5 ins. in depth; then hauled on the street, laid on a sand cushion and grouted with a cement filler, mixed in the proportion of 1 part of cement to 1 part of sand.

In the laying of the blocks particular attention was given to the necessity of having a new surface always on top, so that the cement filler readily adhered to the joints as well as to the surface. This work was done entirely by contract, at a unit price of $2.30 per square yard, including excavation, concrete foundation, maintenance, and guarantee for a period of five years. On the street on which the local street railway company maintained tracks, a

Fig. 7. Mixing Plant Showing Stock Piles and Tank Cars.

price of $2.40 per square yard was paid for the same depth of excavation and concrete foundation.

The city required the contractor to split the entire number of granite blocks on the street, although we knew that an excess of about 25 per cent first class blocks would be obtained over and above that required. For this excess of split blocks the contractor was allowed $15.00 per thousand, delivered to the city engineer at such points as he directed within a radius of ½ mile from the place of cutting. The total amount expended this year for napped and reclipped blocks has been $62,000.

The city of Troy had previously been paying an average price of $3.85 per square yard for close jointed 4-in. Concord, New Hampshire, granite block pavements. This price is fair when one considers that we have never been called upon to expend anything for the upkeep and maintenance of streets on which this type of pavement was laid, although it has now been down about eight years. The city has in view for next year about 22,000 sq. yds. of old granite blocks, which we propose to halve and quarter, rather than discard for new blocks.

Brooklyn.—Mr. H. H. Schmidt, chief engineer, Bureau of Highways, Borough of Brooklyn, informs the writer that in his city "we have not used what is commonly known as 'napped blocks.'" In Brooklyn they have paved a number of streets with a recut granite block, made from the old large stone, but redressed to a regular specification size. The yardage, prices, etc., are given below:

Total sq. yds.	Granite, sq. yd.	Granite, incl. concrete, sq. yd.	Joint filler	Remarks.
[illegible]	$2.[illegible]	$3.66	*	No RR
2,162	2.67	3.50	*	RR
1[illegible]	2.[illegible]	3.4[illegible]	*	No RR
[illegible]	2.[illegible]	[illegible]	[illegible]	RR
1[illegible]	[illegible]	[illegible]	[illegible]	RR
[illegible]	[illegible]	[illegible]	*	RR

*[illegible] Tar [illegible]

All of the above are laid on a 6-in. base.

Manhattan.—Mr. W. H. Durham, chief engineer, Bureau of Highways, Manhattan Borough, states that no reclipped granite block pavements have been laid in that borough until very recently. A contract has just been completed for the paving of about 5,500 sq. yds. of reclipped granite blocks, using the old material found on the street, the blocks being broken in two and laid with the fractured side as the head, with 1-in. sand cushion. Of the total, 1,830 sq. yds. were filled with standard tar filler and 3,670 sq. yds. were cement grouted. This was done for purposes of comparison. The contract costs were, for wearing surface, $1.45 per square yard, and for concrete $5.00 per cubic yard. The pavement has but recently been opened to traffic, so that no statement can be given as to its value from a [illegible] the old blocks were adequate to furnish material for only about 60 per cent of the total area, the remainder having to be supplied with new material by the contractor.

Trenton.—The city of Trenton, N. J., has recently laid about 3,500 sq. yds. of this form of pavement, using cement grout for filler, the blocks being furnished by the city, the contract price per square yard on 5 ins. of concrete was $1.58. The writer has been informed that a considerable yardage of napped, reclipped granite has been laid in Albany and Schenectady, but has no exact figures as to cost.

[illegible] are also used in Schenectady, contain the following interesting features:

[illegible] its of the street, broken in half and redressed, [illegible] less than 6 ins. in length, not less than 3½ ins., nor more than 4½ ins. in width, and not less than 5 ins. nor more than 6 ins. in depth. Blocks less than [illegible] dressed to conform to the above dimensions. Any stones which are soft, brittle or [illegible] or which in redressing develop any imperfections, will be rejected. The Commissioner of Public Works will supply in the number of old granite blocks required to repave the street, the contractor, however, doing the necessary carting.

Paving.—The old granite blocks which have been broken in half and redressed, conforming to the dimensions stated above, shall be distributed along the street just in advance of the paving. Said stones shall be placed in such position and location and the carriage way shall be formed in such manner as to make the most substantial and durable pavement. Each course of blocks shall be of uniform width and so laid that all longitudinal joints shall be broken by a lap of at least 2 ins. and all side and end joints must not exceed ¾ in. The old blocks shall be paved as far as possible with the new faces up. If there is any difference in the width of the upper and lower faces of a block, the wider face must always be placed below. The blocks shall be laid with their longest dimension perpendicular to the center line of the street and upon straight lines across the carriageway.

Kearney, N. J.—The town of Kearney, N. J. (across the river from Newark), laid in the summer of 1913 a napped, reclipped granite block pavement on Tappan Street, between Davis Street and Chestnut Street. The old blocks came from the old blocks taken up from Market Street, Newark. The length of the improvement was 983 ft., the yardage 3,278 sq. yds., and the contract price per square yard of pavement, including the 6-in. concrete foundation, was $2.18.

Newark, N. J.—The first attempt at the napped, reclipped granite work in Newark was made with the paving of a portion of Meeker Avenue, extending from Frelinghuysen Avenue to Western Avenue. The contractor did not pay enough money to get good results and in addition to this his supply of reclipped blocks came from many different streets, representing the original output of different quarries. It was only after the expenditure of much time and patience that the pavement was finally accepted by the city.

The work awarded during the present year represents the paving of six streets and the repaving of three other streets, representing a yardage of 26,900 sq. yds. at an estimated cost of $94,337.45. A portion of this work has already been completed and gives general satisfaction. The various contractors have at their command the blocks from the old Broad Street paving now being replaced with wood block, and the smooth heads of the reclipped blocks and the close joints are very noticeable even by ordinary observers. Reclipped blocks compare favorably with cobbles and cost about 60 cents a square yard less. The city has under consideration the repaving of three very important streets with the napped blocks, the intention being in each instance to use as much of the old granite paving material in the present pavement as conditions will warrant.

[illegible]

The old blocks used on the 1914 jobs in Newark range in length from 10 to 14 ins.

A blockmaker can in a day's work of 8 hrs. nap and reclip 175 large blocks into 350 small ones. It costs the contractor $15.00 per thousand for the small blocks, or $30.00 per thousand for the large ones. These blocks run 21 to the square yard, or 42 to the yard for the small ones.

A rough detailed estimate of the cost of this kind of pavement, which would permit of a variation of possibly 10 to 15 cents, would be about as follows:

Item	Cost per sq. yd.
[illegible]	[illegible]
[illegible]	[illegible]
Laying and handling	.20
[illegible]	.05
[illegible]	.65
[illegible]	.10
[illegible]	.10
[illegible]	.12
Cost per sq. yd. (21 blocks required)	$2.48

[illegible]

Twenty-five or thirty years ago, when the paving of important thoroughfares was under consideration, the main point influencing municipal authorities, aside from the item of cost, was the question of durability. This explains the presence of granite pavements on many streets and highways in various cities where other paving materials would have been selected if they had been known to possess equal durability.

As very few pavements were laid having a concrete base prior to 1890, the entirely unnecessary thickness of 7, 8 and 9 ins. of granite was to a certain degree excusable. But after the general adoption of the concrete base, under granite pavements, it required nearly 20 years of constant agitation to educate the public mind sufficiently to reduce the thickness of the blocks to the reasonable standards of the present day. Some cities have reduced the thickness to 4 inches. We do not approve of that dimension in Newark on account of the impossibility of getting the pavers to distinguish the head from the side of the block when the dimensions are the same.

As to the financial saving affected by the use of the napped, reclipped blocks, in our city with prevailing prices the assessment on the property owner on a 36-ft. street, using new granite and new curbing, would be about $8.00 per front foot, or $200.00 for a 25-ft. lot. If napped and reclipped granite was used the blocks being sold the contractor by the city, the probable cost per foot of assessment to the property owner would be $6.35, or $158.75 per 25-foot lot. If the old paving material on the street is taken up and napped and reclipped and used in the repavement, a still greater saving can be effected, the estimated cost per front foot would be about $4.75, or about $120.00 per 25-ft. lot.

Cleaning Bituminous Pavements Without Sprinkling.

It is a generally accepted notion that water has little effect on bituminous pavements if not allowed to stand. Walter F. Slade, Commissioner of Public Works, Providence, R. I., in a paper before the recent meeting of the American Society of Municipal Improvements gives a contrary view.

The fact that bituminous pavements laid in the city of Providence are not sprinkled with water is a subject that has occasioned frequent comments on the part of visitors to this city who are interested in the maintenance and cleaning of such pavements. I will say briefly that we are convinced that bituminous pavement maintained in good repair can be kept in a condition more satisfactory to all classes that travel the streets, both afoot and in vehicles, without, rather than with, using water for sprinkling.

Bituminous pavements are cleaned by the patrol system, and are kept reasonably free from dust. We believe that the use of water has an injurious effect upon the durability of the pavement. It emphasizes every slight depression and gathers and retains the fine particles that collect on a street, creating an unsightly appearance and a muddy, slippery condition. This results in the skidding of automobiles and a vast increase in the number of accidents.

By keeping the pavements free from water another prolific source of complaint is escaped. Upon the best of pavements there will be depressions sufficient to retain a thin sheet of water. The rubber tires of the swiftly-moving auto, and especially the auto truck, will act as a syringe and force the muddy water in a small stream upon the clothing of any one passing along the sidewalk, and even across the sidewalk, covering windows and buildings with muddy spots.

That more might be learned upon the action of water as affecting the durability of bituminous pavements, it would be instructive to bring out the results of laboratory tests, and a comparison of the condition of streets laid about the same time, where water was used and where not. The whole question is a subject worthy of the earnest consideration of all officers that have charge of the maintenance of bituminous pavements.

TABLE I.—AVERAGE CONTRACT PRICES OF PAVEMENTS LAID IN PHILADELPHIA IN 1913.

	No. of contracts.	Length in miles.	Square yards.	Cost of contingent work.	Total cost.	Cost per sq. yd.
Grading	112	20.28	*981,593	$33,361.56	$473,312.96	$0.45
Paving:						
Asphalt with concrete base	107	11.31	196,574	6,638.72	418,669.00	2.10
Vitrified block with concrete base	23	3.50	64,890	5,440.96	169,110.11	2.52
Granite block (2d hand) with gravel base	1	0.19	1,725	....	2,950.09	1.71
Granite block with concrete base	1	0.44	5,137	....	19,297.17	3.76
Waterbound macadam (surfacing)	7	1.91	21,802	1,929.87	28,732.26	1.23
Bituminous macadam (surfacing)	3	0.29	4,186	....	7,572.75	1.81
Repaving:						
Asphalt with concrete base	28	3.16	45,847	2,650.97	93,401.16	1.98
Wood block with concrete base	36	2.40	39,793	6,143.65	142,912.68	3.44
Vitrified block with concrete base	22	2.78	48,107	3,447.11	114,528.15	2.31
Granite block with gravel base (2d hand block)	1	0.14	5,922	1,088.78	9,201.40	1.37
Granite block with concrete base	4	0.63	19,628	3,034.88	67,022.40	3.26
Granite block (redressed block)	1	0.25	10,546	68.78	16,243.06	1.53
Resurfacing:						
Asphalt with broken stone or bituminous base	20	2.60	45,587	1,089.02	57,202.91	1.23
Asphalt with concrete base	37	4.06	88,933	1,925.19	169,503.50	1.88
Waterbound macadam	5	6.05	90,442	2,067.32	44,947.66	.47
Bituminous macadam	15	5.10	60,976	7,720.89	83,053.92	1.24
Waterbound and	3	1.23	12,206	2,005.75	26,196.43	.56
Bituminous macadam		1.25	14,805			1.17
Patching:						
Asphalt	3	....	228,926	....	297,671.65	1.30
Bituminous macadam	1	....	1,568	....	1,898.99	1.21
Totals	431	67.57	2,013,144	$78,613.42	$2,246,315.75	.....

*Cubic yards.

The following details of construction apply to all classes of work shown in the accompanying tables:

Asphalt paving: 6-in. concrete base, 1-in. binder, 2-in. wearing surface.

Vitrified block paving: 6-in. concrete base, 1½-in. sand cushion, vitrified blocks 3 to 3½ ins. in width, 4 ins. in depth and 8½ to 9½ ins. in length.

Granite block paving: 6-in. concrete base, 2-in. sand cushion; new type close fitting granite blocks with dressed heads 5 to 5½ ins. in depth, 3½ to 4½ ins. wide on top and 8 to 12 ins. long.

Wood block paving: 6-in. concrete base, 1-in. mortar cushion; wood blocks 4 ins. in depth, 3 to 4 ins. in width and 5 to 10 ins. long.

Bituminous pavement: 2-in. bituminous top mixing method with concrete base, varying from 4 to 6 ins. in depth.

Bituminous macadam: 2-in. bituminous top mixing method with waterbound macadam base, 6 ins. in depth.

Waterbound macadam: 3-in. trap rock wearing surface, 5-in. broken stone base, or 4-in. trap rock wearing surface and 8-in. telford base.

TABLE II.—UNIT COST OF PAVEMENT REPAIR WORK ACCOMPLISHED BY CITY FORCES IN PHILADELPHIA IN 1913.

Character.	Square yards.	Cost of labor.	Cost of materials.	Total cost.	Cost per sq. yd.
Repairing:					
Asphalt	20,715	$ 14,614.97	$ 1,736.54	$ 16,351.51	0.79
Granite block	695,408	241,423.25	36,836.26	278,239.51	0.40
Granolithic	2,620	1,838.43	996.29	2,834.72	1.08
Vitrified block	126,423	39,325.86	4,958.84	44,284.69	0.35
Cobble	3,190	1,018.93	84.72	1,103.65	0.35
Rubble	164	52.20	4.03	56.23	0.34
Asphalt block	108	34.00	3.00	37.00	0.34
Resurfacing macadam:					
Waterbound	354,047	63,040.85	98,306.75	161,547.58	0.46
Bituminous	4,475	1,376.00	1,800.00	3,176.00	0.71
Patching and repairing to viaduct:					
Waterbound	201,674	40,151.55	12,476.12	52,627.67	0.26
Bituminous	8,943	3,160.46	1,835.99	4,996.45	0.56
Bituminous surface treatment of roads	431,126	6,443.35	25,150.40	31,593.75	0.073
Miscellaneous work*	....	68,284.73	1,237.22	69,521.95	....
Totals	1,848,893	$480,764.55	$185,426.16	$666,190.71	....

*Miscellaneous work includes the cleaning of macadam roads, the building of trunks, the placing of drains, pipes, etc., together with the maintenance of dirt roads.

Cost of Construction and Repair of Pavements in Philadelphia in 1913.

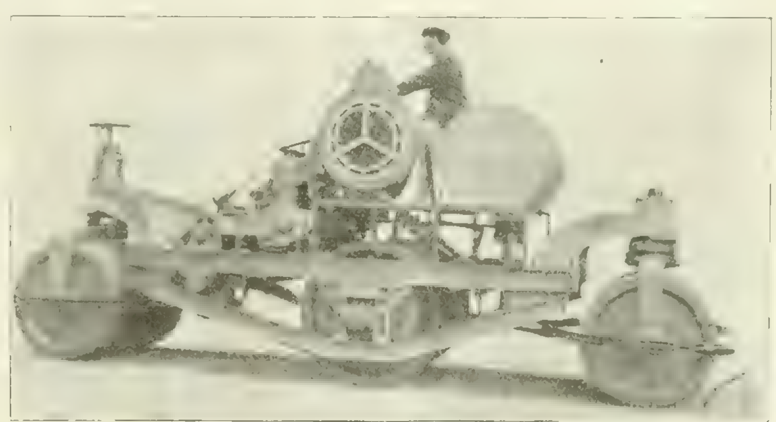
Pavement construction in Philadelphia is carried on almost entirely by contract. A portion of the repair work is accomplished by force account. Accurate records of both contract and force account work are kept. In the case of the latter, cost records are complete and accurate. Tables I and II, from a recent report of W. H. Connell, chief of the bureau of highways and street cleaning of Philadelphia, give the unit prices and costs of their work.

"Harmonic Waves" in Bituminous Pavements and Their Elimination.

The tendency frequently shown in the surface of bituminous pavements to develop waves of equal length under heavy traffic is discussed by W. de H. Washington in the Proceedings of the American Society of Civil Engineers, Vol. XL., p. 1523: Col. R. E. Crompton of London was perhaps the first to apply the term "harmonic waves" to the tendency of bituminous pavements to creep and form miniature hills and hollows. With certain types of traffic, he has found that the distances between the hills and hollows have a certain relative length. On one road examined there were more than 1,000 motor buses per day, and found a series of long waves, which occurred with great regularity throughout the length of the road.

It is probable that improper rolling of the road and the compression of the material are the main factors influencing the formation of harmonic waves. Traffic further develops the harmonic waves due primarily to the vibration of the motor cars passing over them. During the process of rolling it will be noticed that the material rises in front of the front roller and pushes ahead to a certain extent, finally ceasing to move forward, at which time the roller goes over it. If examined, the road surface will be found to have a marked swell.

Col. Crompton has endeavored to solve this problem by devising a machine, Fig. 1, with three sets of rollers, instead of the ordinary tandem roller. The first and third rollers are somewhat lighter than the second one. The first roller consequently smooths and compresses the material, but does not put excessive weight on it. The second and third rollers give the additional compression necessary. It has been found possible to secure a much smoother base and surface for the road with this triple roller than with the ordinary tandem roller.



The Crompton Triple Roller for Asphalt Pavement.

Comment on the Salvage of Road Hauling Machinery.

To the Editors: In your issue of August 26, 1914, a very able article entitled, "The Salvage of Road Hauling Machinery," appeared and has no doubt been read with much interest and carefully considered by engineers and contractors.

However, the proposed solution of the problem which confronts the contractor when he finishes a job and wishes to dispose of equipment is not the only possible one. Many contractors have found it more advantageous to delegate to "operating" or "transportation" companies the task of keeping mechanical hauling equipment in continuous operation the year round independent of whether any one individual contractor has constant use for the equipment or not. In this way the maximum of economy in operation can be secured. Naturally the contractor busy with his own problems of construction has not the time, in fact cannot afford to take the time, to secure work for his trucks and hauling equipment when they are not needed in his own work.

Looked at from the manufacturers' standpoint the proposition as outlined in the article in question is not a very promising one and it does not seem likely that he could ever enter into a salvage of trucks or other hauling equipment on a lasting business basis; although a few without looking into the future might attempt such a method of marketing equipment; namely, by agreeing to buy it back when the purchaser was through using it. However, it seems almost certain that in a short while such a scheme would lead to a flooded condition similar to that which now exists in the second-hand automobile market where used cars have become a drug, and would react in a way very unsatisfactory to all concerned.

Thus it would seem to be to the advantage of manufacturer and contractor to put the task of keeping trucks in operation up to the "transportation" company paying it a legitimate profit for its services. Companies organized for such work as this are now in existence and the number will undoubtedly increase from time to time.

The contractor who does not feel that he has sufficient work to justify the expenditure of a large amount of capital for hauling equipment may find this a very satisfactory solution of his transportation problems.

Yours very truly,

CLINTON BRETTELL,

Transportation Engineer.

The Motor Haulage Co., Broadway and 61st St., New York, Sept. 30, 1914.

BUILDINGS

Details of a New Method Used in Constructing the Retaining Walls of the Lumber Exchange Building, Chicago, Ill.

[illegible]

In planning the design and construction of the Lumber Exchange Building, corner of Madison and La Salle Sts., Chicago, Ill., a novel and effective method was developed in connection with the construction of the retaining walls. The construction work was especially hazardous on account of the necessity of preventing any considerable settlement of the foundations of adjacent buildings. Adjoining this building on the south there is the 16-story Y. M. C. A. Building, which has floating foundations; across the street to the north is the 13-story Tacoma Building, which also has floating foundations; while a 3-story building with floating foundations adjoins the property on the east. The Lumber Exchange Building covers an area of about 101 ft. x 135 ft., the height of the structure being 200 ft. It is an office building, with caisson foundation extending to bed-rock, a steel frame, and tile arch floors. The first story is faced with granite; the second to the sixth stories, with terra cotta; while the sixth to the sixteenth stories have a brick facing with terra cotta trim. Over a considerable part of the area the excavation is carried to a depth of about 50 ft. below the street grade to provide for the basement, sub-basement and second sub-basement.

[illegible]

To avoid the difficulties and uncertainties of the usual method of constructing deep retaining walls, it was decided to adopt a new construction procedure. Figure 1 illustrates [illegible] flange plates as shown in Fig. 1, although parts of it are reinforced with rods) was excavated in alternate sections breaking joints at the centers of the columns. These trenches were excavated 6 ft. wide, the order of excavating the material between columns being shown in Fig. 1 (a). As noted in this drawing [illegible] this section was being shored, section 2 was excavated, etc. [illegible] tongue-and-groove lagging was then placed in a vertical position against the earth, and two lines of 8 x 8-in. drums were placed in a horizontal position against the 8 x 8-in. waling pieces, as shown in Figs. 1 (c) and (d). [illegible] pieces were placed against each length of the lagging. After the lagging and waling were set on each side of the trench, 8 x 8-in. drums and jack screws were placed against the lagging, as shown in Fig. 1, the drums being tightened as much as possible by means of the jacks. This process was continued along the trench until the end of the section was reached and also downward to the bottom of the trench. About 6 ins. of concrete were deposited in the bottom of the trench for a working floor, and the forms were then built from the bottom to the top of the retaining wall. The type of form used and its bracing are clearly shown in Figs 1 (c) and (d). The lagging used was in lengths of 3 ft. to 4 ft., and the waling pieces were from 6 ft. to 8 ft. long, their spacing varying from 1 ft. 6 ins. to 3 ft., depending upon the earth pressure. The 2 x 6-in. longitudinal and the 2 x 4-in. cross bracing for the forms had the same spacing as the waling pieces.

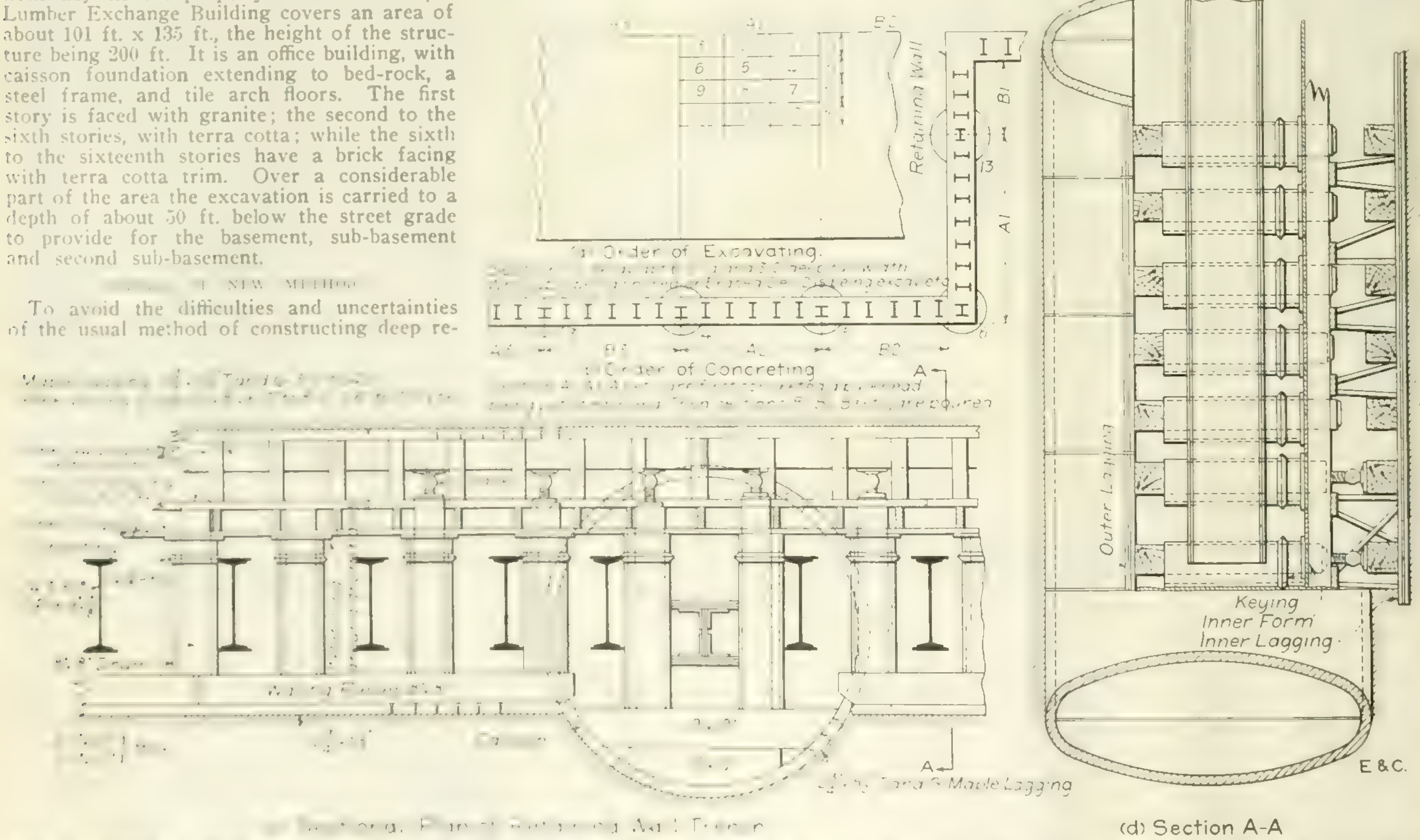


Fig. 1. Details Illustrating New Method of Constructing Retaining Walls for Buildings as Used in Construction of Lumber Exchange Building, Chicago, Ill.

The principal advantage of this method of retaining wall construction lies in the fact that it is possible to pour the wall as a monolith in one operation. This is made possible by boxing in the drums so that they can be removed after the concrete has hardened, the openings left by the boxes being filled later. The drums were boxed in as shown in Figs. 1 (c) and (d), each drum requiring two 1 x 10-in. pieces and two 1 x 12-in. pieces. As shown in Fig 1 (c) the holes through the form, through which the drums projected, were made tight for concreting by means of 1-in. strips. The system of bracing used at the caissons is shown in Fig 1 (c). After the timber construction shown in Figs 1 (c) and (d) had been completed the reinforcement, which in general consists of 24-in. 80-lb. I-beams with 9x⅝-in. flange plates, was set in place and the trench was filled with concrete, the order of concreting being as indicated in Fig 1 (b). The outer 8x8-in. waling pieces and lagging are left in place in the completed wall. When the basement is excavated the inner lagging is removed, the drums are pulled out of the boxes, the inner form and the boxes are taken down, and the holes left by the boxes are rammed with dry concrete, which is held firmly in place by the notches left by the 2x1⅛-in. keys on the boxes.

COMPARISON WITH OLD METHOD.

In order to show more clearly the advantage of the new method over that commonly used in constructing deep retaining walls reference is made to Fig. 2, which shows the construction used for the old method. With this method it is evident that the wall is not monolithic, but consists of a series of horizontal layers, the thickness of the layers, L_1, L_2, etc., being equal to the distance between the drums. It is evident that the inner form for lowest layer, L_1, must be constructed and that layer concreted and the concrete allowed to harden before the drum and jack for layer L_2 can be removed, the inner form continued, and that layer poured, etc. As each layer must be allowed to set for at least 12 hours before new drums can be placed back of the forms in the concreted layer and before the next layer can be concreted, the saving of time permitted by the use of the new method is evident. It will be noted that with the ordinary method the outer lagging is left in the ground, but that the outer waling pieces are not left in the concrete but are removed as the concreting progresses.

ADVANTAGES OF NEW METHOD.

The progress of concreting the trenches is much faster than with the old method, and the objection to the use of rods for reinforcement is largely removed. The drums resist the pressure of the earth until the concrete has thoroughly hardened, and the completed wall acts as a monolith, since the horizontal joints are avoided.

In the old method it was generally necessary to leave out the floor arches in the floor above in order to permit the placing of the I-beam reinforcement, rods not being satisfactory for reinforcement due to the large numbers of horizontal joints, which make it difficult to provide for the shear. In the desire to save time there was always the temptation, when the old method was used, not to give the concrete sufficient time to harden before the earth pressure was applied to it. The new method is especially advantageous when it is necessary to insure adjacent floating foundations against settlement. In this case, the settlement of the adjacent 16-story Y. M. C. A. Building was quite small, even though the party wall and the wall columns of this structure are carried on spread footings, which project a considerable distance into the site of the Lumber Exchange Building. The new method proved to be especially effective in constructing the portions of the retaining wall which contained reinforcing bars.

PERSONNEL

The Lumber Exchange Building was designed, and the method explained in this article developed, by Holabird & Roche, architects, Chicago, Ill. The George A. Fuller Co. was the general contractor; the American Bridge Co. furnished the steelwork; the Oscar Daniels Co. erected the steelwork, and the Ruud-Nielson Co. had charge of the shoring work. We are indebted to the architects of this building for the data upon which this article is based.

Blast Furnace Slag as Aggregate in Concrete.

The following data give the results of an investigation to determine the value of blast furnace slag as aggregate in concrete. The data were abstracted from a paper by W. A. Aiken, presented before the recent annual meeting of the American Society for Testing Materials. The conclusions drawn are based on the results of about 400 tests. About 100 cubes still remain to be tested.

MATERIAL USED AND RESULTS OF TESTS.

The materials used in the test specimens were all commercially produced in Philadelphia, and the work of making the specimens was no better than that under ordinary field conditions of concrete construction. Thorough mixing, however, was positively assured, the work being done by hand. The cement used was a standard Lehigh Valley brand, and the sand was Jersey "gravel," a material universally used in the vicinity of Philadelphia. The following is an analysis of the latter material:

Sieve.	Amount passing, per cent.
¼-in.	97.4
No. 10	90.8
No. 20	73.8
No. 30	63.8
No. 40	55.2
No. 60	35.4
No. 100	11.6

This material is not ideal, as the results of the analysis show, but it was used, as it is the material of the market.

The coarse aggregates, commercially called ¾-in. material, all passed the 1¼-in. sieve and were retained on the ½-in. sieve.

A chemical analysis of the slag gave the following results:

	Per cent.
Silica	34.4
Alumina and iron oxide	23.40
Lime	35.88
Magnesia	3.21
Sulphur anhydride	0.59
Sulphur	1.27
Loss on ignition	1.34

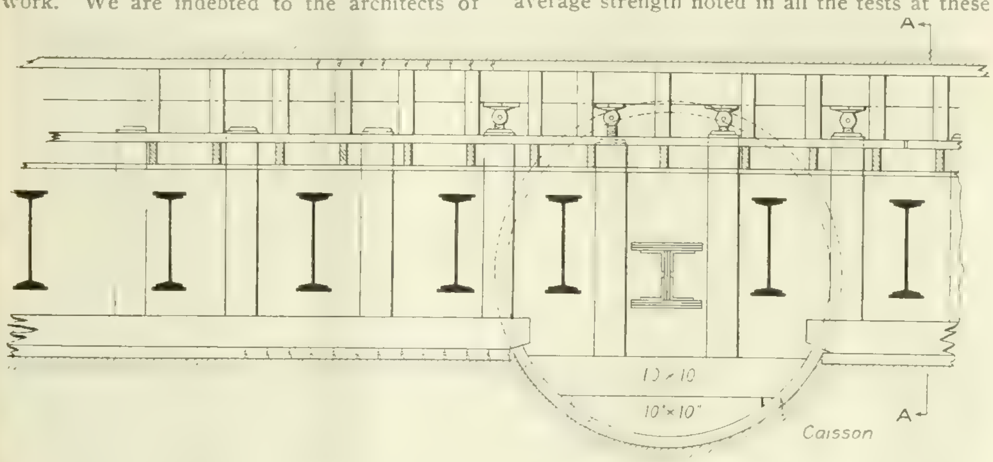
All materials were measured by volume in the proportions of 1 part cement, 2 parts sand and 4 parts coarse aggregate. All concrete was mixed to ordinary working consistency—rather wet than dry. The cubes were all air stored in a dry cellar, being sprinkled with water once a week. Table 1 gives the results of the compression tests of 400 cubes.

CONCLUSIONS.

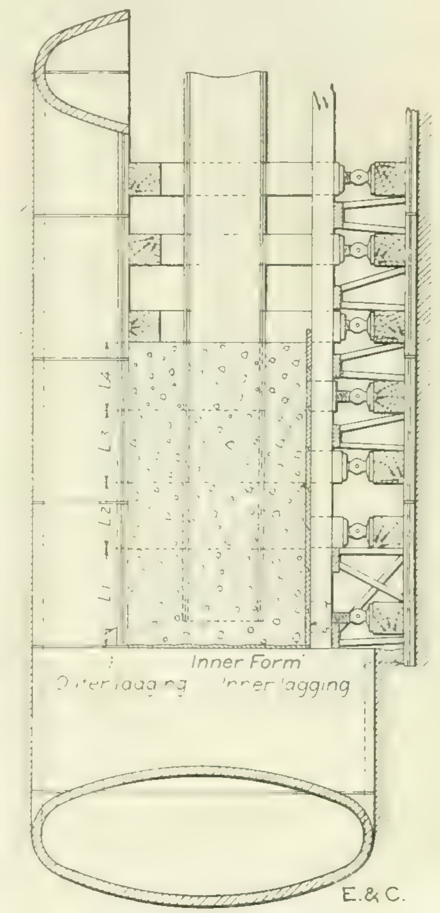
A study of this detailed differentiation of test results shows that at 28 days, 3 months and 6 months, of the number of individual tests failing to agree closely with the several general averages, the large percentage of these show considerably greater strength than these general averages or than the average of the larger percentage of tests which agree so closely with the several general averages. The average strength noted in all the tests at these periods, 28 days, 3 months and 6 months, may therefore be assumed as thoroughly conservative.

A similar study of results at 9 months and 1 year shows that, of the results not in close agreement with each period's general average, or with the larger number of tests at each period which closely approximate these general averages, a somewhat larger percentage falls below this general average than that which runs above. But, as those results which are above the general findings average relatively much more *above* such general findings than the low results average *below* these general findings, it may be assumed that the average strength results of all the tests at the later periods of 9 months and 1 year are equally conservative.

These conclusions are more particularly evident when it is borne in mind that the sand used was not what could be considered first-class material, and this undoubtedly influenced the strength of the concrete. Moreover, the comparatively small size of the slag aggregate must necessarily have influenced the strength



(a) Sectional Plan of Retaining Wall Trench



(b) Section A-A

Fig. 2. Details Illustrating Method Commonly Used in Constructing Retaining Walls for Buildings.

TABLE I.—RESULTS OF COMPRESSION TESTS OF 6-IN. SLAG CONCRETE CUBES.

(Average weight of concrete was 140.8 lbs. per cubic foot.)

Age when tested.	No. of specimens.	Average compressive strength, lbs. per sq. in.	Remarks.
28 days	100	1,561	73 per cent of all the tests averaged 1,533 lbs. per sq. in.; within 2 per cent of the general average, but lower.
			20 per cent of all the tests averaged 1,730 lbs. per sq. in.; within 11 per cent of the general average, but higher.
			7 per cent of all the tests averaged 1,344 lbs. per sq. in.; within 14 per cent of the general average, but lower.
3 months	100	1,952	78 per cent of all the tests averaged 1,922 lbs. per sq. in.; within 2 per cent of the general average, but lower.
			15 per cent of all the tests averaged 2,185 lbs. per sq. in.; within 12 per cent of the general average, but higher.
			7 per cent of all the tests averaged 1,794 lbs. per sq. in.; within 8 per cent of the general average, but lower.
6 months	100	2,589	73 per cent of all the tests averaged 2,583 lbs. per sq. in.; practically in complete agreement with the general average.
			14 per cent of all the tests averaged 3,058 lbs. per sq. in.; within 18 per cent of the general average, but higher.
			13 per cent of all the tests averaged 2,125 lbs. per sq. in.; within 18 per cent of the general average, but lower.
9 months	50	2,841	72 per cent of all the tests averaged 2,874 lbs. per sq. in.; within 1 per cent of the general average, but higher.
			8 per cent of all the tests averaged 3,367 lbs. per sq. in.; within 18 per cent of the general average, but higher.
			20 per cent of all the tests averaged 2,514 lbs. per sq. in.; within 12 per cent of the general average, but lower.
1 year	50	2,797	84 per cent of all the tests averaged 2,812 lbs. per sq. in.; within 1 per cent of the general average, but higher.
			6 per cent of all the tests averaged 3,534 lbs. per sq. in.; within 26 per cent of the general average, but higher.
			10 per cent of all the tests averaged 2,342 lbs. per sq. in.; within 16 per cent of the general average, but lower.

of the concrete, and the results would probably have been higher had the test specimens been larger. It is also to be noted that in so far as the author's observation goes, the results are markedly lower than those published by other investigators of slag concrete.

From the actual strength of the concrete; the weight of the slag per cubic foot (which is less than that of most material used similarly); the recognized solubility of slag, which permits it to act as a puzzolanic material; its alkaline nature, which is especially conducive to rust prevention in reinforced concrete construction; and from the relative high combined percentages of silica, alumina and iron, making for permanency in the resulting concrete; it is concluded that slag of similar composition is in every way satisfactory for use as aggregate in concrete.

Checking Lists for Estimators on Building Work.

Contributed by L. A. Waterbury, Professor of Civil Engineering, University of Arizona.

The preparation of detailed estimates and bills of material for the use of contractors requires considerable care to avoid the omission of items. For work of this character the writer has made use of checking lists which should prove of interest to others. Mimeographed forms are used, made upon 16-lb. bond paper, 8½x11 ins., so that the preparation of checking lists for a particular job is done by checking the required items which are listed upon the forms and by adding any unusual items which do not happen to be included upon the mimeographed list.

Before beginning to itemize the quantities to be billed the checking lists are made out for the entire job. For a new job much of this can be done while studying the plans and specifications, noting in particular those items called for in the specifications which might easily be overlooked in taking off quantities from the plans.

The form shown in Fig. 1 is for use in checking the general subdivision of items

Job ... Sheet ...
... CLASSIFICATION FOR BUILDING
... Date

ITEM NO.	ITEMS FURNISHED BY GENERAL CONTRACTOR: MATERIALS			ITEMS FURNISHED BY GENERAL CONTRACTOR: LABOR			SUB-CONTRACTS			DESCRIPTION OF ITEM
1										PREPARATION AND PRELIMINARY
										...TE AND CEMENT WORK
										...CING STEEL
30										... HARDWARE
										...D HARDWARE

Fig. 1. Checking List for General Classification of Work.

Job ... Sheet ...
Checking List for CONCRETE FOR ...
Listed by ... Date ... Checked by ... Date ...

ITEM NO.	MATERIALS: Required	MATERIALS: Places	MATERIALS: Itemized	LABOR: Required	LABOR: Places	LABOR: Itemized	SUB-CONTRACTS: Required	SUB-CONTRACTS: Places	SUB-CONTRACTS: Itemized	DESCRIPTION OF ITEM
1										FOOTINGS
2										FOUNDATION WALLS
3										FOUNDATION PIERS
4										COLUMNS
5										WALLS EXCLUSIVE OF FOUNDAT'N
6										FLOORS
7										CEILINGS EXCLUSIVE OF FLOORS
8										ROOF
9										COPING AND CAPS
10										WATER TABLES and BELT COURSES
11										ENTABLATURES AND CORNICE
12										LINTELS
13										SILLS
14										CHIMNEYS AND VENT STACKS
15										EXTERIOR STEPS
16										INTERIOR STAIRS
17										HEARTHS
18										SWIMMING POOLS
19										TANKS AND RESERVOIRS
20										SIDEWALK
21										CURBING AND GUTTERS
22										MANHOLES
23										COVERS (Manholes, Cess Pools, etc.)
24										CONCRETE PIPE
25										CAISSONS, WELL CURBS, ETC.
26										
27										
28										
29										
30										
31										
32										
33										
34										
35										
36										

Fig. 2. Checking List for Concrete Work.

Job ... Sheet ...
Checking List for DIMENSION LUMBER FOR ...
Listed by ... Date ... Checked by ... Date ...

ITEM NO.	MATERIALS: Required	MATERIALS: Places	MATERIALS: Itemized	LABOR: Required	LABOR: Places	LABOR: Itemized	SUB-CONTRACTS: Required	SUB-CONTRACTS: Places	SUB-CONTRACTS: Itemized	DESCRIPTION OF ITEM
1										MUD SILLS
2										UNDERPINNING
3										SILLS
4										GIRDERS
5										FLOOR JOISTS
6										CEILING JOISTS
7										BRIDGING (Over 1 in.)
8										STUDDING
9										BRACING FOR STUDDING
10										RIBBON GIRTS (Over 1 in.)
11										PLATES
12										POSTS AND COLUMNS (Unmilled)
13										PILLAR FRAMING
14										BOLSTERS AND CORBELS
15										ROOF TRUSSES
16										PURLINS
17										RAFTERS
18										COLLAR BEAMS (Over 1 in.)
19										RIDGE BOARDS (Over 1 in.)
20										ROOF BRACING (Over 1 in.)
21										STAIR STRINGERS
22										PARAPET FRAMING
23										RAILINGS (Unmilled)
24										TOWER FRAMING
25										CORNICE FRAMING
26										SKYLIGHT FRAMING
27										WOOD LINTELS
28										ROUGH DOOR BUCKS
29										CONCRETE FORMS (Over 1 in.)
30										STAGGING
31										
32										
33										
34										
35										
36										

Fig. 3. Checking List for Dimension Lumber.

Job.......... Sheet..........
Checking List for UNWORKED AND ROUGH UPPERS FOR..........
Listed by.......... Date.......... Checked by.......... Date..........

ITEM NO.	ITEMS FURNISHED BY GENERAL CONTRACTOR						SUB-CONTRACTS			DESCRIPTION OF ITEM
	MATERIALS			LABOR						
	Required	Places	Itemized	Required	Places	Itemized	Required	Places	Itemized	
1										RIBBON GIRTS (1 in. and under)
2										PLATES "
3										BRIDGING "
4										BRACING FOR STUDDING "
5										ROOF BRACING " "
6										COLLAR BEAMS "
7										ROUGH FLOORING
8										SHEATHING
9										UNW'RK'D LUMBER, PILLARS (1 in.)
10										" " STAIRS "
11										" " PARAPETS "
12										" " RAILINGS "
13										" " LOUVRES "
14										" " TOWERS "
15										" " SKYLIGH'S "
16										" " EXTER. TRIM
17										" " INTER. TRIM
18										FURRING STRIPS
19										GROUNDS
20										RIDGE BOARDS (1 in.)
21										LUMBER FOR FORMS (1 in.)
22										LUMBER FOR STAGGING (1 in.)
23										
24										
25										
26										
27										
28										
29										
30										
31										
32										
33										
34										
35										
36										

Fig. 4. Checking List for Unworked and Rough Uppers.

Job.......... Sheet..........
Checking List for WORKED UPPERS FOR INTERIOR TRIM FOR..........
Listed by.......... Date.......... Checked by.......... Date..........

ITEM NO.	ITEMS FURNISHED BY GENERAL CONTRACTOR						SUB-CONTRACTS			DESCRIPTION OF ITEM
	MATERIALS			LABOR						
	Required	Places	Itemized	Required	Places	Itemized	Required	Places	Itemized	
1										FINISHED FLOORING
2										CASING (Windows, Doors)
3										PULLEY STYLE (If not with frames)
4										DOOR JAMBS " "
5										STOPS (Windows, Doors)
6										WINDOW STOOL
7										WINDOW APRON
8										BASE MOULD (Pieces)
9										PICTURE MOULD (If stock pattern)
10										PLATE RAIL " "
11										FALSE BEAMS " "
12										CHAIR RAIL
13										WAINSCOT
14										SHELVING
15										HOOK STRIPS
16										CEILING
17										THRESHOLDS
18										STAIRS (If not in Millwork)
19										DRAIN-BOARD " "
20										BENCHES " "
21										COUNTERS " "
22										INTERIOR COLUMNS(Except Millw'k)
23										LATH (If not included in plaster)
24										
25										
26										
27										
28										
29										
30										
31										
32										
33										
34										
35										
36										

Fig. 6. Checking List for Worked Uppers for Interior Trim.

Job.......... Sheet..........
Checking List for WORKED UPPERS FOR EXTERIOR TRIM FOR..........
Listed by.......... Date.......... Checked by.......... Date..........

ITEM NO.	ITEMS FURNISHED BY GENERAL CONTRACTOR						SUB-CONTRACTS			DESCRIPTION OF ITEM
	MATERIALS.			LABOR						
	Required	Places	Itemized	Required	Places	Itemized	Required	Places	Itemized	
1										LOUVRES
2										RAFTER ENDS
3										FINISHED SHEATHING
4										SIDING
5										RAILINGS (If not in Millwork)
6										CORNICE
7										FRIEZE
8										ARCHITRAVE
9										TENIA
10										SOFFITS
11										LUMBER FOR COLUMNS & PILLARS
12										FINISHED FLOORING (Exterior)
13										STAIRS AND STEPS "
14										BELT BANDS
15										WATER TABLE (If not in Millwork)
16										DRIP CAP
17										CORNER TRIM
18										GRILLE AND PATTERN TRIM
19										LATTICE WORK
20										PARAPETS
21										CEILING (Exterior)
22										SHINGLES
23										BATTENS
24										PICKETS
25										BENCHES (Ifnot in Millwork)
26										
27										
28										
29										
30										
31										
32										
33										
34										
35										
36										

Fig. 5. Checking List for Worked Uppers for Exterior Trim.

Job.......... Sheet..........
Checking List for MILLWORK AND GLASS FOR..........
Listed by.......... Date.......... Checked by.......... Date..........

ITEM NO.	ITEMS FURNISHED BY GENERAL CONTRACTOR						SUB-CONTRACTS			DESCRIPTION OF ITEM
	MATERIALS			LABOR						
	Required	Places	Itemized	Required	Places	Itemized	Required	Places	Itemized	
1										WINDOW FRAMES, DOUBLE HUNG
2										" " CASEMENTS
3										" " FIXED SASH
4										DOOR FRAMES, DBL. WITH TR.
5										" " " NO "
6										" " SIN. WITH "
7										" " " NO "
8										VENT FRAMES (If milled)
9										SCUTTLE FRAMES " "
10										LOUVRES " "
11										STORE FRONTS
12										SASH, DOUBLE HUNG
13										" CASEMENTS
14										" FIXED
15										SKYLIGHTS
16										GLASS (If not with other items)
17										DOORS, EXTERIOR
18										" INTERIOR
19										" SCUTTLE OR BATTEN
20										STAIRS (If milled)
21										RAILINGS "
22										PLATE RAIL "
23										DRAIN BOARD (See Interior Trim)
24										BENCHES " " "
25										COUNTERS " " "
26										COLUMNS (If milled)
27										CORNICE " "
28										FRIEZE & ARCHITRAVE (If milled)
29										SOFFITS (If milled)
30										CABINETS AND CASES
31										
32										
33										
34										
35										
36										

Fig. 7. Checking List for Millwork and Glass.

for building estimates. It will be noted that this form is arranged to indicate whether any particular item is to be furnished by the general contractor or whether it forms a subcontract. The list also shows whether materials or labor, or both, are required. In the column headed "Required," under each subdivision, a check mark is placed opposite each item for which there is anything to be billed. In the column headed "Check List" there is placed the number of the sheet on which will be found a detailed check list for the item. For some items no detailed check list will be needed. In the column headed "Itemized" there is placed the number of the sheet on which the billing of the item begins. When the billing for an item of the general list has been completed, a second check mark is placed in the "Required" column, for which purpose it is desirable to use a different color than that used for the first check. The writer uses black for the first checking and red for the final checking.

In billing the items, all of the materials may be itemized first, then the labor, and to the summary of these may be added the subcontracts. In this case the final checking proceeds down each vertical column in succession. When it is desired to bill the items so as to show the complete cost of materials and labor for the separate items, the checking proceeds across the horizontal lines in succession. Upon completion of the estimate the checking list serves as an index for reference to the itemized bill.

A detailed checking list for concrete is shown in Fig. 2. Similar detailed checking lists for lumber and millwork are shown in Figs. 3 to 7, inclusive. It will be noted that the detailed lists differ from the general list in that "Places" is used as a column heading in the detailed lists where "Check List" is used in the general list. In the column headed "Places" there is inserted the number of places on the job where the corresponding item occurs. Certain items occur on more than one list, since for different cases they will need to be billed in different ways, depending usually upon whether the materials required can be purchased from stock or whether they are special. In case all of the interior trim is obtained from the mill the lists for interior trim and for millwork virtually form a single list. If some item, such as "Ceiling," is not included in the items to be obtained from the mill, whereas the balance of the items on the list come from the mill, no confusion would ordinarily occur, since the items to come from the mill would be checked in the column headed "Subcontracts," and the item "Ceiling" would be checked under "Items Furnished by General Contractor."

For reinforcement, a list having the same items as the list for concrete is suitable. For other items of the general classification, such as brickwork, stonework, etc., detailed checking lists are not difficult to prepare. The lists for lumber are given in detail because the writer has had considerable difficulty in developing lists which were satisfactory.

Maximum Stresses in Tension Reinforcement.

[illegible] tension reinforcement in reinforced concrete [illegible] are entirely disregarded in the ordinary beam formulas. These formulas only take into con- [illegible] in tension and the distance of the center of [illegible]

Some authors state that, as the area of the steel is very small compared with the cross section of the beam itself, the stress in the metal is practically uniform, and they therefore consider the stress as uniform over the steel area.

In the first place, this comparison is not fair. It should be remembered that more than half of the cross section [illegible] phraseology—*inert*, and that less than half of the cross section of the concrete is supposed to sustain any stresses. The comparison, to be fair, should be made between the area of the steel and that part of the concrete which is in compression. On this basis the difference between the area of the steel and that of the concrete is not so great, although it remains rather large.

In the second place, the comparison is not scientific, for the stresses do not depend on the size of the *areas* but on the distances of the layers of the materials from the neutral axis of the beam. Thus, a proper comparison would be between the ratios of the depths of the materials to the distances of their respective centers of gravity from the neutral axis of the beam.

Besides, the assumption that the stresses are uniform over the steel is *diametrically opposed to Hooke's law*.

It is an elementary principle of the theory of beams that the strain varies as the distance of the layers or fibers of the materials from the neutral axis of the beam, and it follows, by Hooke's law, that the stresses must also vary in the same way. It is a well known fact, ascertained experimentally, that, within

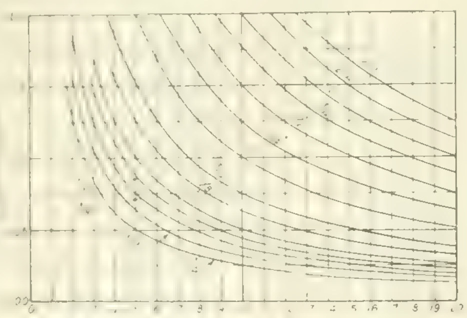


Fig. 1. Diagram Showing Effect of Considering Tensile Stress in Steel to Act at Center of Gravity of Steel Area in Reinforced Concrete Beams.

certain limits, steel is one of the few materials which very closely obeys Hooke's law, whereas concrete is *not*. Therefore it is contrary to both theory and experience to assume that the stress on the steel will be uniform, and it is the height of inconsistency to assume at the same time that the stress in the concrete will not be so.

That the steel in the tension side of a beam is often over-stressed by the use of the ordinary formulas will be clearly seen by referring to Fig. 1. The abscissas represent the distances of the centers of gravity of the steel area below the neutral axis, while the ordinates give the coefficients which must be multiplied by the stress assumed in the ordinary formulas in order to obtain the real maximum stress in the steel. The figures which designate each curve represent the distances of the lowest fiber in the steel below the center of gravity of the steel area.

If we let "y" represent the distance of the center of gravity of the steel from the neutral axis, and "s" that of the center of gravity from the lowest fiber of the steel, it is evident that, in order to deviate as little as possible from the stresses assumed in the formulas, the ratio s/y must be kept as low as possible. It follows that, apart from other considerations, the smaller the size of the reinforcement used, the nearer will the stresses in the steel approach the assumed stresses. For this reason, such fabrics as expanded metal come nearest to fulfilling the conditions of the ordinary equations than any other kind of reinforcement, as their depth is so small that even for a thin slab the ratio s/y is always a very small quantity.

The probability of the steel being over-stressed is greater, for the same area of steel, for some kinds of deformed bars and for square bars with one corner pointing down than for round bars or square bars laid on one side. The danger is also greater in shallow beams and slabs than in deep beams (except when the steel is placed in two or three rows), for, whereas the variation in depth of beams is very large, the range in the size of the bars is very limited, and the ratio s/y is likely to decrease as the depth of the beam increases, even if larger bars are used for the very deep beams.

Again, the steel in T-beams is more likely to be over-stressed than that in solid slabs or rectangular beams, because, for the same width of stem of the T-beam as the width of the rectangular beam, the area of the steel in the T-beam will be considerably greater than the area in the rectangular beam, which means that larger bars must be used for the T-beam or the bars must be placed in two or three rows. The result is that the ratio s/y will be greater for the T-beam than for the rectangular beam.

This crowding up of steel in T-beams and ribs of hollow tile slabs is, in the opinion of the writer, at least a contributory cause of the failure of several structures, for the addition of a ton or so to the stresses assumed in the steel is not a trifling matter by any means.

Table I shows three examples of hollow tile slabs taken from the common practice of three companies specializing in that kind of construction. It clearly shows how largely the maximum stresses often exceed the assumed stresses in the steel.

In the first example the stress assumed is very conservative, but the injudicious use of bars with a comparatively large depth causes the stress in the steel to exceed by 20 per cent the assumed stress. In the other examples the bars used have a smaller depth, and consequently the excess in stress is not so large.

To devise a simple formula which would take into account the maximum stresses in the steel is practically impossible, for such a formula would involve the moment of inertia of the area of steel about an axis through its center of gravity, and this is a cumbersome quantity with which to deal. But it would be an easy matter for a designer to increase the area of the steel or the depth of the beam, found by the ordinary formulas, in the ratio of distance from neutral axis to bottom of steel to distance from neutral axis to center of gravity of steel.

Very truly yours,

M. J. Lorente, C. E.

Lynn, Mass., Aug. 27, 1914.

TABLE I.—ACTUAL AND ASSUMED STRESSES IN STEEL.

Slab.	n.	fc.	fs.	y.	s.	s/y.	Max. fs.
[illegible] 1 in. concrete	15	650	16,000	2.35	0.47	0.200	19,20[illegible]
[illegible] concrete	15	750	20,000	2.70	0.25	0.093	21,90[illegible]
[illegible] ins. concrete	15	750	18,000	2.90	0.31	0.107	20,00[illegible]

Chilean Road Work.—A U. S. consular report states that 415 miles of cart roads were repaired and 58 miles of new highway constructed in Chili during 1913, at a cost of $497,668. Four highway bridges are now under construction and 31 others planned. It is stated that at present there are 823 bridges in Chili.

Time Estimates of Road Construction.—Division engineers of the New York State Highway Department have been directed in submitting estimates for construction to include hereafter an approximation of the number of working days required to complete each contract. This will enable the department to specify time limits on contracts more accurately, as the local engineers who work up the field notes are unquestionably in a better position to weigh carefully the elements which enter into the building of a piece of road.

SEWERAGE

The Milwaukee Sewerage Problem and the Sewage Treatment Testing Station.

As long ago as in 1876 the Milwaukee Board of Public Works reported upon the dangerous sewage pollution of the rivers flowing through the city. This report was followed by the employment in 1878 of a commission to study and report upon the best method of preventing the pollution. A system of intercepting sewers along two of the rivers was recommended, and constructed in part from 1880 to 1886. In the latter year Mr. George H. Benzenburg recommended that the Milwaukee River be flushed with a large volume of water pumped into it from Lake Michigan by means of low lift screw pumps. This was carried out under his supervision in 1888. Another commission reported in 1889 and as a result the Kinnickinnic River flushing works were built to prevent its pollution from sewage discharged into it.

In 1909 a commission, composed of Messrs. John W. Alvord, George C. Whipple and Harrison P. Eddy, was created to study the whole problem of collecting the sewage from the sewers already built and finally disposing of it. This commission made its report in 1910, following which the State Legislature of 1913 passed an act authorizing the creation of a sewerage commission composed of five citizens to study the problem and to carry out such works as were found necessary for its solution. The report mentioned was published in substance in ENGINEERING AND CONTRACTING of May 24, 1911.

Thus, for 38 years this question has been energetically studied by citizens and engineers, and a large sum of money has been expended in alleviating the troubles as they arose, but prior to the creation of the present sewerage commission, no such broad powers had been granted the city which would enable it to carry to a conclusion all the works recommended.

THE PRESENT SEWERAGE COMMISSION.

The present Sewerage Commission consists of Mr. George H. Benzenburg, chairman, a prominent civil engineer and a former city engineer of Milwaukee, and four leading citizens. All were appointed by the mayor, Dr. G. A. Bading. Mr. T. Chalkley Hatton is chief engineer of the commission. The following matter is from a paper by Mr. Hatton, presented at the annual convention of the American Society of Municipal Improvements, held at Boston on Oct. 6-9.

There are three conditions under which this commission is working which are somewhat unusual, but which are necessary to insure success in all undertakings of this character.

Its members are appointed to hold office during the entire period it may require to carry out such works as may be planned. The funds necessary to carry out the work must be provided by the City Council by a special levy of taxes upon real and personal property up to an annual limit of one mill of assessed valuation of such property. The expenditures made are solely under the direction of the commission, subject to the approval of City Comptroller. The commission employs all assistance needed, subject to the civil service rules of the city, and fixes all salaries with one exception. The rate of salary of its secretary must be confirmed by the City Council.

The responsibility for properly and satisfactorily solving this problem is thus put squarely up to the Sewerage Commission, as it should be, and it is the only method by which public works of a special character can be carried out satisfactorily to the citizens of a community.

LOCAL CONDITIONS.

The city of Milwaukee is located upon the west shore of Lake Michigan, and is built around the mouths of three rivers, of which the Milwaukee River is the chief, and which flows directly into Lake Michigan. The Menomonee and Kinnickinnic Rivers join the Milwaukee River near its mouth. The Milwaukee River flows through the northeastern section of the city, the Menomonee flows almost due east, and the Kinnickinnic from the southwest towards the northeast.

Each of these rivers is navigable for about two or three miles from its mouth for boats drawing from 18 to 20 ft. of water, and all are partially bulkheaded as far as navigation extends. Several slips or inner docks have been built in the Menomonee and Kinnickinnic valleys, and a dam across the Milwaukee River about three miles above its mouth.

The drainage areas of the three rivers are approximately 872 square miles, of which the Milwauke has 715, the Menomonee 125 and the Kinnickinnic 32. The estimated average run-off of these rivers in gallons per 24 hours is as follows: 221,000,000 in Milwaukee River, 57,000,000 in Menomonee River, and 15,000,000 in Kinnickinnic River, while the minimum flows have been estimated to be 2,000,000 from the Milwaukee, 1,300,000 from the Menomonee and 650,000 gals. from the Kinnickinnic River.

There being no ebb and flow of tide in the lake, under minimum flow conditions there is very little current in the rivers except such as is occasioned by wind action upon the waters of the lake.

The average water consumption at the present time is about 48,000,000 gals. per day. Assuming the water consumption to be equal to the sewage run-off, as is so frequently the case, and the sewage being discharged into these three rivers within the city limits, it takes no great elasticity of imagination to picture the sanitary conditions of these rivers under such an assumption.

The valleys of the rivers are narrow, and are from 6 to 20 ft. above the level of the water. The large part of the present business and industrial sections are located in these valleys, while the domestic areas are located upon the table lands which rise to a height of 160 ft. above lake level; the natural drainage being excellent and falling away from the lake.

The increase in population of the city of Milwaukee since 1850 has been very rapid, although at a fairly uniform percentage, indicating no boom but a substantial growth. In 1850 the population was 20,000, in 1910 373,857, while in 1914 the population is 410,000. It is estimated that in 1930 the population will be 588,000 and 862,000 in 1950. There are 26 square miles within the present city limits, and the limits for 1950 will probably contain 59 square miles.

The city is largely industrial, its chief industries being killing and packing meats, the manufacture of leather, engines and machinery, steel and, last but not least, the production of beer.

The meat killing and packing, leather and beer industries consume enormous quantities of water and produce a sewage which is much more difficult to dispose of than the normal domestic and industrial sewage.

The leather manufacturers use a daily average of 4,250,000 gals., the packing houses 1,000,000, and the breweries 6,000,000 gals., which does not include the water taken from wells and rivers into these plants, which is an additional large amount.

The foregoing data and description have been given as briefly as possible to permit of a better understanding of the problem involved in collecting and disposing of the sewage of the city.

The city of Milwaukee was much more fortunate and farseeing than most of its neighbors, in that early in its career its officers laid out a complete system of sewers designed to carry off both the sewage and storm-water from a population of nearly one-half million persons. As the growth of the city increased, this system has been extended from time to time until it provides for nearly all the present population and embraces about 400 miles.

This system is on the combined plan and carries both storm-water and sewage in the same conduits, which discharge into the three rivers, with two exceptions; these two sewers discharging into the lake in the southeastern part of the city. This system has 75 outlets into the rivers, of which 47 discharge into the Milwaukee river.

The dam across the Milwaukee river, which is about 14 ft. high, backs the water up for two or more miles, there being very little current in the river during minimum run-off. Public parks adjoin the river on both sides, the water is used for boating, swimming and other sports; it is, therefore, important that sewage pollution be prevented in this portion of the river.

To this end intercepting sewers have been built upon each side of the river from near the northern city limits to a point below the dam. These sewers are large enough to carry off all the sewage and a large proportion of the storm-water, the balance passing through overflows into the river.

The condition of the Milwaukee river was so unsanitary in 1888 that the city built, under the supervision of Mr. George H. Benzenberg, then city engineer, a flushing tunnel, 12 ft. in diameter and 2,543 ft. long, connecting the lake with the Milwaukee river at a point near the toe of the dam. A pumping station is located on the lake end of the tunnel in which a screw pump is located and by which 324,000,000 gals. of lake water per day can be pumped into the Milwaukee river. This pump raises the water from 2 to 3 ft., and at the time it was installed, had the largest volumetric capacity of any pump in the world.

This large volume of water, discharging into the river at the head of navigation, not only creates a current towards the mouth but affords sufficient dilution to increase the dissolved oxygen in the river from 0 to 6 to 8 parts per million.

This tunnel, and its operation, have been a great success from the start, but the sewage flow has increased so much during the 26 years the flushing tunnel has been in operation that the sanitary conditions of the river is again causing great concern.

The Kinnickinnic river is flushed by similar works, which discharge the same volume of water into the river at the head of navigation. This tunnel is 12 ft. in diameter and 7,185 ft. long, and the pump is located at the river instead of the lake.

In order to prevent the pollution of the Menomonee river within the city limits, an intercepting sewer was built in its valley, from the western city limits to Jones Island, which is a low flat strip of ground lying between the mouth of the Kinnickinnic river and the lake. This sewer was built not only large enough to carry all the sewage flow which might otherwise discharge into the river, but of sufficient capacity to carry an equal volume of river water which is admitted into the sewer through four gates connecting the river and sewer at points a considerable distance apart; the idea being not only to flush the sewer by the river water, but to create a current in the river by taking the water out and allowing the lake water flowing down the Milwaukee river from the flushing works to take its place.

At Jones Island a pumping station was built by which the sewage and river water are lifted about 10 ft. and discharged into the lake near the mouth of the Milwaukee river. This pumping station handles about 45,000,000 gals. of liquid per day, which has been going into the lake for the past 28 years.

The water supply for the city of Milwaukee is procured from Lake Michigan through a subaqueous tunnel 7½ ft. in diameter and 3,146 ft. long, from the outer end of which two lines of 60-in. cast-iron pipe extend 5,000 ft. further to a point where the water is 60 ft.

deep. This intake is about 3.5 miles, on a direct line, from the mouth of the Milwaukee river, where all the sewage from the city is being discharged, and as there is very little current in the lake, and much of what there is is directed by the winds, it can be readily imagined that the water supply is in some danger of becoming polluted from the sewage. In fact, the possibility is so great that the water used is treated with a sterilizing agent, which so far has proved satisfactory.

THE PROBLEM.

The problem which the present sewerage commission has to solve is to collect all the sewage which now discharges in the three rivers and lake, both within and without the present city limits, by means of intercepting sewers, carry it to some suitable site where works are to be built for treating it, and carry the treated effluent to some point in the lake where it cannot threaten the purity of the water supply or cause a nuisance to the riparian rights to the lake.

The Menomonee Sewerage Commission, consisting of Messrs. Alvord, Whipple and Eddy, presented a very elaborate and well studied report in 1910, in which it was recommended that the sewage be collected by a system of high and low level sewers, carried thereby to a tract of ground containing about 26 acres, located in the southern portion of the city alongside of the Kinnickinnic river, where it would be given preliminary treatment by chemical precipitation, and disinfectants, and the effluent carried by a 13 ft. conduit in tunnel to the lake front at a point about one-half mile south of the present limits, from whence, for the present, it would be carried through a subaqueous conduit, about one mile from the lake shore, where it would be dispersed into 35 to 40 ft. depth of water, through two steel pipes 5 ft. in diameter and 3,000 ft. long, built upon the lake bottom.

If necessity arose subsequently for a more thorough treatment of the effluent from the precipitation tanks, sprinkling filters were to be built along the lake shore at the junction between the land and subaqueous tunnels. Land being made therefor along the lake front.

The report of this commission left many important points for thorough study before deciding upon the final construction, chief of which were the system of treating the sewage, the method of disposing of the sludge, the site for the disposal works and the location and sizes of the intercepting sewers. As before stated, the system of sewage disposal recommended by the commission was chemical precipitation followed by sterilization.

The site recommended was what is known as the Chase tract, of about 26 acres, located on the Kinnickinnic river and largely surrounded by dwellings and manufacturing industries. The sludge was to be disposed of by hauling to the lake and dumping it therein about 10 to 20 miles from shore.

Very soon after this report was made public objections arose as to the method of sewage and sludge disposal. The commission assumed the process recommended would generate [illegible] of sewage treated, or about 120,000 gals. per day from the present population, 250,000 gals. per day from the probable population in 1930, and 500,000 gals. per day from the population anticipated in 1950. The handling of this enormous volume of sludge by steamer to a point 10 to 20 miles in the lake looked like a large item of daily expense; but aside from this, it was feared that the present international agitation against the pollution of the waters of the Great Lakes would finally result in estopping Milwaukee from thus disposing of its sludge, whereupon other treatment and other disposition would have to be resorted to, and as there was no waste land in the vicinity of the Chase Tract upon which the sludge could be deposited without nuisance to neighboring property rights, it was deemed expedient to make a careful and exhaustive study of other processes of sewage and sludge disposal before finally adopting any process.

The chief opposition upon the part of many of the citizens of Milwaukee, however, was to the site chosen for the disposal works. There is a strong sectional feeling in the city, as is usual in all cities of large magnitude, and the "south siders" objected to having the filth of the "north siders" dumped down in their midst, especially as, in their opinion, the topography of the city presented another site for disposal works which would be a menace to no one.

The present sewerage commission, therefore, decided to build a testing station to try out those processes of sewage treatment applicable to large installations; to take the raw mixed sewage from the pumping station located at Jones Island as fairly representing the normal sewage to be disposed of in the ultimate plant; to carry out these tests with vigor for twelve months and from the results thus obtained design a sewage disposal plant which would, without any reasonable doubt, best fulfill the conditions for the city of Milwaukee.

THE EXPERIMENT STATION.

This testing station has just been completed and its operation has begun, under the direct charge of Mr. William R. Copeland, Chief Chemist to the Commission, assisted by an assistant chemist, a bacteriologist and an engineer.

The several processes being tested are as follows:

Sedimentation and sludge digestion by means of an Imhoff tank, followed by aeration through sprinkling filters, final sedimentation and the sterilization of the final effluent by means of liquid chlorine.

Sedimentation, aeration and precipitation by means of a slate tank, followed by aeration through sprinkling filters, final sedimentation and sterilizing the effluent.

Chemical precipitation by the injection of iron and lime solutions into the raw screened sewage before passing through tanks, followed by aeration through sprinkling filters, final sedimentation and subsequent sterilization.

Sterilization and precipitation by means of lime solution added to electrolytic treatment.

Fine and coarse screening followed by the several tank and sterilizing treatments.

Tests for sludge disposition by dewatering by means of natural filtration through sand and stone beds, and by means of pressing into sludge cakes.

The apparatus connected with the several processes are arranged to treat a maximum of 1,000,000 gals. of sewage per 24 hours, which is a much greater quantity than is provided for in the usual testing station, but it was desired, when laying out the station, to make the several units of such size as would insure getting representative results.

The Imhoff tank is 26 ft. deep and 14 ft. in diameter, and provides for a running through period of from 1¼ to 2½ hours.

The slate tank is 22 ft. long, 13 ft. wide and 6 ft. deep, filled with slates spaced ½ in. to ¾ in. apart, standing vertically, and has a liquid capacity of 10,000 gals. Working in six to eight-hour cycles, this gives a capacity of 40,000 or 30,000 gals. per day. Compressed air is provided for the slate tank, and is to be fed to the tank under low pressure, lifting the sewage from the tank through 2½-in. pipes terminating about 4 ft. above the normal sewage level. The sewage, mixed with entrained air, discharges from the end of these pipes, falls upon a slate roof built over one-half the tank, and flows back into the opposite side of the tank, from which it started; thus the air keeps a constant current in the tank during the five or seven hours the sewage is contained therein, during which time intensified oxidation progresses and the suspended solids in the sewage become attached to the surface of the slates. At the end of the cycle the liquor is drawn off and discharged into a tank holding 10,000 gals., from which it is discharged, at a uniform rate, upon a sprinkling filter, the sludge being drawn off at desired intervals to a sludge tank, thence into a sludge bed partially filled with crushed stones and fine sand through which a portion of the liquor in the sludge is drained off.

This is practically an untried process and it is expected to reduce the area of sprinkling filters from three to four times what is necessary to treat the effluent from any other preliminary tank process. The process was suggested from experiments conducted for the past two years at the Lawrence Experimental Station of Massachusetts State Board of Health, under Mr. Harry Clark, Chief Engineer.

The chemical precipitation tanks are in duplicate, 32 ft. long, 10½ ft. wide, and 10 ft. average depth, each with a capacity of 25,000 gals. The iron, alumina and lime solution tanks are so arranged as to permit the solution to be added to the raw sewage after it passes through the screen chamber and be mixed with the sewage before it reaches the tanks.

Each of the processes is provided with separate sludge beds. There are, 1 for the slate process, 2 for the Imhoff tank, and 14 for the precipitation tanks. Each of these beds is underdrained and contains about 9 ins. of crushed stone, covered with a layer of sand 3 ins. deep.

There are three kinds of screens provided: (a) A coarse grid screen with 1-in. openings placed over the outer end of the intake pipe; (b) A bar screen with ½-in. openings through which the sewage passes after passing through the grit chamber; and (c) a movable wire mesh screen, having ten meshes to the inch, moving through the sewage at the rate of 2½ ft. per minute.

The first two screens are to be cleaned by hand, the movable screen is to be cleaned by compressed air, blown upon the underside of the screen at a pressure of 100 lbs. to the square inch.

All parts of the plant are arranged to enable a close record to be made of every operation. Apparatus for measuring the sewage treated by each process, the amount of electric current used for power, amount of air used at the different pressures, weight of sludge and screenings removed, amount of chemical solutions used and steam consumed can be determined with ease and accuracy.

A laboratory, thoroughly fitted up for carrying on the chemical and bacteriological determinations, has been constructed alongside the plant, and the commission hopes that in twelve months some valuable information will be secured which will advance the science of sewage disposal and enable it to determine the best system for Milwaukee.

Like the testing station built for the Borough of Brooklyn, all apparatus and plants connected with the several processes are built above ground, thus permitting of a much more satisfactory inspection of the processes. This is particularly true of the screens and grit chambers from which it was desired to get most accurate results in view of the recent importance attached to fine screening.

The sewage proposed to be ultimately treated will be from the present combined system of sewers, and the intercepting sewers are being designed to carry about three times as much storm water as sewage; the ultimate sewage disposal plant will, therefore, have to take care of a large volume of the first street wash, containing both organic and mineral solids. In order to measure, and satisfactorily observe, the sedimentation of the grit under such conditions, the grit chambers were built at the top of the operating house, 35 ft. above the ground. The raw sewage, after passing through the rough grid screen placed at the intake end of the suction pipe, is forced by two centrifugal pumps into the grit chambers, which are so baffled as to control the velocity of flow through them. Scales are provided alongside of the grit chambers to measure the grit thus precipitated.

After passing the grit chambers, the sewage can be turned upon either the fine movable screen or the secondary grid screen, or upon both, from whence it flows through orifice boxes to the several processes, the waste, if any, passing through an independent orifice box.

The use of compressed air for cleaning the screens was tried out with much success in the Chicago Stock Yards, and it is to be tried out here with every apparatus necessary for measuring its cost under different processes and volumes.

All of the moving parts of the plant are operated by electric current generated by two 25 K. W. D. C. generators, built in the main sewage pumping station. Steam lines for heating the buildings and the several tanks exposed, have been run throughout the works securing the steam from the three boilers in the main pumping station.

A weir, 20.35 ft. long, has been built across the wooden channel which carries the sewage from the main sewage pump and over which from 30 to 50 million gallons of sewage per day passes, and an automatic liquid recording gage has been set up to measure the total flow for the purpose of determining at all times the proportion between the total sewage pumped by the main works and the sewage treated in the testing station.

The total population contributing to the main pumping station has been taken, the volume of water consumed and run-off per capita secured; the total industrial and commercial run-off per acre has been measured and the analysis made of the effluents from the principal industrial plants contributing to the sewage station.

This information could all be obtained with more than average accuracy as all water consumers connected with the public water supply are metered. In addition to this we have maintained for several months liquid recording gages upon some of the larger main sewers by which means we have secured good records of run-off.

It might be interesting to note here that we had one very unusual situation by which we could determine with unusual accuracy the ratio between run-off and consumption. A sewer 96 ins. in diameter at its outer end drains a section containing 1,530 acres and 62,600 persons. There are no industrial nor commercial uses for the water in this district, it being purely domestic, completely built up, with an average of 50 persons per acre, excluding street areas.

Upon plotting the curves of run-off and consumption for the same day of the week and for several consecutive weeks, a curve showing the underground leakage flowing through the main sewer has been clearly deduced. Up to date this leakage has diminished from 40 gals. per capita per day in April, 1914, to 15 gals. per day in August. The Winter being an unusually open one in this section, followed by an early Spring, the underground maximum run-off was earlier than usual. During June there were rains during 26 days. The underground flow showed up very decidedly during the records taken the latter part of June and first part of July.

Studies are now being made and partially completed for determining the sizes and locations of the several intercepting sewers necessary to intercept the dry-weather flow and a portion of the storm flow, and carry it to the disposal works. The plans for the portion of the disposal works to be built in the near future will provide for treating 100,000,000 gals. of sewage; the ultimate works will treat about 158,000,000 gals.

INTERCEPTING SEWERS.

The intercepting sewers are being designed to carry to the works a maximum of 320,000,000 gals. of liquor, of which over one-half is storm water.

The plan provides for low-level and high-level intercepting systems. The sewage collected by the low-level sewers must all be pumped about 35 ft. high to get it into the disposal works. This represents about one-quarter of the total sewage and storm-water collected, and is collected from the areas lying between contour 30 above and the lake level.

This low lying section is largely adjacent to the three rivers; therefore, the low-level interceptors will be built as near the river banks as existing structures and streets will permit. The greater portion of the river banks being bulkheaded, and either used for wharves or have large commercial houses and manufacturing industries built abutting thereupon, the location and construction of intercepting sewers will be attended with much difficulty and expense. Naturally, the sub-strata adjacent to the rivers is not of the most substantial material and will require special foundations. In order to carry the interceptors to the sewage disposal plant, the rivers will have to be crossed several times. This will be done by inverted siphons, built deep enough to secure firm foundation, which we have found to be from 60 to 80 ft. below water level.

The high-level intercepting sewers will be located to intercept all the dry-weather flow and that portion of the storm-water desired originating above contour 30, and will be extended to the probable limits of the city in 1950. The flow through these interceptors will be carried to the disposal works by gravity.

The design of the main outfall sewer to carry off the effluent from the disposal works has not been begun, but from present indications, it will be a conduit about 13 ft. in diameter, part of which will be sub-aqueous, terminating at a point under the lake where a depth of 30 to 40 ft. of water can be obtained. Its location, however, depends entirely upon where the disposal plant is ultimately located and the character of effluent obtained from the process of disposal finally adopted.

Even after the dry-weather flow is intercepted by means of the proposed system of intercepting sewers, great quantities of putrescible organic matter will reach the rivers during every storm. With the small dry-weather flow through these rivers, the decomposition of this matter might cause them to become a nuisance to the adjacent neighborhood.

To overcome this condition, the Milwaukee and Kinnickinnic rivers have already been provided with ample facilities for flushing by means of the flushing tunnels already built, and which were briefly described at the beginning of this paper. In order to overcome the condition in the Menomonee river, it is proposed to build similar flushing works, taking the water from the lake, carrying it through a tunnel of sufficient capacity to carry from 400 to 500 million gallons to a pumping station located along the river bank, there screw pumps would be built to lift this water a few feet and discharge it into the river at the head of navigation, and at the head of the slips connected with the river; the current thus produced in the river would carry all matters in suspension to the lake, and the clear lake water would add dissolved oxygen to the river water and thus assist in its purification before it reaches the lake.

While the approximate cost of building all works necessary to carry out the general plans has not been made, because the studies and plans have not been completed, preliminary studies already completed indicate that the cost of building the intercepting sewers necessary to take care of the expected population in 1930 of 588,000, will be $2,661,360; disposal plant, consisting of Imhoff tanks, sludge drying beds and sterilizing plant, $1,188,928; pumping plant for low-level sewage, $294,400; main outfall sewer from disposal plant to lake shore, $1,209,600; sub-aqueous outfall sewer to deep water, $1,150,000; Menomonee river flushing works, $1,162,000. The last two items are taken from the report of the Menomonee River Commission. The total approximate cost so far as now known is $7,666,288.

This estimate is based upon using Jones Island as a site for disposal works and installing Imhoff tanks and sterilizing plant without further treatment except dilution into the lake. Subsequent determinations reached from the results of the testing station may change these premises very much, and also the estimates.

It is expected that actual construction upon the low-level intercepting sewers will begin in the Spring of 1915, from which time the work will be carried on as rapidly as the necessary funds are provided. The present sanitary conditions of the rivers should be improved as rapidly as may be possible, and it would appear that sufficient study by eminent men has been given to this problem to insure its proper solution without further delay.

In closing this paper, it seems apropos to call attention to enormous expenditures contemplated in the very near future by the larger cities of the United States for treating their liquid wastes. These expenditures have been postponed from time to time for what appeared to be more urgent public works, such as increased water supply, water purification and improved street pavements, but it is being fully appreciated that the time has come when these closely built up communities can no longer, with safety to themselves, continue to discharge their untreated liquid filth at their front doors.

Boston, Mass., and Providence, R. I., completed works for satisfactorily disposing of their sewage several years ago. Columbus, Ohio, Baltimore, Md., New Orleans, La., and Atlanta, Ga., have recently completed works for treating or satisfactorily disposing of their sewage. Aside from these, no large city of the United States has really undertaken the work, although Chicago carried out works some years ago by which it was then believed its sewage could be properly disposed of, but today Chicago, New York, Cleveland, Detroit, Dayton, Cincinnati, Philadelphia, Rochester and Milwaukee are all studying plans for sewage disposal. Toronto, Canada, is also experimenting with the problem. It is, therefore, apparent that within the next decade the largest sewage works of the world will be constructed, or started, during which time many new lessons on sewage disposal are bound to be learned and the whole world benefited thereby.

The Economics of Sewage Filters.

Types of Sewage Filters.—Sewage filters may be divided broadly into three classes:

(1). Intermittent sand filters or their equivalent. These consist of a body of sand or fairly pervious material of other kinds. The sewage is distributed over the surface of this porous material, and at the bottom the filtered sewage is collected in underdrains. In order to get the benefit of oxidation in the pores of the sand bed, the application of the sewage to the filter is intermittent, with periods of rest and aeration.

(2). The second type is the so-called "contact" filter. This consists of a body of practically any thickness of stone or equivalent material, such as large-sized gravel, pieces of porcelain, brickbats, cinders, or almost any coarse-sized granular material. The sewage is applied to such a filter either from the bottom or from the top, so as to fill the bed. The sewage is allowed to stand in this filter bed for a given time. It is then discharged and the empty bed is allowed to stand for a period.

(3) The third type is the so-called "sprinkling" filter. This consists of a body of stone of a minimum depth of 5 ft., on which the sewage is sprinkled or sprayed and spread by nozzles and distributed in small quantities so that the sewage trickles down over the stones and is collected at the bottom.

All three types of filters effect the purification of the sewage in the same way. Through the action of the bacteria present in the filter bed the sewage is to some extent oxidized and the organic matter is broken up. Unstable forms of matter are changed into more stable forms. While the exact form of action is unknown, it is believed that the three types of filters act in the same way, and the difference is rather a mechanical one of form of application, rather than one of principle of action. The present article, which is a reprint of a paper by Mr. George W. Fuller, presented before the annual convention of the American Society of Municipal Improvements, discusses the economic considerations affecting the choice of a filtration method and governing the proportioning of the major dimensions of sewage filter beds.

Performance of Filters.—The output of a filter of any type, measured at any suitable purification unit, is largely a question of local conditions. It depends upon the nature of the sewage, the nature and fineness or coarseness of the filtering material, the method of application of the sewage to the filtering material, temperature, atmospheric conditions, and many other factors. The intermittent sand filter is best used when it is desired to have a very high degree of purification. The other types, the contact filters and sprinkling filters, are used for a rather lesser degree of purification. It is to be understood that the rate of application of the sewage to the various types of filters must be properly proportioned to the ability of these filters to take care of the sewage. By using a suitable rate under suitable conditions any type of filter can be made to give any degree of organic purification that may be desired.

Rate of Application of Settled Sewage to Filters.—The question of the rate of application of sewage to a sand filter is largely tied up with the question of preliminary treatment in the way of tankage or screens. The following tabulation, quoted from Mr. Fuller's book, "Sewage Disposal," gives for several cities in Massachusetts the population whose sewage can be treated per acre of filter bed, and the time of detention in preliminary sedimentation tanks, storage wells, pump wells, or other means of storage. These figures are not to be taken to represent present conditions.

	Period of detention, hours.	Population per acre of filter.
Andover	1½-[illegible]	950
Brockton	12	1,160
Clinton	12	[illegible]
Framingham	12	375
Gardner (old)	1½	1,310
Gardner (new)	1½	2,000
Pittsfield	12	665
Stockbridge	8	220
Worcester	1½	1,390
Average of all	6.7	937

The Baltimore Sewerage Commission in 1906 estimated that, using a sand filter with 3 ft. of clean sand over the gravel, an allowance of 150,000 gals. of 6-hour settled sewage per acre in 24 hours, corresponding approximately to 1,200 people per acre, would be a proper rate.

Data for contact filters are relatively scant from American practice, and while many English data are available, the differences, owing to the difference in the strength of the sewage, makes such data rather dangerous as [illegible]

A series of experiments in Columbus, Ohio, indicated that 5-ft. deep stone filters, on the contact principle, could safely be operated at an average rate of 600,000 to 700,000 gals. per acre per day. Reducing this to a 4-ft. depth will give about 500,000 gals. per acre per day, which, on the basis of 100 gals. per capita per day, would give a loading of approximately 5,000 people per acre of stone bed.

A series of tests made at Lawrence on contact beds of various depths from 24 ins. up to 18 ft. showed an average output of some 700,000 gals. per acre per day for a depth of stone on an average about 5½ ft. This is equivalent to an output of about 135,000 gals. per acre for each foot of depth of stone, or, for a 4-ft. depth of bed again, is equal to about 500,000 gals. per acre per day, or say a loading of about 5,000 people to the acre

The contact filter installation at Plainfield, N. J., with 3.6 acres of stone bed 4½ ft. deep, gave in 1910 an output on an average of 1,700,000 gals. of sewage per day. On the [illegible] per day, this will correspond with a 4-ft. bed, to about 4,200 people per acre of filter.

For sprinkling filters much more satisfactory data can be had. Sprinkling filters have been used very extensively in this country of recent years and their ratings can be fixed with a good deal more dependence than in the [illegible] ber of plants or projected plants giving the depths of the stone bed of a sprinkling filter and the loading in population per acre fol[illegible]

	Depth [illegible]	[illegible]
Atlanta	[illegible]	[illegible]
Reading	[illegible]	[illegible]
Columbus	[illegible]	[illegible]
Baltimore	[illegible]	[illegible]
Montclair	[illegible]	[illegible]
Philadelphia	[illegible]	[illegible]
Fitchburg	[illegible]	[illegible]
Mount Vernon	[illegible]	[illegible]

The average of all these shows a 7-ft. deep bed and an average [illegible] population to the acre.

Not considering special conditions and just taking fair figures, we may safely state the following:

Intermittent sand filters, 3-ft. bed of sand, loading 1,000 population per acre.

Contact filters, 4-ft. depth of stone, loading 5,000 population per acre.

Sprinkling filters, 7-ft. depth of stone, loading 19,000 population per acre.

The rates of loading, then, for these three types of filters, are in the ratio 1, 5, and 9.

Cost of Sewage Filters.—Costs of construction are so much affected by local conditions, such as the amount of excavation necessary, the cost of various classes of materials, the distance from which various classes of materials must be obtained, details of local construction conditions, such as competition, class of work required, and others, that comparative costs for different localities are only to be used with great discretion, and individual cost and even averages are only a guide to comparative costs in various places. Having this limitation in mind, we will examine in a rough way the cost of various types of sewage filters on the per capita basis.

The average cost of the nine Massachusetts intermittent sand filters cited above is $3,260 per acre, as reported in the Massachusetts State Board of Health Report of 1903. This gives a cost per capita connected to the filters of $3.50.

The 1906 Baltimore Sewerage Commission estimates the cost per acre of filters at $6,350, these filters being suitable for a connected population of 1,200 per acre. This corresponds to a per capita cost of $5.30.

The cost of contact filters, varying, of course, with the degree of the fineness of the design, may be taken, for filters equipped with suitable convenient appurtenances, at $30,000 per acre for a 4-ft. deep bed. This corresponds with a loading of 5,000 population per acre to a per capita cost of $6.

For sprinkling filters 7 ft. deep the cost will be about $45,000 per acre. On the basis of a loading of 19,000 population per acre, the cost per capita will be $2.37.

When considering the relatively low cost of the Massachusetts sand filters compared with the estimate made of the Baltimore sand filters, it is to be borne in mind that the conditions in Massachusetts for the construction of sand filters were unusually favorable and do not represent average conditions through the country. In most places the costs would approximate more nearly those estimated for Baltimore.

Taken in a broad way, sprinkling filters are a far more economical installation in the matter of first cost. Intermittent sand filters and contact filters do not stand far apart in this particular.

Relative Costs of Different Depths.—There is not very much known about the relative advantages of filters of shallow or deep construction. The choice of depth is usually made for entirely different reasons from those of obtaining the most economical construction to obtain the desired amount of purification. Very few tests of a comparative kind have been made to give convincing information, and the interpretation of the tests has not been uniform. In some places the conclusion has been made to make filters, say, 10 ft., at other places 6 ft., and some study is worth while to determine what, if any, difference there is in the cost of such construction at different depths, and which would appear to be the better. It is to be assumed in such comparisons that sufficient head would be available in any case for the greatest depth to be considered and that pumping would not be necessitated by building filters of the greater depth.

For intermittent sand filters questions of depth do not arise. The filters are generally made as shallow as is consistent with getting proper results and sand beds are not usually [illegible] maximum. Shallower beds, even, will give about the same output as the deeper beds, and beds are made deep only so that sand may be removed for cleaning without removing the sand for a considerable period.

With contact filters it is recognized that from the nature of the action of the contact filters, where the amount of air that is drawn in between fillings of sewage is practically equal to the volume of the sewage, and where surface clogging cannot be a serious factor and may even be no factor at all, each unit of volume of the stone forming the filter, say each cubic yard, will give the same output of sewage purification, no matter what may be the depth of the filter.

From this it follows that it is economical to build a sewage filter on the contact principle as deep as local conditions of construction will permit, and the limitation of depth which it is economical to use is therefore made by the factors of earth excavation or fill and the possible head available without pumping.

When it comes to sprinkling filters, the problem becomes a little more complicated. The English experience, as recited in the Report of the Royal Commission, seems to indicate that the output per unit of volume of sprinkling filters is the same, no matter what the depth. Our experience in work in this country does not wholly corroborate this information. Our best knowledge seems to indicate that the output per unit of volume of sprinkling filters is somewhat less for deep filters than for shallow filters. For such conditions, with a relatively decreasing efficiency of the stone of the filter beds for greater depths and at the same time a relatively decreasing cost per unit volume of the stone for deep beds, there must come some point where the greatest output per unit of cost will be obtained.

The Report of the Baltimore Sewerage Commission for 1911 gives some information obtained from tests made in Baltimore as to the relative efficiency of various depths of broken stone of sizes of 1 to 2-in. stone, which is the one most commonly used. Figures obtained from that source are as follows:

	—Depth of bed.— 6 ft.	9 ft.	12 ft.
Relative stability	[illegible]	87	89
Per cent reduction of oxygen consumed	[illegible]	[illegible]	72

Giving equal weight to the relative stability and per cent reduction of oxygen consumed, we get the following:

	—Depth of bed.— 6 ft.	9 ft.	12 ft.
Relative stability	1	[illegible]	[illegible]
Per cent reduction of oxygen consumed	1	[illegible]	1.28
Average of the two	1	1.22	1.25
Relative depths	1	[illegible]	2.00
Relative value of stone per cubic yard	1	[illegible]	.63

Assuming this depth varies at a uniform rate from one end of the curve to the other, we get the following for the relative value of stone per cubic yard:

Depth of bed in feet	6	7	[illegible]	[illegible]	10	12
Relative value of stone per cubic yard	1.0	0.97	0.94	0.92	0.82	0.63

To get comparative figures, then, between the 6, 8 and 10-ft. beds the cost figures for the 8-ft. beds must be divided by 0.94, and the cost for the 10-ft. beds by 0.82, putting them all on the basis of the 6-ft. beds.

For comparative cost a number of factors such as excavation, etc., are naturally omitted, as they are not affected in all places the same way by the depth of the filter. Comparing, then, only those particular costs which are affected per unit of output by the depth of the filter, we get the following:

	Per effective cu. yd. depths. 6 ft.	7 ft.	8 ft.
Floor—Take at [illegible] cts. per cu. yd. for a 6-ft. bed	$0.40	$0.35	$0.32
Tile—Take 11 cts. per sq. ft. for any depth	[illegible]	.44	.40
Walls—Assume cost 17 cts. per cu. yd. for 6-ft. depth	.17	.17	.18
Galleries and Collectors—Assume for 6-ft. depth 25 cts. per cu. yd.	[illegible]	[illegible]	.20
Distribution—Assume 50 cts. per cu. yd. for 6-ft. depth also, as costs theoretically vary only according to quantity delivered, they must be same for all effective depths per cu. yd.	.50	.50	.50
Stone—Assume $1.50 per cu. yd.	1.50	1.55	1.60
Total	[illegible]	$3.23	$3.20

Outside factors will depend on quantity only and not on depth.

It appears, then, that there is some slight saving of cost, which, on the figures given in the above tabulation, amount to about 3 per cent in favor of the 8-ft. deep bed as compared with the 6-ft. deep bed. On the other hand, it is to be recognized that a deep bed will give a good deal more trouble with pooling and freezing than a shallow bed, and the advantages in favor of a shallow bed due to this lesser amount of pooling will be considerably more than this 3 per cent difference in cost. Taking everything into account, the writer believes that a sprinkling filter of not less than 6 ft. and not more than 7 ft. in depth will in the greater number of cases prove the most economical to use.

CONSTRUCTION PLANT
MACHINES DEVICES MATERIALS

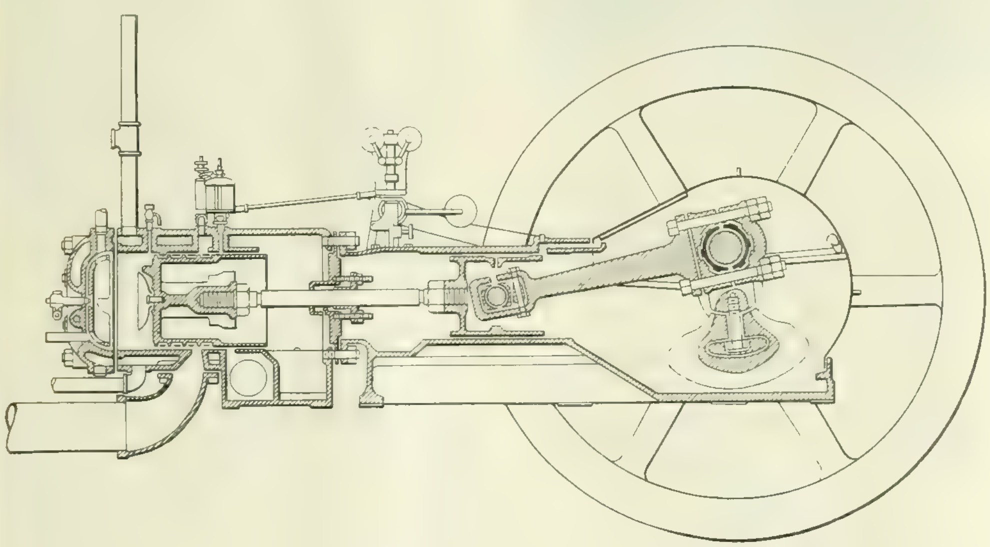
A Low Grade Fuel Oil Engine for General Service.

(Contributed.)

The engine illustrated will operate successfully on any of the ordinary fuel oils. It has no valves, gears, carburetors, mixers, oil or air heaters, magnetos, batteries, timers, switches, coils, wires or spark plugs.

The employment of a single cylinder minimizes working parts and their consequent friction. The crosshead removes from the piston head the angular thrust of the connecting rod with its tendency to wear the top and bottom of the compression chamber. The method of igniting the fuel charge is positive and simple. A thin circular plate is rigidly secured to the piston and after the engine is started fuel injected against this hot plate is gasified and ignited. By this system air only is compressed in the cylinders, the fuel is injected at the proper time and high sustained operating economies are possible.

A cam under the control of the governor rests against the collar shown on the plunger rod, the position of the cam determining and regulating the stroke of the pump and consequently the quantity of fuel injected. A hand-operated lever, also acting upon the plunger, is provided for stopping the engine.



Section of Low Grade Fuel Oil Engine.

of the cylinders more than the sides, with the result that oils of a heavy or asphaltum base will work back and under the piston rings, hardening there and causing excessive cylinder wear. With the crosshead type all bearings are accessible and by compressing in the front end of the cylinder instead of in the crank case, better compression is secured, there being no joints to offer opportunity for leakage, and the compression space is greatly reduced. Lubricating oil from the crank case cannot posturb regulation.

The cylinder is of the valveless, two-cycle low compression type. Water jackets are cast integral with the cylinders, but cover only that portion in which the combustion takes place. Like the cylinder the head is made of close-grained cast-iron and is a single piece casting thoroughly water jacketed. Studs and nuts hold the head to the cylinder and permit internal inspection of the same at any time without destroying the gasket. The trunk type of piston is employed and four self-adjusting eccentric spring rings are provided. These are wider than the admission and exhaust parts, cannot catch or break, and effectually secure the compression. The deflector absolutely insures perfect scavenging of the cylinder at each stroke. This latter result is also due to the relatively high compression obtained in the crank end of the cylinders, this compression only being possible in engines having an air-tight joint between the cylinder and frame. This is explained by reference to the sectional drawing which shows the small size

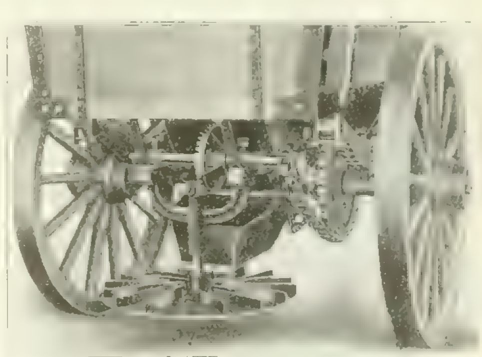
The fuel nozzle is a combination ball check valve and nozzle, is made of bronze and screwed into the center of the cylinder head. It can be quickly replaced and can be cleaned without removal from the cylinder. The value of a proper quantity of water mixed with the fuel in the combustion space has long been recognized, but the attempts to utilize it and to efficiently regulate the quantity to suit varying fuels and loads have not in general been satisfactory. Engines have been equipped with hand controlled water valves but the undesirability of devices requiring the constant attention of the operator must be apparent. Particular attention is invited to the simplicity of the water regulator. It is nothing more than a needle valve which is at all times under the control of the governor and automatically varies the admission of water to meet load requirements. By thus proportioning water supply to the quantity of fuel injected it is possible to obtain an appreciable increase in power and economy, to prevent overheating the cylinder head and burning of the lubricating oil, to eliminate shocks in the engine and to insure freedom from carbon deposits.

The frame is completely enclosed, and removable oil-tight covers for the side and crank case give ready access for inspection of parts and necessary adjustments. Main bearings are cast integral with the frame. They are of the diagonal box type, lined with babbitt metal and provided with grooves for the conveyance of oil. Oil lips are cast on the frame and caps and serve to catch and return to the interior of the frame any oil leaking through the bearings. The crank shaft is of the center crank type, made of open hearth steel forging. Adequate counterbalance weights are provided, these insuring steady operation. Connecting rods are steel forgings. The wrist pin end is of the solid type, fitted with bronze boxes, while the drank end is of the marine type, lined with babbit metal. Flywheels are of extra large diameter to facilitate starting and are of sufficient weight to insure steady operation. For the smaller sizes there is furnished a plain belt pulley and for the larger sizes, friction clutch pulleys which may be bolted to the arms of either fly-wheel.

All of the smaller engines may be readily started by hand, but the largest size and for the smaller when desired there is provided a small vertical single-acting air compressor which is driven from a pulley bolted to the flywheel. This compressor delivers air to a storage receiver, suitable for 150 lbs. working pressure, and a lever-operated air starting valve permits running the engine on air until firing of the fuel charge begins. The engine is made in various sizes by the Chicago Pneumatic Tool Co., Fisher Building, Chicago, Ill.

A Mechanical Spreader for Sanding Streets and for Use in Bituminous Road Construction.

The wagon with the spreader illustrated sprinkles slippery streets, especially those paved with asphalt and wood blocks, with a uniform layer of sand or gravel, replacing former hand methods. The revolving plate forcibly ejects the material scattering it in all directions. The scattering distance varies from 20 to 60 ft. according to the material spread. The wagon is constructed so that a lever operated from the driver's seat regulates the flow of material and the discharge of the disc. One man drives the team and operates the mechanism. The parts of the spreader and wagon are minutely standardized facilitating the obtaining of repair parts. It is claimed that the machine will sprinkle with sand or stone chips upward to 50,000 sq. yds. of surface in one hour. The machine will scatter material varying in size from fine sand to $1\frac{1}{4}$-in. stone. The spreader is manufactured and sold by the Kindling Machinery Co., Milwaukee, Wis.



The Kindling Sand Spreader.

A New Light Portable Excavator.

A truck mounted excavator designed for handling sand and gravel and for trench work,

Portable Excavator for Light Excavation and Sand and Gravel Handling.

cellar digging and light excavation generally is illustrated here. This excavator will handle a ½ cu. yd. bucket on a 20 to 22-ft. boom with a range of hoist up to 12 ft. Except the boom, which is wood, the construction is steel and steel outriggers are provided. Where conditions do not permit the use of outriggers, guy ropes can be substituted. The machine is transported by team and pole, neck-yoke and single and double trees are provided. The engine is vertical, double cylinder and geared giving a rope pull of 4,200 lbs. at a speed of 100 ft. per minute. All other parts including wire rope, blocks and fittings are the manufacturers' standard except the digging bucket which may be any make preferred by the purchaser. The machine has a digging capacity of 20 cu. yds. per hour and in actual work has shown much higher records. The cost of the machine is under $2,000. It is made by the John F. Byers Machine Co., of Ravenna, Ohio.

Steel Cars for Tunnel Construction.

[illegible] lining materials into tunnels under construction are required to have small outside dimensions, light weight, and large capacity [illegible] must be of strong construction and have [illegible] parts. The car illustrated here has been

Koppel Steel Car for Tunnel Construction.

designed to meet these requirements. One hundred of these cars were furnished the [illegible] the Orenstein-Arthur Koppel Co. of Pittsburgh, Pa., for use in tunnel work on the extensive water system which is now being installed there, and hundreds were furnished for work on the New York Aqueduct. The cars have a width of 3 ft. 4 ins.; height 3 ft. 6 ins.; capacity 27 cu. ft. The trunions are cast steel; frames are of special wide flange channel; bearings are roller bearings with a cast sleeve enclosing axles for its full length, thus keeping them free from dirt and providing oil reservoir, and wheels are special cast steel.

Sprinkler Head for Automatic Sprinkler Installations.

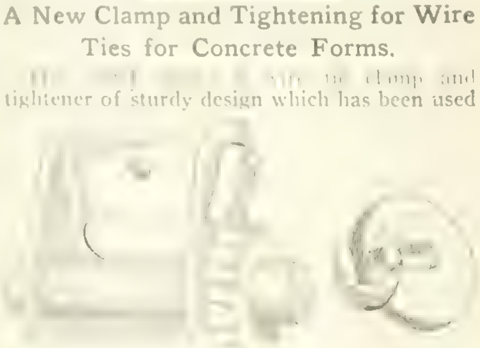
The sprinkler head illustrated here has passed rigid tests of the National Fire Prevention laboratories. This sprinkler head is

Automatic Sprinkler Head.

[illegible] on the manufacturers will, as has heretofore been the universal practice, design and install sprinkling plants complete. The manufacturers are Merchant & Evans Co., Philadelphia, Pa.

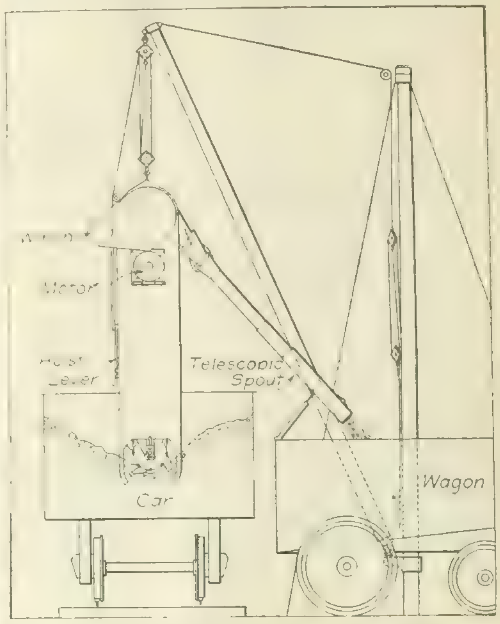
A New Clamp and Tightening for Wire Ties for Concrete Forms.

[illegible] clamp and tightener of sturdy design which has been used



Clamps and Tightener for Wire Form-Ties.

for very heavy work including forms for a concrete wall 3 ft. thick and 29½ ft. high which was poured in one operation. As will be inferred from the sketch, the disk shaped clamp is threaded over the wire and clamped tight by hammering the hook or dog down over the wire. The end of the wire which projects on the opposite side of the form is



De Mayo Portable Car Unloader.

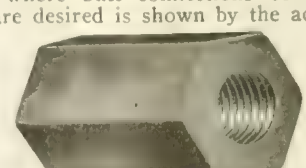
passed through the hole or holes in the drum of the tightener. A special handle or wrench, one of which is furnished with each 50 sets, is employed to twist the wire onto the drum and draw it tight. These clamps and tighteners are adapted to all kinds of forms. They provide for the use of any size wire not larger than ¼-in. These outfits are made by the Peerless Form Clamp Co., Victoria Building, St. Louis, Mo.

A Device for Loading from Cars or Stock Piles into Wagons.

The sketch illustrates a useful device in the form of a self-contained bucket elevator which is being applied to unloading coal, sand, gravel, broken stone, etc., from cars to wagon or stock pile or to loading from stock pile to wagons or cars. The bucket elevator is about 14 ft. long and is held in a steel casing which also holds an operating motor, a hoisting winch and a connection for a discharge spout. In operation the casing is suspended from a derrick, or, it may be, from any yard arm, boom or fall block convenient, and is lowered into the car or stock pile feeding down by its own weight as the material is taken out. The operation is made clear by the drawing. To operate the elevator only one man is required to swing the device about and raise or lower it so as to keep it fed with material. When not in use the elevator is raised up to the boom end and swung clear of cars or other plant in whose way it may be. All parts of the elevator are steel and it is made sturdy and durable. The general sales agents are the Hudson Machinery Co., 20 Park Row, New York City

Malleable Coupling for Reinforcing Bars.

A coupling for column bars and other places where butt connections of reinforcement are desired is shown by the accompany-



Malleable Coupling for Reinforcing Rods.

ing illustration. This coupling is made of malleable iron in a complete line of sizes by the Marion Malleable Iron Works, Marion, Ind.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design and to Methods and Cost of Construction

Volume XLII. CHICAGO, ILL., OCTOBER 21, 1914. Number 17.

An Innovation in River and Harbor Legislation.

The $20,000,000 river and harbor appropriation has been distributed by the Secretary of War and the board of army engineers. A list of the projects benefited was published in our news section last week. Except for the Mississippi, the Ohio and the Missouri and for the Delaware River far east and the Columbia River far west, none of the appropriations is large. The Mississippi receives $5,254,000, the Ohio $1,769,000, the Missouri $950,000, the Delaware $1,000,000 and the Columbia $1,760,000, or together over a half of the total appropriation. East of the Mississippi and south of the Mason and Dixon line projects are benefited to a total of nearly $2,700,000. Texas projects receive about $1,430,000 and Arkansas and Missouri projects receive about $450,000. Considerably over a fifth of the $20,000,000 goes to the South not including the Mississippi River appropriations of over $4,000,000 for improvements within southern states. A total of 171 projects receive appropriations, the lowest amount received by any one being $500.

Twenty million dollars is the smallest appropriation which Congress has made for river and harbor improvements for many years. It is common knowledge how this came about. A handful of senators and representatives compelled it by obstructive tactics. This journal believes that their action was justified. The original Senate bill called for an appropriation of $57,000,000, a portion of which sum was provided for beginning new works to complete which would require some $36,000,000 more. The bill therefore obligated the Government for an expenditure of $93,000,000. It is contended that this amount is no more than benefits to be gained justify the Federal government in expending for river improvements. If this is true it is true only with the qualification that the expenditure must be restricted to economically wise projects. Herein lay the fault of the record-breaking bill of appropriations that was killed in Congress: Its appropriations were not in many instances for economically wise improvements. Indeed, a great portion of the expenditure proposed was economic waste as truly as the havoc now being wrought in warring Europe. No one who reads the items of expenditure called for can honestly refuse to admit this fact. Unless waste in government public works is to be condoned, the members of Congress who decimated the $93,000,000 appropriation bill of 1914 were performing a public service.

The real significance of the river and harbors appropriation bill that did pass Congress does not, however, it seems to us, lie in the fact that it saved much proposed wasteful expenditure, but in the fact that it aroused a public spirit and established a precedent that may be utilized in bringing about a permanent reform. If next year Congress shall proceed in the old log rolling, pork grabbing process of inaugurating and financing river and harbor improvement works, little real good will have been accomplished by the cut of $73,000,000 that a few senators and representatives compelled in this year's appropriations. The enforced hunger of this year will be followed by gorging next year, for it must not be overlooked that this year's appropriation bill does not indicate that Congress has become abstemious—its appetite has merely been temporarily denied. Also the handing over by Congress in the present appropriation bill of the duty of selecting the works to be prosecuted is only a temporary concession to necessity. Knowledge of these facts points out clearly the way which has to be traveled to secure liberal and economic development of river and harbor improvement works.

Navigation and business interests which utilize river and harbor works and engineers and contractors who design and construct them and who want generous appropriations for their development must work to bring about a reform in present methods of Federal inauguration, financing and direction of such works. As we see the matter, there can be no alternative except to submit to radical reductions in appropriations. A few senators and representatives supported by a powerful section of the public press successfully prevented this year the appropriation of more than a nominal sum for river and harbor improvement. Next year the opposition will have increased and the same result will follow. We do not believe that ever again will there be permitted by the old pork barrel methods, the appropriation of $50,000,000 to $100,000,000 for river and harbor works. If such great expenditures are warranted, and many people believe that they are, then a new method must be sought for obtaining them.

We have said that the 1914 appropriation bill has established a precedent which points out a possibility for reform. By it Congress appropriated a lump sum of $20,000,000 to be expended on such works and in such allotments as the war department and its engineers should deem best. Congress relinquished its prerogative of naming the improvements to be prosecuted and the sums to be expended on these improvements. The first step in any reform plan is that this temporary concession shall become permanent. If there is to be systematic development of rivers and harbors for navigation, if flood control and land reclamation are to be undertaken in an expert manner, if in general the conservation and utilization of our territorial waters are to be economically developed, this development must be planned and conducted by engineers and hydraulicians and by navigation and commercial experts. There can be no escape from this conclusion.

Congress left to itself and exercising supreme authority has developed perhaps the most wasteful system of planning and building works for navigation that exists in the whole world. It is true that this condition has come about naturally, that its viciousness is recognized by many members of congress and that a few of these stand ever ready to fight against waste and extravagance, but granting that the will to reform is latent and can be awakened, Congress has neither the time nor the expert knowledge nor is it organized for the economic initiation and planning of public works. Its sole prerogative in such works should be power to grant or deny appropriations for their prosecution. Investigation into the economic necessity of and the initiation, planning and construction of public works must be placed in the hands of a special organization and this organization, we believe, should be a Federal Department of Public Works.

The Benefits to Be Derived from Attending the Coming Road Conventions.

Books devoted to road construction and maintenance are notoriously out of date, not on account of any quality lacking in the books but rather because of the recent rapid development in the science of road making. Many engineers claim they lack the time and inclination to read the technical magazines—one means of keeping in touch with the development of new ideas. The attendance at a convention once a year is perhaps the easiest method of keeping up with the times and undoubtedly is the method requiring the least mental effort for the knowledge gained.

Within the next two months three important road conventions will be held: one in Milwaukee, one in Atlanta, and the third in Chicago. The attendance of all men interested in road building and maintenance upon at least one of these conventions is urged. The benefits to be derived are numerous. Among them should be placed the opportunity afforded of meeting and exchanging experiences with other road builders. The ideas of prominent engineers and road officials expressed in papers read by them are of greater value when heard or read in the light of a personal knowledge of the men themselves. Also it is a source of inspiration to find that problems once perplexing but since solved are being encountered by others. Finally, an excellent excuse is offered for getting away from confining and routine work, seemingly impossible to leave without dire calamity befalling, but which is generally much lightened by a short absence therefrom.

The benefit worthy of special note in connection with road conventions, however, is the personal knowledge gained of newly-developed road machinery and construction plant in general and the opportunity afforded of determining the comparative merits of old and well-established makes of machinery. Economical road construction and maintenance is dependent to such a large extent upon the efficiency of tools used that the benefits derived from their proper selection assume significant proportions.

The Effect of Unbalanced Building Codes.

The lack of uniformity in the provisions of the building codes of cities in various parts of the country has resulted in an increased cost of construction for certain types of buildings. In one city the building code may specify low working stresses for reinforced concrete and comparatively high ones for steel construction, and vice versa, which naturally results in a preponderance of one type of construction almost to the exclusion of any other type. This condition limits competition to those engineers and contractors who specialize in only one kind of construction, and thus increases the cost of building construction in general. Monopoly, as applied to any kind of business, tends to increase costs to the consumer, and the construction business is no exception.

The need of more nearly balanced building codes—codes which do not unduly favor a certain type of construction—are greatly needed in many cities. The unit stresses which should be permitted are not a function of the particular part of the country in which the city happens to be located, yet a comparison of many codes might lead one to believe that such is the case. The officials of cities are not best serving the interests of these cities when they do not make determined efforts to secure balanced building codes.

To emphasize the importance of the above remarks the building status of an important eastern city will be considered briefly, the name of which is not pertinent to this discussion as similar conditions can be found in other cities. Although the following comment applies particularly to reinforced concrete construction, it may apply with equal force, in some other city, to another type of construction. In this particular city the extreme fiber stresses in concrete and steel are respectively 500 lbs. per square inch and 15,000 lbs. per square inch, the ratio of the modulus of elasticity of steel to that of concrete being 15, while the bending moment factor for intermediate spans is *WL*/10. It will be noted that these requirements are much more rigid than those of most large cities in this country. As a result, preference has been given to other types of construction, and the cost of reinforced concrete construction in this city is almost prohibitive. Capital is placed at a disadvantage, as compared with other cities, and the general progress of the city has been retarded. Even though this city requires the building public to adhere strictly to the provisions of its building code, yet the city itself, through its various departments, uses unit stresses of 650 lbs. per square inch in concrete and 16,000 lbs. per square inch in steel. It is thus seen that private capital is compelled to pay a bonus for this type of construction, although the city officials, in making public improvements, use the higher unit stresses.

Many of the provisions of building codes, notably those specifying allowable unit stresses, should be settled definitely by a commission composed of leading authorities in this country—men who will consider the subject broadly and who do not need to cater to the interests which are strongest in each city. The fact that the building codes of many cities are unsatisfactory is due mainly to the mistaken idea which has long existed that each city must solve its problems in its own way, which generally means that the strongest interests in each particular city are favored.

Lengthening the Life of Cast Iron Pipe.

In view of the fact that the experience of engineers with cast iron pipe, extending as it does over a period of 250 years, has not yet been long enough to establish the length of life of this sterling type of conduit, a discussion on means of lengthening this period of usefulness may at first thought seem superfluous. More accurately, however, we may safely say that the maximum length of life of this kind of pipe, when laid and maintained under wholly favorable conditions, has not yet been determined. On the contrary, numerous illustrations might be given in which cast iron pipe has had a comparatively short life. This condition, of course, is due to unfavorable soil or other conditions. It is a matter of the highest economic importance to know how to increase the durability of the pipe when it is laid in soils which make external corrosion likely. One need only consider the extensive use of cast iron pipe in our water works distribution systems to appreciate the importance of this subject to all water works engineers and superintendents.

The importance of the subject is such that we are publishing in this issue an extensive study of the external corrosion of cast iron pipe. The article is very practical and suggests certain precautionary and preventive measures which may be applied. While the author has necessarily discussed certain chemical phenomena, the major portion of the article relates to physical matters familiar to the water works man. The author has performed a great service in bringing together many data pertinent to his subject. The article deserves very careful study.

In the past engineers have, perhaps, given too little attention to the environment of cast iron water mains except, in late years, to take precautions to protect them from the damaging effects of stray electric currents. The article to which we refer enumerates conditions to be avoided. A similar note of warning is sounded in the latest report of the Committee on Water Pipe of the American Railway Bridge and Building Association, as follows:

> In laying underground water pipe it is necessary to take into consideration the soil conditions as affecting the life of the pipe as well as the quality of the pipe itself. It is a well known fact that clay forms the best possible covering for underground pipe, while cinders form the poorest. When laying pipe through cinders, flooring the ditch with clay for a foundation and covering the pipe until it is surrounded with the clay will form an effective protection at a very small expense. It is not always possible to find favorable soil conditions, but in many cases slight changes in the location of pipe lines will double the life of the pipe through laying it in better soil.

A careful reading of Mr. Pugh's discussion and conclusions cannot but prove helpful to those who are entrusted with the responsibility of pipe line location and construction.

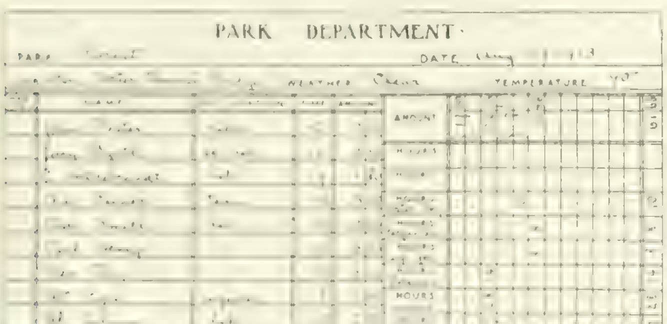
GENERAL

Unit Cost Keeping System for the St. Louis Park Department.

The construction division of the department of parks of St. Louis, Mo., has recently developed a cost keeping system which has operated with such success that an outline of

After very careful analysis of our needs we decided that we would require five general sheets upon which to place all data received concerning the different kinds of work we might endeavor to construct. These sheets would be as follows: Estimating Sheet; Foreman's Daily Report; Labor Cost Distribution;

any notes as to quality or nature of the materials to be handled. This sheet is similar to a contractor's proposal or bid. After we have made it out, and it is signed by the superintendent, it is given to the Park Commission, who accepts or rejects the bid. When the bid is accepted we commence work.



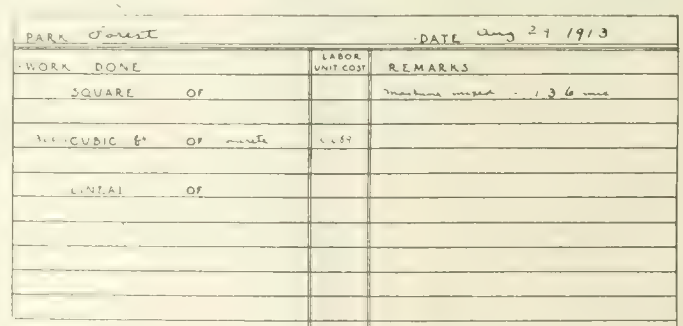


Fig. 1. Foreman's Daily Report Sheet, St. Louis Department of Parks.
(... Sheet at right.)

its main features is given in a paper by Mr. Nelson Cunliff, Superintendent of Construction to the Engineers' Club of St. Louis. We take the following portions of this paper from the August Journal of the Association of Engineering Societies:

Labor and Material Cost Distribution, and ...

The estimating sheet indicates the quantities as they are taken from the plans; the units adopted for arriving at the total estimated cost; proposals or bids of outside firms and

From the time the foreman starts work he keeps a record which he turns in on blanks known as the "Foreman's Daily Report." (Fig. 1.) This report sheet tells us 17 conditions as follows: (1) The location of the job; (2) the character of the job; (3) date of the

PARK DEPARTMENT.

Construction Classification.

Use One Letter and One Number.

A. Paths.
B. Roads.
C. Steps.
D. Lawns.
E. Sidewalks.
F. Sewers and Drains.
G. Catch Basins.
H. Curbs.
J. Lakes, Ponds and Pools.
K. Shrubbery and Trees.
L. Water System.
M. Equipment and Tools.
N. Toilets.
O. Tennis Courts.
P. Baseball Fields.
R. Buildings and Structures.
S. Bridle Paths.
T. Picnic Grounds.
U. Golf Links.
W. Concrete Seats.
X. Bubble Fountains.
Y. Bridge.
1. Supervision.
2. Lost Time.
3. Grading and Shaping.
4. Wrecking and Cutting Down.
5. Hauling Material.
6. Seeding, Sodding and Planting.
7. Form Work.
8. Plumbing and Piping.
9. Carpentering.
10. Watchman.
11. Excavating and Filling.
12. Concrete Work.
13. Reinforced Concrete.
14. Granitoid.
15. Miscellaneous.
16. Cement Finish.
17. Surveying.
18. Draughting.
19. Foundation Work.
20. Dust or Screenings.
21. Oil or Covering Materials.
30. Brick Work.
31. Painting and Glazing.
32. Stucco Work.
33. Hardware.
34. Lathing and Plastering.
35. Tile Partitions.
36. Ornamental Iron Work.
37. Sheet Metal.
38. Marble and Slate.
39. Heating and Ventilating.
40. Inspection.
41. Cinders.
42. Drainage.
43. Rip Rap or Telford.
44. Rolling.
45. Macadam.
46. Cleaning Up.

Fig. 2. Street Showing Construction Classifications, St. Louis Department of Parks.

report; (4) kind of weather; (5) name of the foreman; (6) kind of work done and later classification; (7) total hours of labor under each class; (8) rates per hour; (9) total pay; (10) number of units of each class of work done; (11) units of materials and supplies used; (12) units of materials received; (13) units of materials in stock; (14) delays, time and cause; (15) time machines are actually working; (16) kind of machine or tool used and its condition; (17) remarks.

The foreman does not have to use all of these blanks but uses those necessary to meet the exigencies of the particular work he is on. The clerk in the office reserves the privilege of making the classification by letters or symbols (Fig. 2) to best meet the needs adopted in the estimating sheet. The object of "Amount of work done," "The materials Used," etc., is to enable the superintendent to figure on the job, a quick daily unit cost for the preceding day. For example, I went onto a piece of work the other day in which one of our engineer corps was in charge: He reported that he had unloaded a car of rock and that the cost had been $10. I immediately told him that that was too much to pay for unloading a car, and that his men had been soldiering on him; he admitted that such might be the case and if so he would remedy it on the next car. When this car came in he was able to report that he had unloaded it for $5.

It is impossible in our system to subdivide our general units so that we might have a daily unit cost for form work, handling cinders, placing steel, etc., all of these items being part of one of our estimating units, but the idea is that the superintendent who is supposed, when he sees the daily report, to know the gang's capacity for a day's work, will be able to call the foreman's attention to the fact if he is not delivering the amount of work per capita that he should.

The foreman's daily report is taken to the office and the clerk who, after checking the numbers of hours on the time sheet and the

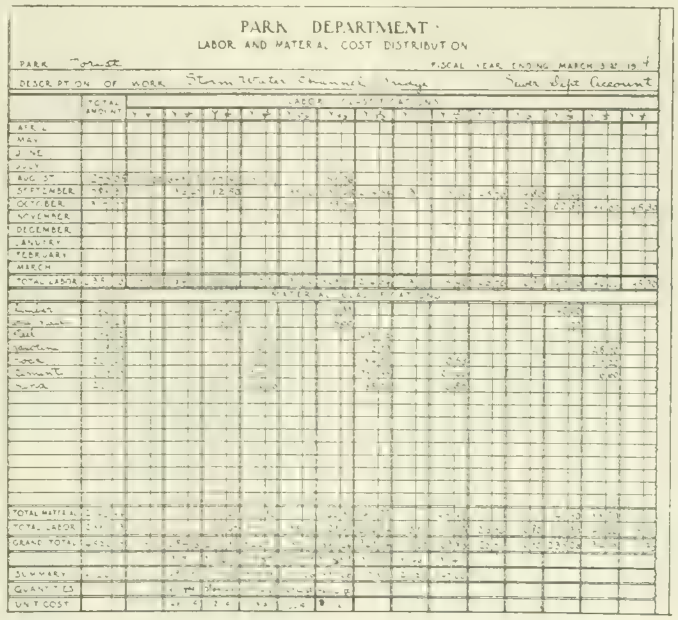


Fig. 4. Labor and Material Costs Distribution Sheet, St. Louis Department of Parks.

·PARK· DEPARTMENT·

·LABOR· COST DISTRIBUTION· (DAILY)

PARK· Forest ·MONTH·OF· August 1913.

·DESCRIPTION·OF·WORK· Storm Water Channel Bridge

DATE	TOTAL AMOUNT	Y 1/F	Y 11/F	Y 7/F	Y 12/F	Y 7/FB	Y10	Y4
1								
2								
3								
4	31 50	3 -						
5	40 50	3 -						
6	43 50	3 -						
7	43 50	3 -						
8	43 70	3 -	40 70					
9	47 30	3 -	44 50					
10								
11	49 -	3 -	46 -					
12	45 08	3 -	[illegible]2 08					
13	[illegible]3 40	3 -	10 40					
14	50 85	3 -	45 60					
15	56 60	3 -	43 50					
16	51 25	3 -	46 -					
17	1 25							
18	57 65	3 -	47 50					
19	39 23	3 -	23 82					
20	62 60	3 -	43 80					
21	88 42	3 -	51 .5					
22	95 55	3 -	40 -					
23	86 55	3 -	48 50					
24	1 25							
25	87 90	3 -	33 40					
26	45 81		7 00					
27	30 86		6 -					
28	60 11		9 -					
29	61 66		7 -					
30	27 27		6 -					
31	4 50							
	1275 05	57 -	648 55					

Fig. 3. Labor Cost Distribution Sheet, St. Louis Department of Parks.

distribution of labor, enters the labor distribution on the "Labor Cost Sheet" (Fig. 3) by means of symbols that will get this distribution under the proper units and check back to the estimating sheet. The symbols referred to represent a certain unit of work on a certain job, for instance, finishing on the steps or digging trenches on a sewer system. We have tried to extend our units out to the last item of work done but have found it impracticable on account of the number of symbols required, for instance, we would like to know the cost of form work on the foundation of No. 1 steps on a job on which there are four or five sets of steps. This would mean that it would be necessary for us to use four symbols which would complicate our system and make our office work cost more than it would be worth; so we simply report form work under the unit upon which the form work was done. However, if the unit cost is high or out of proportion, we can always go into unit detail by referring to the foreman's daily report.

Next we have the monthly labor and material cost distribution sheet. (Fig. 4.) Our payrolls and material bills are paid once a month, therefore we find it convenient to balance our accounts from month to month. This sheet is so arranged that we may start work on a park and finish some unit on which we have estimated, and then return several months later and complete the whole. This enables us to have a gang specialize in a particular line of work throughout the season, going from park to park doing their special kind of work, and does not in any way interfere with us in closing our books at the completion of any one unit. On this labor and material sheet material may be bought on one requisition and divided among many units of construction. In this way we can obtain the low price given for large quantities. At the bottom of the sheet you will find the quantities and unit cost of any particular subdivision. I would draw attention to the fact that there are no overhead charges or interest and depreciation charged off the sheet as this is taken care of in our office account and tool and equipment account. The quantities shown are built up on the foreman's report and actual measurements taken on the job after the work has been completed. These quantities are also shown on the estimating and recapitulation sheets for detailed study.

Up to now we have dealt with the book and cost-keeping system as is touched by the engineer. In our work money is appropriated by the General Assembly for certain construction items and under ordinance we must report to the Auditor and Comptroller what this money has been expended for. This necessitates our appropriation register sheet. This sheet is possibly a little more complicated than the systems in vogue in contracting offices, but it [illegible] as stated in [illegible] appropriated for [illegible]

In appearance this system may look a little voluminous, but it is kept by one clerk who devotes his entire time to this work and the [illegible] there are about 200 men on the payroll which [illegible] These 200 men have been scattered over as many as 22 different jobs at one time. The one thing that the system lacks is a definite [illegible] time during the progress of the work except as [illegible] However, at the completion of a piece of work it does furnish complete data as to the cost of any given unit and from this greater accuracy may be had in preparing estimates for [illegible] conditions.

Probably the most severe criticism that the [illegible] that they estimate their work too low causing the contractors' bids to be rejected because insufficient funds have been appropriated to meet a proper cost of the work. I feel that if more attention were given to the obtaining and keeping of accurate cost data such as I have described to you tonight, there would be fewer bids rejected on account of being too high. As engineers and contractors we should do what we can to distribute true records of cost on various work that we have been connected with. This theory, I know, is not one adopted by our leading contractors. They are generally reluctant to give out any information regarding unit cost of any work with which they are connected. This I feel is a matter of pride, and must be overcome before the engineer and the contractor will have the confidence they should have in one another.

Costs of Collecting, Hauling, Transferring and Transporting Municipal Refuse.

A very important element in the collection, haul, transfer and transportation of refuse materials is the cost. Many local factors enter into the cost element, and unless these are considered and understood, the cost data are misleading. Standard forms for recording cost data of refuse collection are not used extensively, so that the data presented should not be taken without qualification. In all cases, reference should be made, if possible, to the original source of the information. Methods of analyzing the cost of various parts of the service were given by Mr. Samuel A. Greeley in his paper before the recent annual convention of the American Society of Municipal Improvements. Mr. Greeley's paper, which is here given in full, also contains some actual cost data.

TABLE I.—COSTS OF COLLECTION (LOADING AND TEAM HAUL) OF MUNICIPAL REFUSE.

City.	Year.	Population served.	Total annual cost.	Total annual tonnage.	Total annual yardage.	Unit cost. Per ton.	Unit cost. Per yard.	Cost per capita.
Garbage:								
Boston	1912	480,000	$131,648.40	54,215	...	$2.43	...	$0.27
Chicago	1912	2,300,000	381,174.00	119,159	...	3.20	.	0.17
Cleveland ...	1912	596,400	127,800.24	43,555	...	2.93	.	0.21
Columbus ...	1912	192,700	34,779.23	18,789	.....	1.85	.....	0.18
Dayton	1908	110,300	21,000.00	9,941	13,479	2.11	$1.56	0.19
Detroit	1910	465,766	66,865.67	34,065	....	1.96	.....	0.15
Evanston ...	1910	24,978	3,186.00	2,800	4,670	1.14	0.68	0.13
New York	1911	4,400,000	275,380.00	336,984	....	0.82	.....	0.06
Winnipeg ...	1911	151,918	19,371.90	15,510	..	1.25	....	0.13
Ashes:								
Boston	1912	480,000	346,396.01	247,203	...	1.40	..	0.73
Evanston ...	1910	24,978	2,174.00	.	13,461	.....	0.16	0.09
Winnipeg ...	1911	151,958	5,793.73	5,227	.	1.11	..	0.04
Rubbish:								
Columbus ..	1912	192,700	47,887.12	...	75,096	.....	0.64	0.25
Evanston ...	1910	24,978	5,108.00		29,479	..	0.17	0.21
Ashes and rubbish:								
Cleveland ...	1909	543,000	119,982.00	84,547	202,752	1.42	0.59	0.22
Cincinnati ..	1909	360,000	116,631.68	87,611	222,634	1.33	0.52	0.32

References: These data are all from annual reports.

ELEMENTS OF COST.

The elements of the cost of each part of the collection service can be segregated and studied advantageously by the following method. The unit quantities used in the computations were assumed for certain local conditions and will not necessarily apply everywhere. They are presented here to illustrate the method of analysis.

Loading.—The cost of loading will vary with the character of the material, the district served, the season of the year, the unit cost for labor in each locality and other local conditions. The method of analysis for loading garbage follows:

a. Number of people per house or per [illegible] 10
b. [illegible] to make one collection or to give service to [illegible] 1
c. Interval [illegible] between collections 2
d. Capacity of garbage wagons in tons.. 2
e. [illegible] in tons [illegible] per day 0.273
f. [illegible] after two days interval between collections, tons [illegible] 0.546
[illegible] people requires 100 collections, taking 100 minutes' time.
[illegible] to load a [illegible] above [illegible]
[illegible] 6.1 hours. If the cost of the team, wagon and collector be taken at 75 [illegible] loading [illegible] the cost per ton for loading garbage ($2.28) 2.28

The analysis can be applied to the loading of [illegible] any refuse material, if the proper unit quantities and basic data be first determined. The cost per ton for loading other refuse materials in accordance with assumed data will be as follows:

Materials.	Cost of loading per ton.
Ashes	$0.417
Rubbish	2.62
Mixed refuse	0.56

Motor Trucks.—The cost of loading a motor truck can be studied in a similar way. The cost of operation will be greater per hour and the rate of loading will have to be increased proportionately to make the cost comparable with loading a team drawn wagon. The cost of haul by motor truck will be less.

The use of motor trucks in refuse collection service will increase. A relatively high loading cost can be reduced by limiting the motor truck to transportation after the loading of the wagons by the so-called traction and trailer system now being tried on a large scale in New York City and used in quite a number of European cities.

Hauling.—The refuse material loaded in the collection wagon must be hauled to the transfer station or place for final disposal. This will be done by horse-drawn vehicle or by motor. The length of haul will be from the point of last collection to the place of final delivery. This distance or haul must be covered twice for each complete load.

The cost of haul will depend on the rate of travel, the weight of the load and the cost of the team and the driver, or motor and mechanic. The cost of team haul may be analyzed as follows:

Assumed:	Per hour
Rate of travel, miles....................	3.0
Cost of outfit....................	$0.75
Cost per mile of travel....................	0.25
Cost per mile of haul....................	0.50
Cost per ton-mile haul with a 2-ton load	0.25

The cost of haul by gasoline or motor truck may be analyzed as follows:

Assumed:	Per hour.
Rate of travel, miles....................	6.0
Cost of outfit....................	$2.40
Cost per mile of travel....................	0.40
Cost per mile of haul....................	0.80
Cost per ton-mile haul with a 5-ton load	0.16

The rate of travel will vary considerably from different sections of a large city, being slower through streets congested with a large volume of traffic. In such districts, collection work should be done at night or during the early morning hours.

Transfer Stations.—The operation of transfer stations should also be considered as a part of the cost of transportation. A transfer station to handle 600 cu. yds. a day, or 375 tons, may cost, depending upon type of building and local conditions, about $50,000, including land in a fairly well-built up section.

The annual cost of operation may be estimated as follows:

Interest at 5 per cent....................	$ 2,500
Depreciation of plant....................	1,250
Labor:	
1 foreman	1,200
4 laborers	3,600
Repairs and supplies....................	2,500
Total	$10,800

This is equivalent to a cost of 9.4 cts. per ton.

COST OF TRANSPORTATION.

The cost of transportation of refuse from the transfer station to the place of final disposal depends upon the method used. The cost for several methods is discussed below.

Trolley Transportation.—Assume a typical transfer station receiving 600 cu. yds. of refuse material per day. Assume trains to be made up of one motor car which carries no load and two trailers. Assume each trailer to have a capacity of 25 cu. yds. To move 600 cu. yds. 24 trailer loads are required. If the place of disposal be so located that each train can make two trips a day, six trains will be required. Assume that three motors can handle the six trains. The daily cost of operation will then be:

	Per day.
Motor cost, three at $25	$ 75.00
Trailers, twelve at $6	72.00
Total	$147.00

If the 600 cu. yds. of refuse weigh 375 tons, the cost of trolley transportation will be 40 cts. per ton.

Barge Transportation.—A good, serviceable tug will cost about $30,000 and deck scows about $7,000 apiece. The annual cost of operating a fleet may be as follows:

Annual cost of tug:		
Interest at 5 per cent		$ 1,500
Depreciation on 15-year life		1,389
Labor:		
Captain	$2,100	
Engineer	1,800	
Fireman	1,000	
Deck hands	1,800	6,700
Repairs		2,500
Fuel		3,500
Supplies		1,000
Insurance		200
Total		$16,789
Annual cost of barge:		
Interest at 5 per cent	$ 350	
Depreciation	324	
Deck hands	1,800	
	$2,474	
Assume that 1 tug serves 4 barges		9,896
Total annual cost of fleet		$26,685

If each barge makes one trip per day, carrying 100 tons of refuse, the cost per ton amounts to 22 cts.

In like manner the elements of cost can be determined for other methods of transportation.

Steam Railroad Transportation.—The cost of transportation by steam railroads depends principally upon the switching charges. These will range from $5 to $15 per car. A car will hold about 40 tons of garbage, so that the switching charge will average about 20 cts. per ton.

Available Collection Costs.—Actual cost data should be studied to check the costs estimated above, but these are not available for a large number of cities. The costs for collection service are generally recorded to include both loading and hauling in one figure, while costs of transportation are frequently given separately. The cost data for some cities in which the itemized cost of collection is available have been summarized in Table I (p. 376).

Chicago Data.—Jacobs and Senfield have made a careful analysis of the cost of collecting garbage, and ashes, and rubbish in Chicago. These data are compiled in excellent detail and accuracy. The average cost for the five years—1908 to 1912—are given in Table II.

TABLE II. — AVERAGE COST OF REFUSE COLLECTION, LOADING AND HAULING, AT CHICAGO, ILL.

Year.	Cost per ton of garbage.	Cost per cu. yd., ashes and rubbish.
1908	$3.78	$0.56
1909	3.76	0.57
1910	3.43	0.59
1911	3.19	0.62
1912	3.20	0.60

If ashes and rubbish together weigh 1,000 lbs. per cubic yard, the cost of collection per ton amounts to $1.20.

IMPROVED METHODS FOR RECORDING COST DATA.

The value of unit cost data for loading, hauling, transferring and transporting refuse materials should be realized by city officials. Accurate records should be kept and published in similar forms in different cities, so that comparisons can be made and a check secured on the efficiency of the local work. The total costs should be divided and recorded in accordance with the details discussed to show the cost of each element of the work.

The Engineer's Reading and Study.

To the Editors: I have read with keen appreciation the article in your Sept. 23 issue on "The Engineer's Reading and Study," and I thought you would be interested in knowing that our company has a small but carefully selected library which is used for the development of its employes.

The usefulness of this library has recently been extended to the Southern states, in which our company operates, through an article contained in our company paper. As a result we now have about 50 men in the field who are taking up definite courses of reading which we have prescribed for them.

In each case we suggested to the reader that the public library be visited and that such books as can be had locally be secured from the library, and those that cannot will be sent from our library in Atlanta.

It would be advantageous for companies engaged in engineering work to exchange information on this kind of work, and maybe your paper could be the means of disseminating such information as would bring together and effect exchange of information on methods of promoting reading and study among engineers and industrial workers.

A catalog of the library is here given and some of the sample courses are indicated.

Very truly yours,

KENDALL WEISIGER, Efficiency Engineer.
Southern Group of Bell Telephone Companies,
Atlanta, Ga., Oct. 8, 1914.

(We publish herewith the titles of the books in the circulating library of the Chief Engineer of the Southern Group of Bell Telephone Companies, as mentioned in our correspondent's letter. Three courses of reading are indicated on the list of books as follows: Books marked (1) form the course of reading for managers; those marked (2) form the course for plant chiefs; and those marked (3) form the course for plant foremen and wire chiefs. The list of titles follows—Editors.)

Telephony and Electricity:
- Lessons in Practical Electricity—Swoope (2) (3).
- Telephone Principles and Practice—Wilder (1) (3).
- Telephony—McMeen & Miller (2) (3).
- American Telephone Practice—K. B. Miller.
- History of the Telephone—Casson (1) (2).

Organization and Management:
- Principles of Scientific Management—F. W. Taylor (2).
- Efficiency—Emerson (1) (3).
- Twelve Principles of Efficiency—Emerson (2).
- Increasing Human Efficiency in Business—Scott (1) (2).
- Choosing a Vocation—Parsons.
- American Office Management—Schultz.

Health and Hygiene:
- The Efficient Life—Dr. L. H. Gulick (1) (2) (3).
- Reproduction and Sexual Hygiene—Hall.
- Handbook of Health—Dr. Woods Hutchinson (1) (2) (3).

Letter Writing and English:
- The Correct Word and How to Use It—J. T. Baker (2) (3).
- Manual for Writers—Manly & Powell.
- How to Do Business by Letter—Cody.
- Handbook of English for Engineers—Sypherd.
- Modern Business Correspondence—Erskine (1) (3).
- How to Write Letters That Win—System Company (2).

Development of the Mind:
- Memory and the Executive Mind—Robinson (1) (2).
- How We Think—John Dewey.
- Mind and Work—Dr. L. H. Gulick (1) (2).
- Intellectual Life—P. S. Hamerton (3).
- Vocation and Learning—Hugo Munsterberg (1) (3).

Inspirational:
- Addresses to Engineering Students—Waddell & Harrington.
- Self Measurement—Wm. DeWitt Hyde (1) (3).
- Life of Benjamin Franklin—Franklin (1) (2) (3).
- Life of Abraham Lincoln—Carl Schurz (1) (2) (3).
- How to Live on 24 Hours a Day—Arnold Bennett (1) (2).
- The Investment of Influence—Hillis.
- Letters From a Self-Made Merchant to His Son—Lorimer.
- Making Good—J. T. Faris (3).
- The Exceptional Employe—O. S. Marden (1) (2) (3).
- Making the Most of Ourselves—C. D. Wilson (2).
- How to Get Your Pay Raised—N. C. Fowler.
- As a Man Thinketh—James Allen (1) (2) (3).
- The Magic Story—F. V. Dey.
- A Message to Garcia and Other Preachments—Hubbard (3).

WATER WORKS

The External Corrosion of Cast Iron Pipe—Precautionary and Preventive Measures.

Cast iron pipe is in such general use that a knowledge of its strong points, its limitations and the precautions necessary to insure its durability under certain conditions is of the highest practical importance to the water works engineer and superintendent. The purpose of the present article is to warn the engineer of conditions where special protection is needed against external corrosion and to suggest preventive measures adapted to different cases. Tuberculation and electrolytic corrosion have been exhaustively studied elsewhere and are not here considered. The information here given is taken from a paper by Marshall R. Pugh, entitled "External Corrosion of Cast Iron Pipe," presented before the American Society of Civil Engineers on Oct. 7, 1914, and published in Vol. XL of the Proceedings, pp. 1641-1691. In the full paper numerous examples of the great durability of cast iron pipe are first given, after which other instances are cited in which it has deteriorated rapidly. Mention of the composition of cast iron and the theories of its corrosion is followed by a study of conditions contributing to and inhibiting corrosion. Reasons are sought for the deterioration observed in typical cases, and precautionary and preventive measures are considered. This abstract is limited largely to the last mentioned portions of the original paper. Only external corrosion is considered, though many facts must be applicable as well to internal corrosion.

The art of casting iron was unknown until the 14th or 15th century. The earliest records which appear to be authentic relate to several old pipes of various diameters laid, by order of Louis XIV, near Paris, to supply the town and parks of Versailles. These were laid between 1664 and 1688, or from 226 to 250 years ago. They aggregate a length of about 26,000 ft. and are still in use. Other cast iron pipe still in use in Versailles, aggregating 27,500 ft., was laid in 1685, or 229 years ago. These pipes were laid in lengths one meter long and the sections were joined by means of bolted flanges. The few repairs which have been required have generally been necessitated by the bad condition of the flange bolts which have rusted out. Instances of the great durability of cast iron pipe as observed in European cities could be multiplied to great length.

In the United States the earliest pipes were of bored logs. Philadelphia adopted them, although a length of ¾ mile of iron pipe was laid in 1804 as an experiment. The new Fairmount water works in Philadelphia had a distribution system of cast iron pipes. This was the first of its kind in America, at all events the first on an extensive scale. These pipes were imported from Europe and were of the bell and spigot type in 9-ft. lengths. These pipes are of particular interest owing to the fact that they are still in use after 95 years, and give every evidence of continuing their usefulness. A century's use has not rendered these pipes unfit for service. The late James B. Francis cited an experience of Lowell, Mass., as follows:

It is within my experience and knowledge that pipes at Lowell, which were obtained from Philadelphia in 1830, were then intended to be used under a pressure of 150 ft. head, and are now in use under 200 ft. head, without any signs of failure after 60 years' use.

Even when immersed in the fresh water of streams and rivers, this resistance to corrosion is manifested. Cast iron cannon taken from a vessel sunk in the Delaware River for more than 40 years were perfectly free from rust. Mr. Pugh sums up his observations on the durability of cast iron pipe under favorable conditions by saying that our experience of 250 years with this type of pipe has not been sufficiently long to establish just what its life is.

EXAMPLES OF DETERIORATION OF CAST IRON PIPE.

A knowledge of this fact, however, may lead to mistaken conclusions. One of the earliest observations relating to the rapid deterioration of cast iron pipe was that of Mr. Thomas Duncan, who in 1853, in describing the Liverpool Corporation Water Works, said:

During the process of taking up and adjusting the pipes in the lower districts, near the margin of the docks, where the soil is impregnated with common salt, they were found, in many instances, to have become so soft, that they could easily be cut by a knife. This was found to be the case where they had not been laid for more than 20 years; whereas, in the higher districts, where they had been laid for nearly 50 years, they were found to be as good as when first laid down. In all instances the hardest pipes had deteriorated least, and were found cleanest on the inside.

In other words, under certain conditions, cast iron pipe is not durable, unless protected. Therefore it is of great importance to the engineer to determine just what those conditions are and how to guard against them.

The salt of the ocean is particularly destructive to cast iron, and its action is manifested in a peculiar way, differing materially from ordinary rusting, scaling and pitting. Cases occur, and with considerable frequency, where trouble has resulted from laying cast iron mains in saline soils.

In 1902, according to information furnished by Mr. D. H. Townley, superintendent of the Elizabethtown, N. J., Water Co., that company laid about two miles of 6-in. cast iron main across salt meadows. It was ordinary coated water pipe, about 0.45 in. thick, and was laid about 8 ins. below the surface of a salt marsh. The marsh surface was about 18 ins. above ordinary high water, and therefore was covered at times by salt water. The nearest trolley is about three miles away, so that stray currents have nothing to do with the observed deterioration of pipe. Seven years after the pipe was laid it began breaking under a pressure of 40 lbs., and it became necessary to renew the whole line in 1912, after a life of only ten years.

Internally the pipe was in good condition. The external appearance was equally good, there being an entire absence of pitting. On closer examination this appearance was found to be deceptive, for in many places, sometimes the top, sometimes the bottom, and at other times the sides, were found to have large spots where the iron appeared to be leached out, so that it could be cut like putty, or rather like soft graphite, which it strongly resembled in appearance. This action was not regular, sometimes skipping several feet, but was worst at the bottom of the pipe. In some instances these soft places had been eaten completely through the walls of the pipe, which, although not pitted or swelled out, had entirely lost all the characteristics of iron. After exposure to the air this metamorphosed iron hardened.

At Perth Amboy, N. J., similar trouble was experienced. A 16-in. main, crossing a salt meadow for some 3,500 ft., had to be abandoned altogether in less than 20 years. Mr. A. H. Crowell, superintendent of waterworks, under date of March 20, 1913, says:

Our second main, a 24-in., laid under the same conditions (as the 16-in. main) about seven years ago, now shows evidence of the same deterioration, softening in spots so that it may be pared with a knife on the exterior. This line the Board has decided to uncover and leave exposed in open trench to the flow of the tides, our experience having shown that pipe when so laid exposed to the light and air is not affected; in other words, we have reached the conclusion that this particular deterioration occurs from chemical action due to contact of roots of salt grasses.

In connection with the deterioration, one peculiarity noted is that when such pipe is removed and exposed to the light and air for a period of several weeks it will harden and ring apparently sound.

This pipe, which was laid in black meadow muck from which salt grass is cut, is buried with 3 ft. of cover. The meadow is completely submerged at extreme tides, with an ordinary range of about 4 ft. The pipe was ordinary gray cast iron, tar-coated, weighing 140 lbs. per foot, and the deterioration occurred at all points, but principally on the top and upper sides. As in the Elizabeth pipe, the exterior did not pit, but deterioration occurred from softening in spots which finally blew out.

At Atlantic City, N. J., according to information furnished by L. Van Gilder, engineer and superintendent of water works, the water supply is delivered to the city from the Absecon Pumping Station, the mains leading through salt marsh for a distance of 22,000 ft. In 1882 a 12-in. cast iron main was laid, which in 1888 was paralleled by a 20-in. cast iron main; and in 1901 the capacity was further augmented by laying a 30-in. steel main alongside the others. In 1911 a 48-in. wood stave main was laid parallel to the preceding three for about 7,000 ft., after which it takes a different route for the remainder of the line. Both the cast iron mains are soft in spots and blow out from time to time in places under a normal pressure of 50 lbs. The deterioration is precisely like that noted at Elizabeth and at Perth Amboy. The steel pipe is even less satisfactory. Though laid at a later date than the two cast iron mains, it is rapidly deteriorating, and 6,000 ft. of it are now out of service.

The top of each of the two cast iron mains is about 1 ft. below the surface of the meadow; the steel main is about half its diameter below the surface, and the top is covered with meadow sod.

At Richmond, Va., according to Mr. E. E. Davis, superintendent of water works, about 400 ft. of cast iron pipe, laid in a salt blue marl, in 20 years became soft enough to cut with a knife, the action appearing to be equally great at its top, bottom and sides. A trolley track 450 ft. distant may possibly have contributed to the disintegration.

Trouble has been experienced also from saline soils at points remote from the ocean. At Syracuse, N. Y., there are salt springs which have been used in the manufacture of salt since their discovery by the Jesuits in 1654. They are situated along the shores of Onondaga Lake, and in 1797 were taken over by the state, which passed laws governing the manufacture of salt.

The Bureau of Water of Syracuse, according to Mr. George A. Glynn, superintendent, has had much trouble from the corrosion and disintegration of water pipe in the salt lands. A line of pipe runs in a very low section of these lands, the major part of which is under water a large portion of the year. The part which is continually submerged, however, has not disintegrated, but that adjacent to it, where the main was laid about 1904, began to fail after several years' service. The corrosive action appears to have been similar to that noted heretofore. The pipe was sound in parts, and at the leaks it could be cut with a knife. The salt has been for a century at the point where the disintegration occurs.

A third soil destructive to cast iron pipes is also noted by Mr. Glynn as occurring at Syracuse. He says in a letter:

Since submitting this matter to you and Dr. Pattee, I have observed two cases of disintegration in different parts of the town, the pipe being eaten precisely as that in Spring St. In each case the soil above the pipe was composed in the main of coal ashes and apparently the constant trickle of water had created some chemical action which ate the pipe. As a layman, I have concluded that coal ashes, where there is a possibility of a flow or trickle of water through them, are responsible for this particular kind of disintegration.

A. A. Reimer, superintendent of water works at East Orange, N. J., notes an instance in his experience where an 8-in. main laid in clay and ashes—marl with some cinders mixed with it—went out of service every six months.

In Germany, R. Krzizan cites a case where a portion of an asphaltum-coated cast iron water main which had been in service for 20 years suddenly became defective. A number of conical holes were scattered irregularly over the surface and were surrounded by graphite-like material containing particles of metallic iron. There was nothing in the water running through the pipe to account for this formation, but crystals of gypsum were distributed irregularly in the clay in which the pipe was laid. Krzizan attributes the corrosion to local currents set up by contact between the graphite particles and the iron, in the presence of a solution of calcium sulphate at the points where the crystals of gypsum were contained in the clay adjacent to the pipe.

A question arises as to what effect the alkali soils of the West would have on cast iron pipe, and it is hoped that information on this point may be brought out. At Montrose, Colo., for example, there are areas of such high alkalinity that continued experience there will be useful in giving an idea of what may be expected in such cases. The source of most of the alkali in that region is the underlying shales, and whenever they are near the surface its presence is assured. The low valley soils are also in general heavily charged with alkali. Mr. R. L. Smith, superintendent of water works at Montrose, states that the system, which was built in 1888, consisted of 4, 6 and 8-in. wrought iron pipe, which deteriorated so rapidly that it was all replaced either by cast iron or wooden stave pipe except about 800 ft. of the 8-in. pipe in the southwestern part, which is still in very fair condition. Cast iron pipe which has been in use for nearly 20 years has appeared to be in an excellent state of preservation, wherever it has been examined, and has never given any trouble. The alkali soil, however, has been disastrous to the wire of the spiral-wound wood stave pipe. It was laid in 1905 and will have to be removed within the next couple of years. There are no trolleys, and the pipe is laid 4 ft. below the surface.

Before attempting to draw any conclusions from the facts presented, Mr. Pugh first considered the composition and character of cast iron, and then reviewed the theory of its corrosion and studied some experimental data bearing on the subject. His theoretical discussions are here summarized in the briefest terms.

THE COMPOSITION OF CAST IRON.

Cast iron is a substance of varied composition, both physically and chemically. Pig iron may be termed a frozen solution of carbon and iron. If no other impurities were present, it would contain 4.3 per cent of carbon. The free graphite found in cast iron is due to the same action as the formation of the solid plates of salt which separate out of the salt water when it freezes, with the important difference that whereas the ice does not retain any salt in solid solution, the solidified iron retains much carbon.

It has been known for a long time that in pig iron which has been heated considerably above the fusing point the carbon tends to separate in the graphitic form, whereas the same metal, heated to a lower degree, on cooling would retain more carbon in the combined condition. Much study has been devoted to this obscure subject, and a series of modifications of iron and iron-carbon, all due to different conditions of heating and cooling, has been investigated, but it would be out of place to consider the subject in this paper.

To summarize briefly, the metal collecting in the bottom of a blast furnace is a saturated solution of carbon in iron, and as it cools to the solidifying point carbide is thrown out of solution. This splits up into iron and carbon. Iron solidifies, the mother liquid splits up, and the same process is repeated. It is analogous to the behavior of a freezing salt solution.

The carbon in a casting, amounting ordinarily to between 3 and 4 per cent, may all be in the combined state, forming the intensely hard carbide of iron, Fe_3C, with a characteristic white fracture, or it may be in the form of free graphite, the numerous plates of which cut up and weaken the matrix of ferrite, the whole having the gray fracture found in gray iron.

Each added impurity (and many are invariably present) increases the complexity of the casting, both chemically and physically, and sets up innumerable nodes for electrolytic differences of potential.

CORROSION OF CAST IRON: EXPLANATORY THEORIES.

The rusting of iron is by no means the simple process it was once thought to be. How heterogeneous a casting may be, both in point of composition and in physical structure, has been shown. This adds materially to the difficulty and explains the often apparently discordant results obtained by careful investigators.

Five separate theories have been advanced to account for the rusting of iron, but at present three of these have been practically eliminated, leaving only two over which the fight still wages—the acid theory and the electrolytic theory. The one undisputed fact is that iron cannot rust in air unless water is present, nor can it rust in pure water unless oxygen is present.

The Acid Theory.—This theory regards corrosion as resulting from an acid, usually carbonic, which unites with the iron to form ferrous carbonate ($FeCO_3$) or the soluble ferrous hydrogen carbonate ($FeH_2[CO_3]_2$), and the liberated hydrogen combines with any dissolved oxygen in the water to yield water. The oxygen of the air next converts the soluble iron salt into hydrated oxide, or rust, liberating the carbon dioxide (CO_2), which is now free to attack a fresh portion of iron. With a supply of water and oxygen, a small portion of carbon dioxide could thus rust an indefinite quantity of iron.

The Electrolytic Theory.—This theory holds that the presence of an acid is not necessary to cause rusting, but regards the action as electrochemical.

Whatever may be the abstract facts concerning the electrolytic and acid theories, if absolutely pure iron, pure oxygen and pure water are used, both reactions take place under actual conditions of corrosion.

Natural waters are usually charged with dissolved oxygen and carbon dioxide from the air, thus readily permitting the action of carbonic acid on the iron. Even the purest commercial wrought irons are not homogeneous, either physically or chemically, so that electrolytic action also comes into play.

CONDITIONS CONTRIBUTING TO CORROSION.

Hale, in studying experimentally the corrosion of iron pipes by water flowing through them, concluded that practically carbonic acid, coupled with dissolved oxygen, is a most potent factor in corrosion.

Hale also found that the rate of corrosion increased proportionately to the increase of free carbonic acid. Analyses for soluble iron and dissolved oxygen proved that this increase was due to the fact that the carbonic acid dissolved the iron more rapidly, and, more iron being in solution, oxidation is hastened.

The aqueous solution of most acids dissolve iron. Cast iron subjected to the slow action of dilute acids, as, for example, acid water from the mines, retains unchanged the external shape of the casting, but the composition of the metal is altered, iron being removed in solution.

Very dilute solutions of alkaline hydroxides, such as sodium hydroxide, or caustic soda, and calcium hydroxide, or slaked lime, rapidly absorb carbon dioxide, forming carbonates which in such dilute solution do not prevent the iron from rusting. On the contrary, by hastening the absorption of oxygen by the iron, they increase corrosion.

Rain-water, particularly in thunderstorms, contains nitric acid or nitrates, and polluted waters also may contain nitrates and nitrites in solution. Magnesium and ammonium chlorides are found to corrode iron, even in the absence of air. This gives a clue to the particularly destructive effect of sea water (which contains magnesium chloride) on cast iron, even at great depths below the surface, where fresh water would not contain enough dissolved oxygen to exert any deleterious influence on the iron.

The rate of motion of water also has a bearing on the rapidity of corrosion. In certain circumstances it causes greater aeration and it also brings on a constantly renewed supply of air to the rusting metal. Mr. J. B. Jamieson gives an interesting example of this, in which a city water main, running from a large feeder main to a dead end, had a number of side branches or lateral mains connected to it. There was, therefore, a continually diminishing quantity of water and a diminishing rate of flow the greater the distance from the large feeder main. After 45 years of use the pipe was found almost choked with rust at the point where it left the large feeder main, but had less and less rust toward the dead end. The pipe was practically clean between the last lateral and the closed end.

It is a noticeable fact that in most cases of bad pipe corrosion that have come to Mr. Pugh's notice, not only was there salt water, aeration, and more or less motion in the water, but the pipes were laid, as a rule, in muck with rank swamp vegetation and life in close contact with the metal. The action of living matter, therefore, becomes a subject of interest, and will be found to merit careful consideration in many instances in which it has not hitherto been given a thought. The direct action of any organism is doubtful. Apparently, however, iron is sometimes dissolved by some acid secretion from organisms. It is even necessary to consider the earthworm. It has been estimated that on an average some 25,000 worms exist in every acre of soil, and that 15 tons of soil in each acre annually pass through the worms. These worm castings are acid when fresh and contain also small quantities of ammonia. With 15 tons of acidified soil per acre per annum, with a small admixture of ammonium salts, there may well be a corrosive action on any embedded iron.

Our knowledge of the humic acids is very meager. It has been generally considered that they have an important part in the corrosive action of swamp waters. The indirect action of humus may be extensive under certain conditions. Jodidi refers to the fact that the amount and nature of humus formed depends on the organic materials which undergo humification, on temperature, moisture, aeration of soil, presence of salts, acids, etc., and the character and quality of microbes present. The comparatively easily soluble products are leached out and the less soluble ones accumulate in the humus. The humus is further oxidized by the action of air; the carbon to carbon dioxide; the hydrogen to water; the nitrogen to nitric acid; and the sulphur to sulphuric acid; and putrefaction leads to a number of secondary products, such as amino acids and acid amides.

How important these acids may become practically is illustrated by some recent experiences in Illinois. Clifford Older, in a paper before the Illinois Society of Engineers and Surveyors, says:

> In southern Illinois a peculiar condition exists which undoubtedly has a material bearing on the corrosion of steel structures. In that portion of the state popularly known as Egypt a large proportion of the surface soil, when tested, shows a marked acid reaction. This condition exists generally throughout the greater part of the state which lies south and east of the Kaskaskia River and south of the latitude of Mattoon. To a less marked degree, the soil is acid in many places elsewhere in the state where land has been farmed for 50 years or more.
>
> This acid condition does not exist for a depth of more than a few inches, and steel buried at a greater depth does not seem to exhibit any unusual tendency to corrode.

Mr. Older notes the complete destruction of the webs of I-beams and channels having a thickness of ¼ in. in from 10 to 12 years.

Dr. Adeney ascribes the corrosive action of the Vartry water supply of Dublin to the presence of minute quantities of peaty matter, which ferment slowly, with the formation of carbonic acid, as well as small quantities of nitric acid.

The destructive effect of carbonic acid has already been referred to; in swampy waters, for the reasons mentioned, it is high, and in many well waters it is very high. It is, moreover, very soluble in water, so that it is in available form to attack any iron exposed to its influence.

It can be readily understood, therefore, that by their various secretions plants and animals are important accessories in the corrosion of iron.

CONDITIONS INHIBITING CORROSION.

In reviewing the condition contributing to corrosion, it was shown that whereas dilute solutions of alkalis did not retard corrosion, strong solutions neutralize the acids and preserve the iron for an indefinite period. Mr. Allerton S. Cushman found that 5 per cent of lime applied to boggy, sour land exerted a pronounced retarding action on the corrosion of ironwork embedded in it. The same authority further says:

> There should be many cases where the property of alkalis to inhibit corrosion could be made of more practical use than has been done. Whenever iron posts or standards are set directly in the ground the liberal use of slaked lime should be beneficial.

The effect of concrete in protecting iron and steel is well known.

Cast iron is obtained by pouring the molten metal into sand molds. The outer skin, therefore, is silicious. It has long been known that silicon tends to protect iron from corrosion, and Jouve has shown that a 20 per cent alloy of iron and silicon is not attacked by acids, but is virtually uncorrodable. Wallace refers to having seen natives in India forge iron on a stone anvil, and says this iron, which was charcoal iron, does not rust on exposure to the weather, but takes on a fine brown patina. The question was raised whether the stone anvil did not siliconize the skin of the iron. In experiments by Messrs. McCollum and Logan, of the National Bureau of Standards, it was found that, under a given length of exposure in soil, if the corrosion of machined cast iron be considered as 100, that of cast iron still retaining the original surface as cast would be only 93. Krohnke also notes the fact that the surface formed when pipes are cast enables them to resist attack for a long time.

Silicon is an undesirable alloy for iron in many ways, so that a surface treatment, such as naturally occurs in casting, is probably the only method of making use of its rust-inhibiting properties.

Chromium, nickel and copper, when present in small quantities, contribute greatly to the resistance of iron to corrosion. A steel containing about 1 per cent of chromium was found by Hadfield to corrode about half as much as ordinary mild steel. Similarly, 3 per cent of nickel added to steel reduced its relative corrosion in salt water, as compared with open-hearth steel, about 50 per cent.

Phosphorus in iron also retards corrosion, differing in this respect from sulphur, which, by forming sulphuric acid, powerfully attacks the metal.

The coating of iron with zinc and tin is also used to inhibit corrosion. Zinc is electro-positive to iron, and if the two metals are in contact, and are immersed in a corroding solution, the zinc will pass into solution and the iron will remain unaffected. Galvanizing, therefore, is not merely a mechanical shield to the iron, but protects the metal for some distance around it. It is well to emphasize this fact, as it is a point frequently misunderstood. In a pocket manual of tables and information for engineers, bearing the date of 1911, this statement occurs: "With both zinc and iron exposed and in contact with water, corrosion proceeds more rapidly than it would if the zinc were not present." This statement is absolutely misleading and erroneous.

Tin, on the contrary, is not electro-positive to iron. Being itself practically unaffected by corrosion, it protects the iron mechanically. The tin coating, as commercially applied, is full of minute pinholes. It does not inhibit corrosion of the iron at these points, thereby differing from zinc.

A STUDY OF TYPICAL CASES OF DETERIORATION OF CAST IRON PIPE.

With these various facts in mind, an examination of certain typical cases of pipe failure may give information of value to the engineer who is called on to design works involving pipes subjected to deleterious influences.

It has already been noted that most, if not all, pipe failures occur from the influence of salt or saline waters.

A 10-in. main runs southward from Elizabeth, N. J., through the meadows to Morse's Creek. At this point it is continued by a 6-in. line, also running southward. The general elevation of the meadow, as previously mentioned, is about 18 ins. above mean high tide, and the bottom of the pipe is approximately at the level of mean high tide. There is no trolley near. Curiously enough, the 10-in. pipe, which was laid in 1893, has shown no appreciable deterioration except at a point opposite the works of the Standard Oil Co., where escaping acid waters caused corrosion. The 6-in. line, although not laid until 1902, has broken down completely. Both sizes were made by the Warren Foundry Co., and there is no apparent reason for the greater resisting power of the 10-in. line.

The salt marsh is intersected by creeks and ditches, and the surface is largely covered with fox grass. There are patches too wet for the fox grass, and these are covered with what is known as quill sage, a coarse, rank, swamp growth. It is in these patches that the chief damage to the pipe occurs. The coarseness of the quill sage distinguishes the patches covered by it from those on which the finer fox grass is found.

In a ring of the 6-in. pipe two places deeply corroded may be noticed, yet the exterior is smooth, to casual examination sound, and is neither swelled nor pitted, but retains its original sharp outline. At another point in the ring the same thing is noticed, except that a slight crack occurs between the sound and corroded parts in one place. Just below this point the corrosion extends nearly through the pipe. At another point the pipe has been completely penetrated. The interior is smooth and not tuberculated.

The corroded portion has the general appearance and consistency of graphite, and cuts like that substance. When first removed from the swamp it is much softer, more like soft putty. When it is pulverized for analysis it is brown and makes a brown streak. It is [illegible] of the pipe, the marks resembling those made [illegible]

If some filings of the unchanged metal are sifted on a card beneath which is placed a horseshoe magnet, they arrange themselves in magnetic curves. Filings from the corroded part, on the contrary, if also sifted on a card with the magnet beneath, fall as an inert mass. From this it will be seen that, when carried to completion, the iron is completely changed from the metallic state, and that which is not leached out as chloride remains as hydroxide. One piece of the corroded portion contained only 42.2 per cent of iron.

At a part not very deeply corroded the specific gravity of a complete cross section of the pipe was 5.18. The specific gravity of a piece of the corroded part of the pipe was only 2.2. The specific gravity of cast iron, as given by Trautwine, varies from 6.9 to 7.4, and is assumed to be 7.15.

Under the microscope samples of this pipe show the thin plates of free carbon referred to in describing the composition of cast iron, interspersed with the ferrite. It is also noted that the graphite plates are much thicker at corroded points than they are in the unchanged metal.

A piece of the pipe corroded in spots shows that the sharp original outline of the metal remains even after the metallic iron has been completely removed or oxidized. A paraffin dam was placed around this piece of pipe and ferroxyl reagent poured on. Where there was a deep corrosion, as at the bottom of the figure, no change took place. The iron alongside this developed two red nodes, showing the presence of hydrogen ions, and a wavy cloud of blue indicated where the iron passed into solution. A section of pipe, when placed in a petri dish with ferroxyl, strongly developed a red color all along the outside or deeply corroded part, and a perfect mass of blue from all the uncorroded portion. This action is much more vigorous than that undergone by either a steel wire nail, a wrought iron nail or a piece of cast iron soil pipe lying in the same petri dish. The chlorine present in the corrosion doubtless accounts for the vigor of the reaction observed.

At Atlantic City the behavior of the pipe was similar to that at Elizabeth. The conditions also were analogous, the chief difference being that the salt meadow was only 6 ins. above mean high tide, so that the pipe was subjected to more frequent alternations of water and moist air.

It has been shown that aqueous solutions of most acids dissolve iron. If hydrochloric acid is used, ferrous chloride and hydrogen are the resultant products, the former oxidizing into ferric chloride in the presence of air. It has also been shown that, if only a trace of acid is present, the iron will be deposited from the solution as hydrated ferric oxide. It is probable that in salt water the reaction may be represented by the equation:

$$Fe + 2NaCl + 2H_2O = FeCl_2 + 2NaOH + H_2.$$

In the presence of air or of dissolved oxygen in the water this free hydrogen oxidizes to water, the iron in solution yields ferric hydroxide, and the process continues. The chemical analysis of the portion practically free from metallic iron contains as low as 42.2 per cent of that element, a small part of which is in the form of chloride. Clearly, part of the iron has been removed in solution and part remains as the hydroxide, the carbon present in the combined form being left behind, and, as shown in the microscopic sections, has thickened the plates of free carbon originally in the free iron.

The oxides of iron are electro-negative to the metal, so that electrolysis plays its part, as shown by the ferroxyl reagent, and the extraordinary vigor of the reaction is illustrated by the great mass of blue produced.

The chlorides of sodium and potassium, as heretofore mentioned, cannot corrode iron in the absence of air. Magnesium chloride, which is present in sea water in small quantities, is able to do so, but to get the full corrosive effect of sea water plenty of air is essential.

It has been shown that the greater the surface of liquid exposed to the air the more the corrosion is intensified.

In the salt marshes in question the tide rises and falls, passing through a multitude of creeks and ditches all over the meadows. At Elizabeth it ordinarily reaches about to the bottom of the pipe; at Atlantic City it covers it. This affords an ideal condition for corrosion. The liquid at high tide touches or surrounds the pipe; at low tide it falls, air rushes in to take its place, and a moist, saline, oxygenated, spongy mass of earth and roots surrounds the pipe. Carbonic acid probably contributes to the action. The Atlantic City meadow soil was free from organic acids. In the sample of soil taken ½ in. from the pipe there was 46.57 per cent of Fe_2O_3, and even 3 ft. away iron occurred in large quantities, there being 9.11 per cent of Fe_2O_3. This would appear to show that it had been dissolved from the pipe as $FeCl_2$, and after diffusing or flowing away from the pipe, as the tides rose and fell, was deposited as hydroxide.

It is further to be noted that at Elizabeth, where, on account of the elevation of the meadow, the top of the pipe was not so frequently wet as the bottom, the blowouts generally were in the bottom of the pipe. At Perth Amboy the top and upper sides suffered most. At Atlantic City, also, the condition of the pipe was worse at the top than at the bottom. This was to be anticipated on account of the relative elevations of the pipe and high tide. At Elizabeth the high tide reached the bottom of the main, consequently the top was not subjected to the saline influences so frequently. At Atlantic City the pipes ordinarily were covered at high tide and, being laid farther below the surface, air was more plentiful at the top, which, therefore, in presence of an ample quantity of salt water, and with a more abundant supply of oxygen than the bottom, suffered most.

At Syracuse the portions constantly submerged did not give out. Probably, too, the concentrated saline content in these brine-soaked lands produced an inhibiting effect. The damage took place above the permanent water table, where both air and salt water were enabled to reach the pipe.

Analyses of the corroded and uncorroded portions of the Syracuse pipe were made for the Water Department by Dr. Ernest N. Pattee of Syracuse University. He reports:

> The corrosion contained 33 per cent of insoluble residue consisting of about 10 per cent graphitic carbon and 22 per cent of silicon (Si) and silica (SiO_2). Total carbon in the corrosion = 11.2 per cent, consisting almost entirely of graphitic carbon. Sulphur in corrosion = 3.0 per cent. Apparently the process which accounts for the corrosion either dissolves the greater part of the combined carbon or converts it into the graphitic variety.

An analysis of the soil surrounding this pipe, made by Mr. Edward J. Pugh, disclosed that it was acid to phenol-phthalein, and contained 3.42 per cent of organic acids. This is interesting, from its analogy to the soil conditions in Illinois, cited by Mr. Older, where steel bridges were damaged so badly by the acid soil.

On the whole, the evidence chiefly observed with cast iron pipe would seem to indicate that organisms, whether animal or vegetable, had but little to do with the effects observed. Salt water and air seem to be the active agents, probably aided by carbonic acid, and the exclusion of either is followed by increased durability of the pipe.

The evidence given by Mr. Reimer, in which cinders caused pipe corrosion, the similar experience at Syracuse and the German case resulting presumably from local electrolysis due to the presence of gypsum crystals, indicate the necessity for caution where pipe is likely to be subjected to such influences. Some years ago a tunnel leading from the Susquehanna River to the power house at the Central Pennsylvania Traction Co. at Harrisburg was constructed under the supervision of Mason D. Pratt. This tunnel passed through the property of the Central Iron Works, on parts of which there were large slag dumps. The water flowing into the tunnel was so sulphurous from the sulphur in the slag that it greatly impeded the progress of the work and was most unwholesome for the workmen. Cast iron pipe in such a location would undoubtedly undergo rapid deterioration.

Where low lands are filled with cinders ordinarily no trouble need be feared, but when subjected to periodical wetting and aeration some precautions are desirable.

Acid mine waters are also very corrosive and should be guarded against when pipe is subject to their action. In the mines themselves this would generally be economically impracticable, but in municipal works in the vicinity, where the pipe may be attacked, preventive means should be resorted to.

PREVENTION OF EXTERNAL CORROSION.

Bearing in mind the silicious skin of a cast iron pipe and its protective effect, and also noting that chromium alloys of iron are resistant to acid attack, it occurs to the writer that some chromium compound such as chromite mixed with the facing sand of the moulds might produce in castings a protective chromium skin.

It is a subject wherein an apparently well-reasoned argument may be completely upset. For example, as zinc in water, either fresh or salt, protects iron and is itself attacked, so if iron and copper are immersed in a liquid capable of acting on each, and are connected by a conductor, a current will pass from the copper to the iron, and the iron will be dissolved.

Mr. H. J. Force is authority for the statement that cast iron used in pumping machinery by the Delaware, Lackawanna and Western Railroad Co. was badly corroded by acid mine waters. As bronze bushings, valves and impeller blades were used, it was thought that the action was due to galvanic action from contact of the two metals in the acid water. On substituting cast iron for the bronze valves and bushings, so that the parts in contact would be of the same metal, the corrosion increased, and the cast iron deteriorated even more rapidly than when in contact with bronze.

It is unsafe to reason too positively from results obtained with liquid electrolytes as to what will happen to iron buried in soils. Moreover, time is such an important factor that conclusions drawn from accelerated tests may be most misleading. Messrs. McCollum and Logan, in their Bulletin on the Electrolytic Corrosion of Iron in Soils, conclude that:

The relatively high rate of self-corrosion of cast iron as compared to the other kinds of iron tested is contrary to the generally accepted idea that cast iron is more resistant to self-corrosion than wrought iron. It is not improbable that this impression in regard to the superiority of cast iron has grown out of the fact that cast iron structures are usually made relatively heavy and they also tend to corrode more uniformly than wrought iron or steel, both of which factors would tend greatly to increase the life of the former.

Mr. Pugh cannot subscribe to this view. The ordinarily accepted idea of the superior durability of cast iron arises, in his opinion, from the fact that cast iron, generaly speaking, is more durable than other commercial forms of wrought iron and steel.

One explanation of this discrepancy may be furnished by the experiments of K. Arndt, who determined the extent of corrosion in iron and steel by the oxygen absorbed by the metal in rusting. The cast iron at first corrodes as rapidly as the mild steel, and more than twice as fast as the weldless tube. At the end of 43 days, however, the mild steel had undergone twice as much oxidation as the cast iron, and the weldless tube four times as much.

It would seem, therefore, that even in the corrosive zones, in which deterioration of the cast iron pipe has been greatest, the remedy is not to be sought by the substitution of wrought iron or steel, but rather by the improvement, if possible, of the skin resistance of the metal. Where, for any reason, such as with flanged pipe, bolts and nuts are necessary, bronze need not be feared on account of its supposed galvanic effect on the iron.

A second method of protecting the pipe, based on the well-known rust-inhibiting properties of the chromates and certain other substances, whereby the iron is rendered "passive," does not seem practicable, in that a sufficient quantity of such material to protect the iron indefinitely could scarcely be added to the soil surrounding the buried pipe at a reasonable cost. Therefore, this interesting phase of the subject has not been taken up in this paper. A temporary passivity would be of no practical importance.

The use of lime in the trenches, so as to encase the pipe in an inhibiting alkali, ought to be of service, and is worth trying. Encasing in concrete would be expensive, but would protect the pipe, if properly done.

Another method, suggested at Perth Amboy, is to leave the pipe in open trench. This might help materially, as the alternate wetting and drying would probably be less destructive than the constant exposure to moist air, salt, and carbonic acid in the swamp or meadow soil. At Atlantic City this principle has been carried one step farther, and the new 48-in. cast iron water main across the salt marsh has been raised above the surface of the meadow, so as to be entirely in the air. This main is supported on reinforced concrete bolsters, 5 ft. 2 ins. long, 20 ins. thick, and about 5 ft. 6 ins. high, so that the bottom of the pipe is about 1 ft. above the meadow. Each bolster, in turn, is carried on two piles of an average length of 37 ft., driven by a 2,000-lb. hammer, the final penetration required being not more than 1 in. under five strokes of the hammer falling 14 ft. The bolsters are 6 ft. from center to center. Between the pipe and the bolster there is a piece of hard vulcanized fiber, ⅜ in. thick. On the pipe laid in summer, expansion was provided for by allowing a minimum space of ¼ in. between the shoulder and spigot end of each pipe. In addition, a sleeve is inserted once in each 2,000 lin. ft. of pipe. The entire design has been carefully studied and worked out by Mr. Lincoln Van Gilder, Chief Engineer, and T. Chalkley Hatton, Consulting Engineer of the Water Department. Several thousand feet of the main are already completed. It would seem to solve the problem, either in a warm climate or where the main is large enough to be exposed without danger of freezing; and where it is permissible to have the pipe above the surface.

At Elizabeth an analogous, though less elaborate, expedient has been adopted. A 6-in. pipe could not be raised above the meadow and left exposed without danger of freezing, so it was laid on the surface, covered with a mound of meadow soil or mud some 6 ins. thick, and the mound was then sodded with fox grass. This was done in the spring, so that the sod would make vigorous growth and protect the embankment. Although this gives great promise of obviating the trouble, it has not been in place long enough to predict its success with absolute certainty.

Very frequently, conditions are met which prohibit such a solution of the problem. At Syracuse, for example, where the mains are laid in the streets, such a mode of construction is not to be thought of.

Bearing in mind the protective effect of zinc, it might at times be desirable to galvanize the cast iron in such locations, instead of giving it the ordinary bituminous coating. The United States Government has utilized galvanizing in some instances for the protection of gratings, covers, and similar castings, but as far as the writer is aware, it has never been applied to cast iron pipe. Double galvanizing, as it is called, should be used in such cases. This is a trade term, which does not mean that the iron has been dipped twice, but merely that it carries a much heavier coating than the ordinary galvanized metal. The heavy coating is not suitable for articles which may be subjected to bending, as it will crack off, but this objection does not apply to a rigid article like cast iron pipe.

CONCLUSIONS.

First.—Under ordinary conditions of soil, cast iron pipe has a probable life of from one to three centuries, as far as external corrosion is concerned.

Second.—Under certain soil conditions, such as salt marshes or saline soils, cast iron pipe may be rendered useless in from 7 to 20 years.

Third.—At times, cinder and slag fills may exert a strong deleterious influence. Acid mine waters are also destructive.

Fourth.—Substituting wrought iron or steel pipe for cast iron is ineffectual. Cast iron will outlast the others.

Fifth.—Remedies fall under four heads:

A.—Increasing the skin resistance of cast iron.

B.—Utilizing the protective influence of alkalis by surrounding the pipe with lime or cement, where such is practicable.

C.—Exclusion of acids, salt, or air.

D.—Galvanizing the cast iron pipe, thus protecting it at the expense of the zinc.

Sixth.—So many opposing factors are at work simultaneously that great caution is necessary in reasoning from effect to cause and in suggesting remedies based on observed fact. It is the unexpected that frequently happens.

Provisions Governing Water Main Extensions in 135 American Cities.

Practice varies widely as between various cities in the matter of making extensions to the water works distribution system. The present article gives in condensed form the provisions governing water pipe extensions in 135 cities of this country. Cities are given alphabetically under states. The figures following each city's name in parentheses are the population in 1910 and the percentage of total population now supplied, where the latter figure is available. The percentage supplied is very interesting and instructive and, we have no doubt, will occasion some surprise. This figure indicates the large amount of mains which are yet to be laid in many cities which have a public water supply before the population will all be made tributary to this supply.

Alabama.—Anniston (12,794) (60). One hydrant to each 500 ft. of mains ordered. No other requirement. Bessemer (10,864). Just return on investment.

Arizona.—Phoenix (11,134) (98). No special requirements. Mains extended on application of citizens.

Arkansas.—Pine Bluff (15,102) (60). Requirements of $80 revenue for each 500 ft. of water mains extended.

California.—Pomona (10,207). Applicants pay cost of extension and receive all payments for water until amount is refunded. San Diego (39,578). Large mains laid by bond issue. Smaller mains out of department receipts. Stockton (23,253). Mains extended 100 ft. for each consumer.

Colorado.—Denver (213,381) (99). Mains extended by special ordinances.

Connecticut.—Bristol (13,502) (80). Lines extended when revenue gives fair returns on investment. Hartford (98,915). Mains extended upon a guarantee of 10 per cent return on cost. Manchester (13,641). Mains extended when income warrants extension. New Britain (43,916). Mains extended on 8 per cent of cost of investment. New Haven (133,605). Mains extended on application. New London (19,659). Extensions made on guarantee of 5 per cent of cost of line. Stamford (28,836). Mains extended when receipts equal ten per cent of cost of line.

Delaware.—Wilmington (87,411) (98). Mains extended on application.

District of Columbia.—Washington (331,069). Mains laid only on bona fide need of water for existing houses when sewerage is provided and street has been graded. Flat assessment of $1.25 per linear foot on each lot abutting new main. Average revenue of this source past five years $83,293.64.

Florida.—Tampa (37,782) (75). Mains extended on order from city. One fire plug for each 400 ft.

Georgia.—Athens (14,193). Mains extended on application of consumer. Atlanta (154,839) (85). Mains extended whenever demand for water for domestic use and fire protection warrant expenditure. Macon (40,655) (75). Mains extended when the customs warrant expenditure.

Indiana.—Anderson (22,476). Require six services to block. Elkhart (19,282) (70). Mains extended on order from city. One hydrant for each 500 ft. Evansville (69,647) (70). Ordinarily when revenue equals 20 per cent of cost of addition. Ft. Wayne (63,933) (95). When number of consumers warrants extension. Gary (16,802). When income equals 10 per cent of cost of line. Hammond (20,925). When revenue equals 6 per cent of cost of line. Indianapolis (233,650)

(68). Under franchise city has authority to order 40,000 ft. of mains per year, taking one hydrant for each 500 ft. so ordered. Logansport (19,050). When revenue equals 6 per cent of cost of mains. Marion (19,359) (75). Whenever revenue justifies extension. New Castle (9,446). Mains extended on guarantee of 10 per cent on cost of line. Richmond (22,-324). Mains laid on orders from city where revenue to each 500 ft. equals $49 from domestic use, or city takes one hydrant at $49. South Bend (53,684). Lines usually laid where revenue equals 10 per cent of cost of line. Terre Haute (58,157) (60). Mains are laid on orders from city with one fire hydrant for each 365 ft. Company lays voluntarily when consumption is $50 for each 365 ft.

Iowa.—Burlington (24,324). Ordinance requires laying of two miles of pipe each year provided revenue equals 6 per cent of cost. Council Bluffs (29,292) (66). Mains laid under contract with consumers with requirement of one consumer to every 50 ft. Davenport (43,028). Mains laid on order of city council with provision for one fire hydrant and six consumers to each 400 ft. Des Moines (86,368). Additions made on order of city council with requirements of two miles per year. Dubuque (38,494). Extensions made when petitions pay 15 per cent annually on cost of mains. Iowa City (10,091) (75). Additions made on order from city council with one hydrant at least $12 to each 400 ft. Keokuk (14,008). Additions made on one hydrant for every 732 ft. or five consumers for each 366 ft. Sioux City (47,828). Line laid when revenue equals 10 per cent of cost. Waterloo (26,693). Extensions made where petitioners guarantee 15 per cent on cost for five years.

Kansas.—Atchison (16,429) (60). Contract requirement extensions at least one consumer to each 100 ft. Ft. Scott (10,463). Contract requirement extensions at least one consumer to each 100 ft. Hutchinson (16,394). Extensions made for six consumers and one fire hydrant for each 450 ft. Parsons (12,463). Mains are extended with provision of one hydrant each 600 ft. Pittsburg (14,755) (90). Extensions made on guarantee of 10 per cent on cost. Wichita (52,450). Extensions made on order from city with one fire hydrant for each [illegible] ft.

Kentucky.—Lexington (35,099) (70). Mains laid on order from city with hydrant for each 400 ft., or by payment of cost of line which is rebated from service within limit of eight years. Louisville (223,928) (90). Mains laid when revenue equals 10 per cent of cost of lines. Newport (30,309). Additions made when revenue equals 8 per cent of cost.

Louisiana.—Alexandria (11,213). Extensions made wherever revenue justifies. New Orleans (339,075) (71). Extensions made where revenue equals 6 per cent of cost of line.

Maine.—Brunswick (6,621). Extensions are made where revenue equals 10 per cent of cost of line.

Maryland.—Hagerstown (16,507). Extensions made on request of mayor with provision for one hydrant for each 500 ft.

Massachusetts.—Beverly (18,650) (100). Extensions are made whenever the revenue justifies the line. Brookline (27,792). Extensions are made where revenue equals 5 per cent of cost of main. Clinton (13,075). Extensions made on vote of Water Commis- [illegible] are made where revenue or deposit equals [illegible] 295) (99). Ordinance allows city to order 25,-000 ft. per year. Framingham (12,948). Extensions made by board, usual requirement of 8 per cent of cost. Gloucester (21,398) (90). Extensions made on vote of water commissioners. Haverhill (44,115). Lines extended whenever consumers justify extension. Lynn [illegible] antee of 5 per cent of cost. Malden (44,404). Extensions made on guarantee of 7 per cent of cost of main. New Bedford (96,652). Extensions made whenever revenue equals 6 per cent of cost of main. North Hampton (19,-431). Additions are [illegible] per cent on cost of line. Somerville (77,236). Extensions are made whenever revenue justifies laying of main. Springfield (88,926). Mains extended on guarantee equivalent to 15 cts. annually per foot of pipe.

Michigan.—Adrian (10,763). Additions made on basis of one consumer per 100 ft. and one hydrant per [illegible] ft. of pipe. Ann Arbor (14,817) (80). Additions made on orders from city with one hydrant to each 700 ft. of pipe. Battle Creek (25,267). Mains are laid on guarantee of 7 per cent on an estimated cost of $1 per foot. Bay City (45,-166) (75). Extensions made on petitions. Detroit (465,766) (99.9). A bonus of 5 per cent for three years on the estimated cost of extension is required less the water rates in sight when line is laid. Flint (38,550). Extensions made when revenue equals 6 per cent of cost of line. Holland (10,490). Extensions made when revenue equals 10 per cent of cost of line. Ishpeming (12,448). Extensions made on order of city council.

Minnesota.—Austin (6,960). Extensions made on guarantee of 6 per cent of cost of line. Duluth (78,466) (75). Extensions are made on guarantee of 8 per cent annually on cost of main. Minneapolis (301,408) (95). Extensions made on payment of actual cost of line with maximum of 70 cts. per front foot. St. Paul (214,744). Water mains laid on assessment of 10 cts. per lineal foot of frontage of each lot in front of which main is laid. This assessment runs for ten years.

Mississippi.—Jackson (21,262) (90). Extensions made when number of consumers justify investment. Meridian (23,285) (72). Extensions made on a five-year guarantee of 25 per cent of the cost of the main.

Missouri.—Kansas City (248,381) (95). When number of consumers justify the expenditure. Independence (9,859). Extensions made on basis of one consumer to each 100 ft. St. Louis (77,403) (99.9). On petition of property owners. Sedalia (17,822). Extensions made on orders from city council with 10 hydrants to mile or on basis of $50 for domestic consumption for each 600 ft. Springfield (35,201). Extensions are made on basis of hydrants to each 600 ft. or 75 ft. to each consumer.

New Hampshire.—Dover (13,247). Extensions made on basis of guarantee of 5 per cent.

New Jersey.—Newark (347,469) (100). Extensions made on basis of 10 per cent of cost of line. Paterson (125,600). Extensions are made on basis of guaranteed revenue of 10 cts. per foot per annum.

New Mexico.—Albuquerque (11,020). Additions made on orders from city council on basis of 10 hydrants per mile and domestic revenue equal to 6 per cent of cost of line.

New York.—Binghamton (48,443). Frontage tax for all mains, bringing to the department about $10,500 annually. Buffalo (423,-715) (100). Extensions made on orders from common council. Geneva (12,446). City is figuring on frontage tax but has not definitely decided on amount. Kingston (25,908). Extensions made on basis of 6 per cent guarantee. Little Falls (12,273). Extensions made on guarantee of revenue equal to 10 per cent on investment. Annual frontage charge of 2 cts. a foot on all streets where mains are laid. North Tonawanda (11,955). Additions made on orders from board of public works. Troy (76,813). Extensions made on orders from common council. Have yearly frontage tax of 20 cts. on improved property and 2 cts. per foot on vacant property. This covers water for family use, etc. Utica (74,419). Extension made on order of council with one hydrant to each 525 ft. or on petition of property owners on guarantee of 10 cts. per foot or 10 per cent on actual cost for 5 years.

North Dakota.—Grand Forks (12,578). Extensions are made by water department and cost of line up to 6 ins. assessed against property, assessment being divided into 10 equal payments. Trunk mains in excess of 6 ins. are paid for by department.

Ohio.—Cincinnati (363,591). Extensions made on guarantee of 10 per cent per annum on cost of 6 in. main. Where mains are laid in newly improved streets the cost is assessed against abutting property. Force mains in excess of 6 ins. paid for by city. Extensions made on assurance of 6 per cent interest on investment for 10 years. Columbus (181,511) (94). Extensions made on assurance of 6 per cent interest on investment for 10 years. Delaware (9,076). Extensions made on order of city council with one hydrant for each 440 ft. Elyria (14,825). Extensions made on guarantee of 10 per cent of cost of line. Fremont (10,000). Extensions made on requirement of one consumer to each 100 feet of mains. Massillon (13,879) (98). Extensions made on guarantee of one consumer for each 528 ft. Piqua (13,388). Extensions made where income exceeds 60 per cent of cost of investment. Springfield (46,921). Extensions made when revenue equals 6 per cent of cost. Zanesville (28,026). Extensions made when revenue equals 10 per cent of cost.

Oklahoma.—Guthrie (11,654) (50). All extensions made by bond issue.

Pennsylvania.—Allentown (51,913) (100). Extensions made by ordinance of city council. Frontage charge 4-in. main, 30 cts.; 6-in. main, 50 cts.; 8-in. main, 70 cts.; 12-in. main, $1.05; 16-in. main, $1.50; on each side of street. Altoona (52,127) (99). Additions made from revenues of water department. Frontage charge of 25 cts. per foot. Average revenue for past 5 years $15,000. Erie (66,-525) (100). Extensions made on bond insuring annual return equal to 7 per cent of cost. Johnstown (55,482) (75-80). Extensions made when revenue equals 10 per cent of cost. Philadelphia (1,549,008) (100). Extensions are made on authority of councils. Frontage tax of $1 for both sides of street or $2 per lineal foot of pipe laid. Receipts from this source have averaged $131,624.95 for the past five years. Reading (96,071) (99.9). Extensions made on petition of property owners. Frontage tax of 50 cts. per foot except on corner lots where allowance is made of one-third of length of lot. Average receipts for past five years $2,443. Wilkinsburg Sta., Pittsburgh (18,924). Extensions made when revenue is equal to 15 per cent of cost of line. If cost does not equal that, petitioner pays cost of line and is rebated.

Rhode Island.—Providence (224,326) (100). Extensions made when revenue equals 7 per cent of estimated cost.

Tennessee.—Memphis (131,105). Extensions made on basis of one consumer to each 100 ft. In new sub-divisions owners pay cost of line and receive rebate.

Texas.—San Antonio (96,614) (90). New city contract provides for 8 miles of pipe per year but revenue of 12 cts. per foot must be guaranteed before mains are ordered.

Utah.—Ogden (25,580). Extensions made on petitions.

Virginia.—Roanoke (34,874) (80). Extensions made when cost of main is deposited with water company. Said deposit is held until income on line equals 10 per cent of cost.

Washington.—North Yakima (14,082). Extensions made whenever business will warrant. Usually figured on basis of 33⅓ per cent return on gross cost of line. Seattle (237,194) (95). Extensions made on petition of property owners or on order of city council, entire cost of 8-in. cast iron pipe assessed against property owners. Excess above 8 ins. paid for out of water fund. Spokane (104,402) (95). Extensions made upon petition of 51 per cent of property affected. Cost paid by assessment against property. Tacoma (83,743). Extensions made on local improvement district plan. The abutting property in residence district pays cost of new main figured on basis of 6-in. line. Cost above this cost being paid by water department. In manufacturing or congested districts, the entire cost irrespective of size of main, is assessed against property benefited. Walla Walla (19,-364). Extensions in old part of city made at the expense of water department. In the new additions a frontage charge is made. The money so received is merely used as a credit on the water rent.

Wisconsin.—Ashland (11,594). Extensions made on order of city. City paying for hydrants and hydrant rental at the rate of 10 hydrants per mile of main. La Crosse (30,417). Extensions are made upon petition and order of the city council. Madison (25,531). Extension made upon petition, paving of street and connecting ends to insure circulation. Property pays entire assessment in front for 4-in. and 6-in. mains. Excess over 6 ins. is paid by city. Milwaukee (373,857) (100). Extensions made on petition. Abutting property is assessed on the basis of the cost of laying a 6-in. main. Excess of 6 ins. is paid out of water fund. Assessment for past five years has been 50 cts. per lineal foot on each side of street.

Wyoming.—Cheyenne (11,320) (99). Extensions made upon petition of a majority of property owners along the line of proposed extension.

ACKNOWLEDGMENT.

The foregoing data have been rearranged from the recent report of the committee of the American Water Works Association on Tabulation of Water Rates and Other Information of Interest to Water Companies.

Cost of Laying Cast Iron Water Pipe for Railway Service.

The accompanying tabulations, Tables I and II, give cost data on laying cast iron water pipe for railway service. The data are taken from the report of the Committee on Water Pipe of the American Railway Bridge and Building Association, as presented at the annual convention held at Los Angeles. Mr. C. R. Knowles, foreman of water service of the Illinois Central Railway, is chairman of the committee.

TABLE I.—AVERAGE COSTS OF LAYING CAST IRON WATER PIPE IN ACTUAL RAILROAD PRACTICE.

Diameter of pipe, ins.	4	6	8	10	12	14
Pipe at $30 per ton	$0.36	$0.54	$0.78	$1.06	$1.38	$1.53
Yarn at 7 cts. per lb	.0012	.002	.003	.0035	.004	.005
Lead at 5 cts. per lb	.031	.0425	.05	.07	.08	.09
Loading and handling	.01	.015	.02	.025	.03	.04
Trenching and refilling (4 ft. cover)	.11	.15	.16	.18	.20	.22
Laying, caulking, etc	.05	.07	.10	.12	.15	.17
Total per lin. ft.	$0.56	$0.82	$1.11	$1.46	$1.84	$2.06

The cost of pipe laying will vary greatly owing to local conditions, cost of labor, etc., but Table I gives the approximate cost as determined by actual railroad practice. The figures are the costs per lineal foot of pipe laid.

The figures in Table I are based upon class "C" pipe for a 300-ft. head and 130 lbs. pressure, with the exception of 14-in. pipe, which is based on class "B" pipe for a 200-ft. head and 86 lbs. pressure. Fourteen-inch pipe is usually installed only for penstock lines with a comparatively low head.

Table II gives the cost of laying cast iron water pipe on the Iowa Division of the Chicago & Northwestern Railway.

The following notes pertain to Table II: Ditches average 2½ ft. wide and 5 ft. deep. Earth chiefly clay sufficiently stiff to stand. Braces used more or less. No curbing and very little trouble on account of water. Wages, laborers, 20 to 22½ cts.; caulkers, 25 cts.; foreman, 30 to 32½ cts. per hour, with 5 per cent added for superintendence and accounting. Price of material includes special fittings and valves. Average depth of lead joints, 1¾ ins. to 2 ins.

TABLE II.—COST OF LAYING CAST IRON PIPE, IOWA DIVISION, C. & N. W. RY.

Material:						
Diameter of pipe, ins.	4	6	8	10	12	14
Weight per lin. ft., lbs	22	34	47	64	82	103
Lead per joint, lbs	9	10	13	15	20	23
Jute per joint, lbs	0.5	0.75	0.75	1.0	1.0	1.25
Total cost of material, lin ft	$0.38	$0.60	$0.68	$1.10	$1.45	$1.65
Labor:						
Excavation, per lin. ft	$0.23	$0.27	$0.32	$0.35	$0.40	$0.48
Backfilling, per lin. ft	0.04	0.04	0.04	0.05	0.05	0.06
Jointing, per lin. ft	0.03	0.04	0.04	0.05	0.05	0.06
Total cost of labor, lin. ft	$0.30	$0.35	$0.40	$0.45	$0.50	$0.60
Total cost material and labor, per lin. ft	0.68	0.95	1.08	1.55	1.95	2.25

RAILWAYS

Methods and Instruments Used in Making Precise Surface and Underground Surveys for the Canadian Northern Ry. Tunnel, Montreal, Canada.

Contributed by Mr. J. L. Busfield, Assistant Engineer, Department of Surveys and Alinement, Mackenzie, Mann & Co., Ltd., Montreal, Can.

In the construction of the new transcontinental line for the Canadian Northern Railway it was deemed necessary that a terminal station should be constructed in the heart of the city of Montreal, and the most practicable method of reaching a desirable site with the lines already constructed to the east and west of the city was found to be that of building a tunnel through Mount Royal, situated on the northwest side of the city. In addition to the surveys for the tunnel and passenger terminal, provision had to be made for the construction of an overhead railway viaduct from the tunnel portal to the water front.

Figure 1 is a plan and profile of that portion of Montreal through which the railway passes; it will be seen that the city proper is situated between the southeasterly slopes of Mount Royal and the shore of the River St. Lawrence, while the tunnel is located to pass under the northerly part of the mountain where it reaches its greatest height. A site for the west portal was selected in the neighborhood of the belt line of the Canadian Pacific Railway, and the east portal was located at Cathcart street. For the preliminary surveys a rough line was projected over the mountain in the approximate line of the tunnel and levels were run over this line by ordinary methods requiring no description. From the information thus obtained a definite line and grade could be determined for the tunnel. As will be seen by Figure 1, the location selected for the tunnel gives a tangent about 2½ miles long from the west portal to a point near McGill University, where the alignment follows a 2° curve into the city.

Fig. 1. Plan and Profile of Mount Royal Tunnel, Canadian Northern Ry., Montreal, Can.

SURFACE SURVEYS.

The steep slopes of the mountain made it obviously impossible to make direct measurements between the two ends of the tangent with any degree of accuracy, so it was decided to run a traverse around the side of the mountain. A suitable route was chosen for this traverse, selecting, and permanently marking, angle points as far apart as possible, keeping away from hills and rough ground.

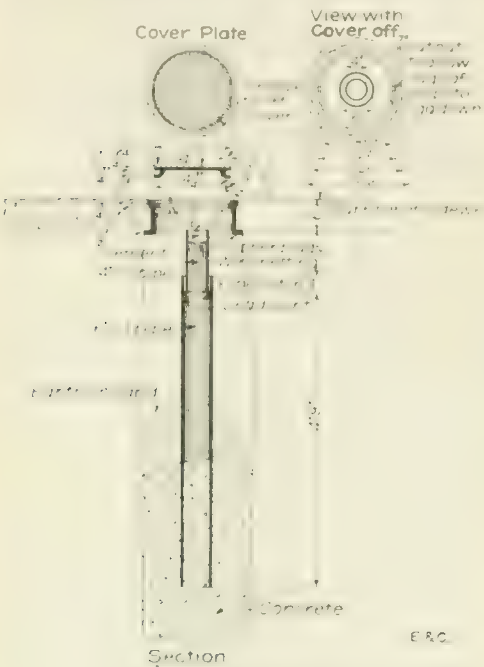


Fig. 2. Survey Monuments Placed on Base Line for Mount Royal Tunnel, C. N. Ry.

Survey Monuments.—All the angle points having been selected, they were permanently marked by drilling a ¼ in. hole into the stone or concrete of the sidewalks about ½ in. deep, and hammering into this hole a piece of ¼ in. hollow brass tubing. A copper rivet hammered into the tube then completed the point, with the exception of the punch mark or knife scratch to mark the precise point. Ten to fifteen minutes is ample time for two men to set one of these points and many of them have remained in place for nearly two years. In the city itself, in order to hold some of the survey points more permanently than was possible with the rivets, some of the more important ones were selected and monuments sunk to a depth of 8 ft. in order to insure that they would not be disturbed by frost, which frequently reached a depth of 6 to 7 ft. The type of monument used is illustrated by Figure 2.

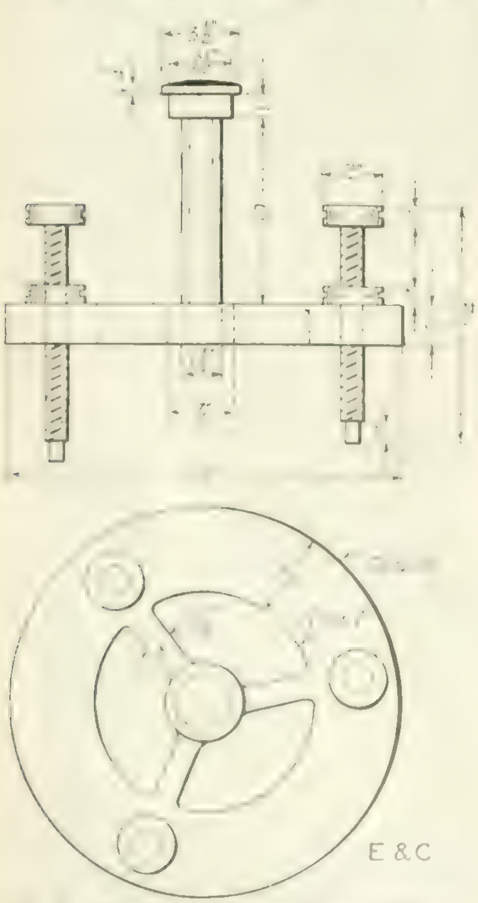


Fig. 3. Details of "Spider" Used in Measuring Base Line for C. N. Ry. Tunnel.

Base Line Measurements.—In order to make the traverse around the mountain sufficiently accurate, the length of the route being about four miles, it was necessary to adopt some form of base line measurement. A large proportion of the lines to be measured came on sidewalks or roads so it would have been impossible to use any form of measurement requiring stakes, and a form of portable measuring point was therefore used, called a "spider," made of a cast iron wheel carried on screw legs and supporting a vertical piece with a brass head on which is the cross scratch to which measurements were made. (See Fig. 3.) Four of these spiders were used, each weighing about 44 lbs.

Previous to making the precise measurements "spider" points were marked on the sidewalks by means of a chiselled cross every 99 ft. on the lines of the traverse, being put on line either by eye or by a transit, whichever was easier. Where the lines were not on sidewalks the spider points were marked by driving ship spikes into the ground. While these were being marked by one party of four men at the rate of about two miles per day, a second party consisting of leveler and rodman was following and taking the elevations of all the spider points and entering them in a book specially provided for that purpose. Before any of the actual measurements were made all the tapes that were to be used were compared with a standard tape, standardized by the Bureau of Standards under the same conditions as were to be chosen in the field, namely, under a tension of 12 lbs. and supported at intervals of 20 ft. Corrections were also made to a temperature of 62° F. Steel tapes 100 ft. long and ¼ in. wide, divided into hundredths of a foot, were used, the thousandths being estimated by the observer.

The actual modus operandi of making the measurements (illustrated by Fig. 4) was as follows: Two spiders would be set up at adjacent spider points and the tape stretched across their tops, attached to the forestay at the front end and to a cord passing around a bicycle wheel with a 12 lb. weight attached at its rear end. Four wooden standards were then lined in by eye at 20 ft. intervals, and the hooks for supporting the tape set on a straight line between the tops of the spiders so that when hooked up and with tension applied the tape would practically conform to a straight line between the tops of the spiders. The height of each spider above its spider point being measured and the elevation of the spider point being known, the difference in elevation of the two ends of the tape could then be obtained and hence the correction to horizontal. Thermometers were suspended from the standards and readings taken for every set up so that the proper correction for the expansion or contraction of the tape could be made. When the tape was properly hooked up and had the correct tension, readings of the intersection of the cross hairs or scratches on the two spiders were taken by the two observers at either end of the tape and called out to a recorder, who entered them in the field notebook and rapidly substracted the two readings. The tape would then he moved slightly over the tops of the spiders so that a different pair of readings would be obtained and again called out to the recorder. At least four readings would be taken in this way, but if the differences thus obtained varied more than one or two thousands, the readings were continued until a reliable measurement was obtained. In addition to the tape readings the recorder also booked the height of the spiders and the

Fig. 4. Method of Measuring Base Line Showing Use of "Spiders," Bicycle Wheel, Tension Outfit and Intermediate Supports for Tape.

LINE 25-26 Date — Party ....

Points	Readings	Diff.	Aver. Diff.	Elev. of Point	Height of Tape	Elev. of Tape	Diff. of Elev.	Temperatures. 1	2	3	4	Aver. Temp.
Spider 3	99.851 839	99.012		339.75	1.33	341.08						
	99.877 .863	99.014										
	99.950 .939	99.011	99.014				3.46	79.3	78.4	80.2	78.8	79.2°
	99.994 .983	99.011										
	99.793 .782	99.011										
	99.806 .795	99.011										
Spider 4	99.997 1.000	98.997		343.18	1.36	344.54						
	99.921 924	98.997										

Fig. 5. Sample Page of Field Notes Taken on Base Line Measurement.

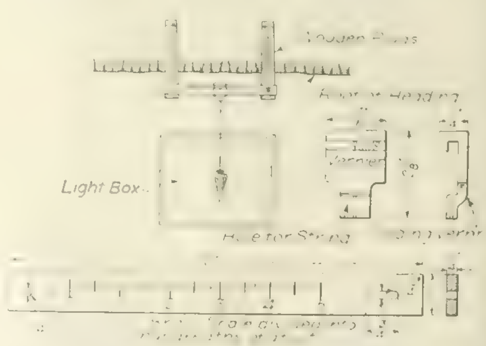


Fig. 6. Details of Brass Scale and Sliding Vernier, Also Method of Attaching to Tunnel Roof.

mean temperatures. A portion of a typical page from the recorder's field book is illustrated by Fig. 5. After a set of readings had been taken the whole apparatus was carried ahead and a new measurement made in the same way, but one spider was always left in place behind the one actually being measured from, so that in case of a spider being accidently moved the other could be used again, thus saving the party from having to go over the same ground twice. At the starting point and at all rivet stations instead of one of the spiders being placed over the rivet it was set up at one side merely as a support for the tape, and the reading on the tape was obtained by transferring the point on the rivet up to the tape by means of a transit placed a short distance away from, and at right angles to, the tape.

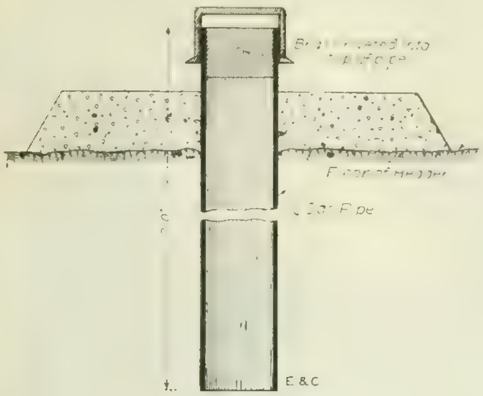


Fig. 7. Survey Monument Used for Underground Surveys Mount Royal Tunnel, C. N. Ry.

This method of measuring required ten men, namely chief of party, transitman, recorder, 2 chairmen, 2 spider-placers, 1 thermometer reader, 1 man for forestay, 1 man for tension wheel. Under favorable conditions as many as 1,500 to 1,800 ft. were measured per hour, but ten spider points per hour made a good average for a day's work on the sidewalks; good time was made by establishing friendly rivalry between the forestay and tension wheel men, each trying to be the first to call "Ready for tension." The majority of the work was done in the day time but in the busy sections of the city it was necessary to work during the night time.

Measurement of Angles.—Great care had to be taken in reading the angles of the traverse, a Berger transit, 7¼-in. plate, reading to ten seconds was used for this work. In setting the instrument up over the angle points a second transit was used to insure the placing of the vertical axis of the large instrument exactly over the center of the cross on the rivet. At the two points sighted at wooden targets were used, being set up vertically and precisely over the points by means of a transit. In reading the angle one observer would read the angle once, "wrap it up" five times and then go backwards until zero was again reached on sighting at the foresight target. This method would give a very close approximation of the angle to 2", and this process being repeated by an independent observer a reading to about 1" could be obtained. This part of the work was necessarily slow and tedious, requiring a party of four men, who were only able to read about five or six angles per day.

A very important part of the preliminary work was that of establishing over the mountain a tangent in the same vertical plane as the tunnel center line. The line was first run over the mountain in short stages and after the necessary cutting had been done through the wooded parts the transit points were reduced to three in number. At one of these bed rock was obtained but at the other two concrete monuments had to be built to hold the line permanently.

UNDERGROUND SURVEYS.

The construction on the tunnel was commenced in July, 1912, by driving a bottom heading eastwards from the west portal, and shortly after this date the two shafts shown in Fig. 1 were commenced. These shafts, called the Dorchester and Maplewood, or working Nos. 1 and 3, were 58 ft. and 238 ft. deep respectively; a bottom heading 12 ft. wide and 8 ft. high was driven westwards from the former and headings were driven in both directions from the latter. The Maplewood shaft was holed through to the west portal in April, 1913, and the headings were completely holed through from end to end in December, 1913, with an error in alignment of ¾ in. and in grade of ¼ in. at the final meeting point, which was below the highest point of the mountain. Throughout the work at the western end the tunnel center line was determined and permanently marked in the floor of the headings by monuments, but at the eastern end it was found advisable on account of the curve to run an engineer's line entirely separate from the tunnel center line, with tables giving the relation of the latter to the former.

Monuments and Scales.—Before proceeding with a description of the methods used in laying out the lines described above, note must be made of a piece of apparatus used very frequently and found almost invaluable in the alignment work. This was a brass scale fitted with a sliding vernier as shown by Fig. 6. Whenever it was necessary to obtain an average of a number of points set by a transit one of these scales was used, being rigidly attached to the roof timbers or to plugs set into the roof, with a plumb-bob suspended from the sliding vernier in front of a light box or illuminated screen. They were only used as a means of obtaining averages and for temporarily holding the engineer's line, and as soon as a good set of readings was obtained the line was transferred to a permanent monument set in the floor of the heading. Different types of monuments were used but the one found most satisfactory is illustrated by Fig. 7, and was made from 2 ft. length of 1½-in. gas pipe with a piece of brass solidly riveted into the top. For giving center line and grade at the working face, spads were set in the roof every 50 or 60 ft. throughout the tunnel.

Fig. 8. Method of Alining Shaft Plumbing Wires, C. N. Ry. Tunnel at Montreal.

At the west portal the heading was driven direct from an open cut so the alignment was readily transferred into the heading by setting up a transit in the cut and sighting directly onto a scale placed as far into the heading as possible. At the Dorchester street shaft or working 1-W, the line had to be transferred down the shaft, which was 20 ft. long by 10 ft. wide, and before the main tunnel tangent was reached four angles had to be deflected, two on account of the curve at McGill University, and two because of the fact that the shaft had to be located 30 ft. to the north of the tunnel center line. It was of importance that these angles should be turned off accurately, but it was of equal importance that the angle points should be very carefully located with regard to chainage, and in order to insure the requisite accuracy, the chainage was carefully transferred down the shaft by means of a plumb line, and from there to the last angle point base line measurements were made in the same way as already described for the surface surveys.

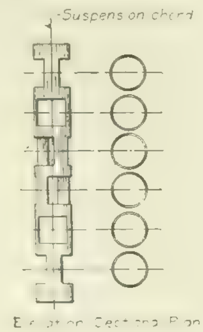


Fig. 9. Special Movable Target for Tunnel Alinement Surveys.

The Maplewood shaft had to be located about 24 ft. to the south of the tunnel center line on account of property restrictions, consequently the lines for driving the headings had to be transferred down the shaft and then offset over 24 ft. to the center line, and run east and west. Great accuracy was not essential in this part of the work as the headings only had to be driven about 2,100 ft. before meeting the heading from the west portal, after which the line was carried through from the portal. However, when the headings were actually holed through the error in alignment and grade was negligible.

Shaft Plumbing.—For transferring the survey lines down the shafts, No. 8 steel piano wires were used, suspended as far apart as possible. They were hung from reels attached to solid wooden frames and were passed over a notch in a tangent screw on the front of the frame by means of which they were finally adjusted to their exact position at the surface, as shown by Fig. 8. The two wires were very carefully set on line at the surface and an instrument man was kept on watch during any series of observations, a precaution which was fully justified, as frequently the wires would be very slightly jarred off line by some careless laborer or by the vibration caused by the nearby hoisting engine. At the lower ends of the wires 12 and 30-lb. weights were suspended at the Dorchester and Maple wood shafts respectively, and immersed in pails of water to reduce the swinging and oscillation to a minimum. In producing the line into the heading the transit man would set up the transit about 10 ft. away from the nearest wire and would then "buck" into line until the transit cross hair would bisect each wire. In doing this, when sighting at one wire the other would be out of focus to such an extent as to be invisible. When the transit man had got on line he would throw the line ahead onto a scale which had been previously set up as far away from the instrument as possible. The operation was repeated a large number of times and contained until enough readings had been obtained on the scale to leave no room for error. The observers at transit and scales were frequently changed, the transit was also always reversed and set off line between all readings.

Turning Angles and Producing Tangents.—It has already been mentioned that in the heading from the Dorchester shaft it was necessary to turn off four angles very precisely as the exactitude of the main tangent depended largely on the accuracy with which these angles were turned off. A ten second transit was used, and it was first set up over the angle point by means of a second transit so as to eliminate any possible error due to inaccurate centering of the plumb-bob, etc. The angle was then turned off roughly and a small mark made on the foresight monument, the precise angle to this point being obtained by wrapping up the angle five times and taking the mean. By taking the difference between the angle thus obtained and the angle as it should be, the distance between monuments being known, it was then possible to calculate the distance the true point should be to the right or left of the preliminary point. After a new point had been taken in this way the angle was again wrapped up five times, by different observers, and the whole operation repeated until there was no possibility of an error of even one second.

After the last angle had been turned off to a point as far away as possible, it was necessary to produce the tangent westwards and at the same time the tangent was gradually being produced eastwards from the west portal. In order to produce the line from any two monuments a scale was set up at the foresight and instead of setting up the transit immediately over the intermediate monument it was set up about 10 ft. away and bucked into line between the adjacent monument and the backsight, thus eliminating the use of a plumb-bob. A large number of readings would be taken on the scale and the average transferred to the monument below by means of a transit. In making some of the long sights necessary on the tangent the ordinary plumb-bob was not satisfactory as a sighting point and some special targets were used; one of these which was found to give very satisfactory results was designed by Mr. A. F. Duguid, transit man of the western division, and was used in places where a movable target was necessary. United States and Canadian patents have been applied for in connection with this type of target (illustrated in Fig. 9). The one originally used was made from an 18-in. length of $2\frac{1}{2}$-in. seamless steel pipe by cutting in it pairs of slots 2 ins. long and exactly opposite each other at intervals in the whole length of the pipe. These slots were accurately cut to 0.001 in. and the hole in the cap through which the suspending cord was passed was also accurately centered; both ends of the tube were weighted with lead to give stability. When suspended in front of an illuminated screen this target presents a silhouette which encloses a series of bright areas of varying width. If the vertical cross hair of the transit is made to bisect these bright areas the line of sight will be in the same vertical plane as the point of suspension of the target. It is immaterial from which direction the target is sighted at, as there are always a number of bright areas in view. Good results were obtained with this target at distances of as much as half a mile in a smoky heading.

Leveling.—It was of great importance that the grades of the opposing headings should be very exact, consequently the levels were transferred along the headings with great care and precision. The permanent monuments already described were used for bench marks, and all elevations were independently checked by different observers.

For the actual alignment of the headings alone great precision would not have been necessary, as an error of even two or three feet in line would have caused no serious effects, but a great deal of excavation work was being done back of the headings, in breakups, and it was essential that the alignment should be accurate for this part of the work. In the case of the elevations it was of course necessary that they should meet within an inch or two at the most, as an error of say three inches would have meant either additional concrete for the track on one side or else additional excavation on the other.

Fig. 2. View Showing Old Wooden Lining Blaw Forms, Concrete Delivery Pipe and Top of Concreting Car, Burlington Tunnel at Arminto.

Procedure and Progress in Placing Concrete Lining in Burlington Railroad Tunnel at Arminto, Wyo., by Compressed Air Method.

(Contributed.)

The Chicago, Burlington & Quincy Railroad has in progress the construction of a line connecting Thermopolis and Orin Junction, Wyo. This is the final link connecting the Colorado Southern with the extension between Billings and the Colorado Southern Railway, passing through Cheyenne to Denver. This completes the so-called Gulf-to-Sound route of the Burlington system. At the Summit of the grade between Casper and Thermopolis is a tunnel 800 ft. long. This tunnel was excavated, a year ago, in rather treacherous sandstone which required timbering throughout its length. The tunnel is now being lined with concrete and the timber is being removed as the concrete lining is put in. The compressed air method of mixing and placing the concrete is being used, as described in the present article.

The tunnel is approached with long cuts on both ends; the west end having an approach cut nearly a mile long, and the east end having a cut about one-half mile in length. Materials for concrete are brought in gondola cars from Casper, where the sand and gravel is dredged from the river. This material is stored alongside the main line near the entrance to the tunnel approach.

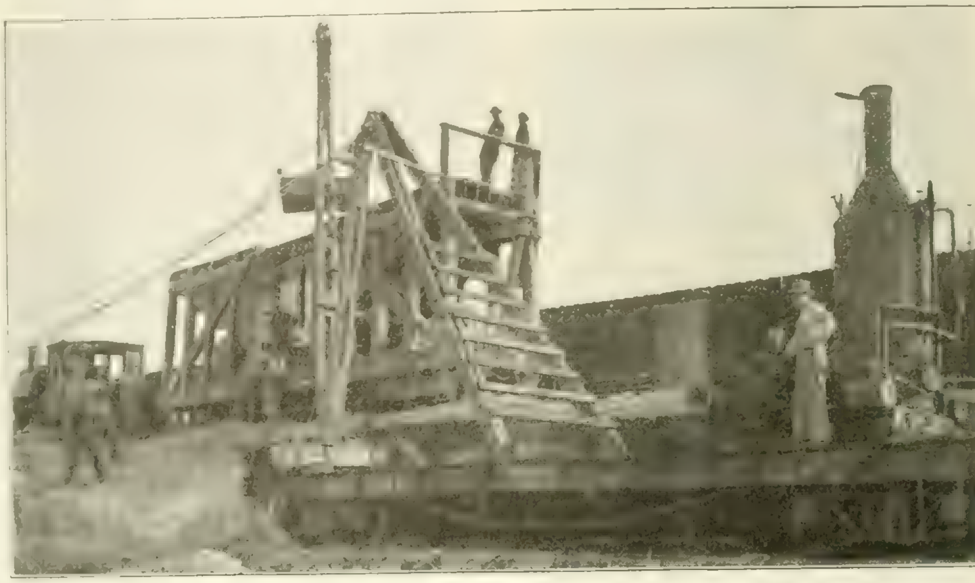


Fig. 1. View Showing Loading of Bins on Mixer Car for the Lining, by Compressed Air Method, of C., B. & Q. R. R. Tunnel at Arminto, Wyo.

Cement is stored at the same side in a shed built for the purpose. Water for steaming and mixing purposes is brought to the site of the work in railway tank cars. The concreting car, described later, is loaded by means of a portable derrick from the material piled along the main line, as shown in Fig. 1. The derrick handles a wooden skip, as a clamshell bucket was not at first available. This skip is loaded by several men who shovel the bank-run gravel into it.

Three operations constitute the work of lining the tunnel, first, the removal of tim-

Fig. 3. View Looking Through Steel Forms at Delivery End of Car During Concreting.

The joint in the delivery pipe just below the springing line of the steel arch centering is a loose flange joint. The pipe turns from side to side at this point during concreting. When the car is taken out to be loaded the pipe is disconnected at this joint, and the upper part is left suspended from the timbers by block and tackle shown.

bers; second, the erection of the Blaw steel forms, and third, the mixing and placing of the concrete.

The timbers, which are shown in Fig. 2, are pulled down by means of cables fastened to the bottoms of the uprights and passed through snatch blocks, placed in the center of the track, and thence to a dinky engine. When the posts are pulled out the roof falls and the debris is cleared away and loose rock is picked down to insure the safety of the men erecting the forms. To further protect the men, planks are laid from the remaining timbers to the concrete so that in case any small pieces of rock should fall it would be caught on these planks, or if a large rock should come loose it would be impeded by the planks and would give warning in time to permit the men to get out of the way. The amount of timber taken down at one time depends upon the apparent safety of the rock above and varies from 5 to 20 ft.

The erection of the forms consists of placing a steel channel rib, which fits the section of the tunnel, and connecting it with the rib last placed by means of steel plates 4 ft. long and 3 ft. high. These plates are built up solid to the top of the arch, as the concrete, placed by the compressed air method, permits the forms to be built complete before concreting begins. The steel ribs of the forms are 4 ft. apart, corresponding to the length of the plates, and from one to five sections are set up at a time, depending upon the character of the rock. A view of the forms in place is shown in Fig. 3.

The mixing and placing outfit consists of a pneumatic concrete mixer and conveyor mounted upon a 40 ft. flat car, equipped with bins holding 26 cu. yds. of material. The car is shown in Fig. 1. The cement is stored in bags under one of the bins and discharges toward the center of the car through chutes into a measuring hopper. This measuring hopper is lifted and tilted automatically to discharge into the mixer. The lifting device consists merely of a 6 in. air cylinder. The 8 in. delivery pipe leads from the mixer under the car and vertically up at the end to crown of the arch, where a 90° elbow enters through the bulkhead of the form, as shown in Fig. 2. Air is supplied from a compressor at the mouth of the tunnel through a 4 in. main laid on brackets fastened to the timber posts and a connection is afforded from this main to the air receiver on the car by means of a hose. This car was completely described in Engineering and Contracting of March 18, 1914.

The proper air compressor capacity to run a portable outfit of this kind is about 300 cu. ft. of free air per minute, compressed to from 80 to 100 lbs. In beginning the work, however, a compressor of 134 cu. ft. capacity was furnished by the railway from one of its yards, as this was the only machine available at that time. This compressor was operated by a gasoline engine and was found to be badly worn and inefficient. The amount of air furnished by the compressor was approximately 80 cu. ft. per minute. This, of course, effected a reduction in the output of the mixer and conveyor and is mentioned because it applies to the data given below.

The car is taken into the tunnel by means of the dinky engine and spotted at a point next to the forms. The upper elbow of the delivery pipe was previously suspended in place so that when the car is spotted the upper pipe is bolted to the pipe on the car, which comes directly under it. The air connection is then made and concreting is immediately begun.

The first work done by the outfit, which includes the lining of the first 20 ft. of forms, required five carloads of concrete material to fill the form. The entire work of tearing down timbers required 128 men hours; the total time erecting forms was 229½ men hours; the total time loading gravel and cement onto the car required 140 men hours; the total time required for mixing and placing concrete in the forms was 204 men hours; the total amount placed was 132 cu. yds. From these figures it is seen that the number of men hours required for the various items per cubic yard of concrete were as follows:

	Men hrs. per cu. yd.
Tearing down and clearing timbers	.97
Erecting forms	1.74
Mixing and placing concrete.............	1.5
Loading gravel and cement............	1.06

The delays noted on this form were 3½ hours on account of blowing off an 8-in. nipple and the replacing of same, and 2 hours' delay on account of a derailment of the derrick car.

The substitution of a clamshell bucket for the wooden loading skip will, when available, cut the cost of loading the car to about ⅓ men hours, or say 10 cts. per cubic yard, and the substitution of a 300 cu. ft. compressor will make it possible to mix and place one batch per minute. The number of batches on the car varies between 112 and 118. The time required at present to unload one car, including time of transporting and for connecting and disconnecting pipe, is about 210 to 240 minutes.

ACKNOWLEDGMENT.

The concrete mixer and conveyor used for the work was furnished and the portable outfit was designed by the Concrete Mixing & Placing Co., 123 W. Madison St., Chicago, Ill., to whom we are indebted for the information here given.

BRIDGES

Construction and Cost Data on Some Highway Bridge Work in Alaska.

The construction of highway bridges in Alaska presents many interesting features, and it is somewhat difficult for engineers and contractors to appreciate the uncertainties and difficulties of bridge construction in that country. The following construction and cost data on a highway bridge in Alaska were abstracted from a paper by Maj. F. A. Pope, in Professional Memoirs for April-May, 1914. The article also describes conditions and some future work which is soon to be started. The work described is on the road from Valdez to Fairbanks. The Taslina Bridge, which is described, was built in February-May, 1906, and as it was the first large piece of work done by the board it is a good example of work done under frontier conditions. In Alaska most of the large bridge work is done in winter, as the thick ice takes the place of falsework.

THE TASLINA BRIDGE.

The Taslina River crosses the Valdez-Fairbanks and Eagle trail 110 miles from Valdez. It is a very rapid stream, from 300 to 400 ft. wide, and, except in the early spring, unfordable. Until this season a frail ferry has been in operation during the summer, but several persons have been drowned, chiefly through accidents to the ferry. The rapid fluctuations of the stream make it problematical whether a substantial ferry could be maintained. The drift carried by the stream in floods would render it impossible to operate a ferry during such periods.

For these reasons the board determined on a bridge across this stream. The necessary funds became available in January. The construction party left Valdez February 16 and completed the work May 12. At that season the horses used on the work could not be taken out over the Coast Mountains, two men remaining to care for them who were also employed in laying guard rails, etc., and later in packing out the bedding, equipment, etc.

The bridge consists of two Howe truss spans of 108 ft., two king-post spans of 50 ft., and approaches, to a total length of 450 ft. The clear width is 10 ft.

The main trusses rest on pile bents, which are protected by rock-filled crib piers. These piers are 40 ft. long and 12½ ft. wide at the base and 30 ft. long and 10 ft. wide at the top, with pointed ends. The outer ends of the small trusses rest on similar smaller piers. The trusses are all constructed of hewn timber, the lower chords being built up of from four to six pieces, bolted and keyed together. The tension members of the king-post trusses are of wire rope from the old ferry cable.

The expense of construction was largely increased by the indifferent character of the timber available. By culling the best trees for a radius of 6 miles from the bridge, sufficient was found, but few pieces could be obtained larger than 10 by 10 ins., 20 ft. long. As forage costs 18 cts. per pound, the cost of getting out the timber was very high.

The freighting of the greater part of the material from Valdez to the site of the bridge was contracted for at 12¼ cts. per pound, but as this rate was lower than was profitable under adverse conditions, and as these conditions occurred, the contractor threw up the contract; and while the agreement had been drawn so as to prevent financial loss to the government, it was only by a considerable effort that the freight was gotten to the bridge before the snow disappeared. This statement is inserted to show the peculiar disadvantages of contract work of any kind let to the lowest bidder in a country in which the ordinary facilities for executing work are as lacking as they are in Alaska.

Detailed records of the distribution of labor were kept, but lack of time prevents the preparation of their summary for the report. The total cost of the bridge, including plant, was $19,434.43. The itemized costs of this bridge are given in Table I.

It should be remembered that all supplies, material, and equipment, including the engine boiler, came from Seattle, 1,700 miles by sea to Valdez, and were then taken on single-horse sleds over a poor trail crossing the Coast Mountains, which extend far above the timber line, and finally delivered at Taslina, 110 miles from Valdez.

The experience with this structure indicates that, in view of the small size of the available timber, the spans of wooden bridges constructed in the interior should be limited to about 80 ft.

OTHER WORK, CONSTRUCTION CREWS AND DATA.

As an example of the character of the streams which must be bridged it may be mentioned that one small stream has washed out two bridges, the second one being supposed to meet almost any possible flood. Glacier streams are, however, subject to sudden and remarkable floods, and their beds are readily shifted. A third bridge has been constructed with a trestle 748 ft. long with a single king-post truss 30 ft. long. The width of the stream is about 30 ft. This bridge cost $7,798.39, of which nearly 50 per cent was for forage and freight on supplies. The cost of the whole section was about $1,790 per mile.

All streams are bridged except the Gulkana, Tanana and Salchaket Rivers and Piledriver Slough, all of which have trail bridges or rope ferries. A bridge consisting of two 150-ft. Howe truss spans is to be built across the Gulkana River this year. This is a difficult stream, and a suspension bridge was considered at one time but found too expensive. An experimental rock-filled crib put in the river by the present engineer officer, First Lieut. G. E. Edgerton, Corps of Engineers, has stood the spring ice run, so it has been decided to build a truss bridge.

The standard type of truss bridge on this road, as elsewhere, was that of the Taslina Bridge, except that the length of span was reduced to 75 ft. on account of the timber being short and not very strong. The standard required for the road was a good earthen highway, properly crowned, ditched and graded at least 10 ft. wide, the clearing being 16 ft. wide. The Fairbanks end was built under the immediate supervision of the superintendent there. Supplies were sent up the Tanana

River by boat, sometimes by sled, to different points along the river, and then carried where needed by wagon or pack-horse, depending on the condition of the roads.

Supplies for the Valdez end, except near Valdez, had to be carried by sled on account of the prohibitive cost of summer transportation. This required careful planning in advance. From the amount of work desired to be done the next year the number of crews needed was decided in the fall before. The

TABLE I—DISTRIBUTION OF EXPENSES, TASLINA BRIDGE.

Plant.			
Tools, at Valdez	$ 284.90		
Pile driver, at Valdez	1,[illegible]		
Horses	1,597.00		
Sleds and harness	574.47		
Camp outfit	247.10		
	$3,784.22		
Deduct horses charged Route 6	575.00		
			$3,209.22
Transportation Material and Crew to Bridge:			
Own teams, labor and forage (7¼ tons at 11½ cts. per lb.)	$1,735.92		
Hired transportation (10½ tons at 12¼ cts. to 25 cts. per lb.)	2,936.08		
Wages of crew en route	892.00		
			$5,564.00
Getting Out Timber (15,000 lin. ft. at 19 cts.):			
Felling	$ 769.15		
Swamping	173.60		
Hauling, including forage of teams	1,911.12		
	$2,853.87		
Charged to piling	$ 301.92		
Charged to cribs	603.83		
Charged to trusses	1,234.64		
Charged to decking	713.48		
		$2,853.87	
Substructure:			
Cutting out ice for piers (five)	$ 267.35		
Thawing	168.65		
Piling (150 piles at $7.82):			
Setting up driver, making frame, repairs, taking down and storing	$ 220.28		
Proportional cost timber	301.92		
Driving piles	406.29		
Cutting fuel	44.30		
Night fireman	200.30		
		$1,173.09	
Cribs ([illegible] cu. yds. at $[illegible])			
Material, [illegible]	$ [illegible]		
Proportional cost timber	603.83		
Framing	[illegible]		
Filling, including forage of teams	890.67		
		$1,975.21	
Riprapping		[illegible]	
			$3,721.03
Trusses (two 108 ft. [illegible])			
Materials, iron, at Valdez	$ [illegible]		
Proportional cost timber	1,234.64		
[illegible] (5,280 lin. ft. at 16c)	851.21		
[illegible]	[illegible]		
[illegible]	[illegible]		
False work	41.90		
			3,320.57
Decking (4,500 sq. ft. [illegible])			
Proportional cost timber	$713.48		
[illegible]	[illegible]		
[illegible]	[illegible]		
Wheel guards	[illegible]		
			1,298.03
[illegible]			
[illegible]	$ [illegible]		
[illegible]	[illegible]		
[illegible]	[illegible]		
[illegible]	[illegible]		
			[illegible]
Total			$[illegible]

following were the standard bridge crews and equipment:

Personnel

1 [illegible] 1 [illegible]
1 [illegible] [illegible]
1 [illegible] [illegible]
1 teamster. 10 laborers.

Equipment.

[illegible]
Pile-driver.
[illegible]
Carpenter tools, axes, ship augers, etc.
Camp outfit
Sleds, depending upon condition.

The [illegible] ried through a hose from the extra boiler.

The number of men and horses needed and the supplies and equipment for each crew were worked out in detail, and the supply point for each crew's summer work determined. The supplies and material, except bridge material and occasionally tools, were bought late in the fall by the disbursing officer after advertising in Valdez and Seattle. Bridge equipment and occasionally tools were bought by the engineer officer in Seattle at the same time. Also such sleds for transportation and horses as might be needed during the winter. These supplies were usually delivered at Valdez early in January. The latter part of January, when the days lengthened so that sledding could be done to advantage, the sledding crews were organized. Transportation was with double-ender, single-horse sleds. Bob sleds would ordinarily have been cheaper and easier on the trail, but on account of the great snowfall and drifts over Thompson Pass and consequent expense and uncertainty of keeping the trail open, double-ender sleds had to be used. Also over the Delta Summit, on account of sudden storms and danger of open water, it was not safe to attempt to use bob sleds. To have had enough sleds and horses to carry all equipment at once would have been too expensive and a system of relays one day apart was therefore used. The sleds would be loaded and taken as far as they could for a day and camp established and the sleds unloaded, then more trips back and forth until all supplies were hauled to this point; then another camp one day further on would be established in the same way, and so on. This was found much cheaper than to attempt to haul things straight through. Freight would be stored in various caches along the road. A cache might be defined as a depot where supplies are stored for safe keeping. For safety, these caches were put either near a telegraph station or a road-house; the keys were delivered to the telegraph operator (a signal corps soldier), or to the road-house keeper. When the freighting was completed, bridge crews would be left to do their work before spring and the remainder of the men, with horses and sleds, taken back to Valdez. Owing to the great cost of transportation only the very best of all kinds of supplies were bought. Grain was double-sacked to prevent loss; hay was double-compressed, to save space, and sacked—triple-compressed hay being found to break up and become powdery. The cost of transportation, of course, varied, but with sled roads in good condition it was found that to take supplies to Millers Roadhouse, about 250 miles, cost about 13½ cts. per pound. The cost increased greatly with the distance, as the supplies necessary for the sledding crews naturally reduced the amount of freight hauled.

In the spring any extra horses needed were bought, usually in Seattle by the engineer officer or superintendent, and the road crews were organized. When there were not sufficient laborers in Valdez it was necessary to get them in Seattle, but this was seldom done, as the Alaskan laborer, being familiar with conditions, was much more efficient.

Thompson Pass was not usually open for summer transportation before about the middle of June. The crews would then be sent in to the different places where they were to work. The bridge crews left inside having completed their work would be started on road work, it being possible to start on this work before the crews could get over the pass. During the break-up, that is, when snow was melting, the pass is practically impassable. The crews usually returned early in October.

Design, Construction and Detailed Labor Costs of the Substructure of the Double-Leaf Trunnion Bascule Bridge at Chicago Ave., Chicago, Ill.

(Staff Article.)

I. Design.

The new Chicago Ave. Bridge, which spans the Chicago River at Chicago Ave., Chicago, is a double-leaf trunnion bascule structure with a clear span of 161 ft. 3 ins. and a length, center to center of trunnions, of 188 ft. 9 ins. The bridge has a clear roadway of 36 ft. and two 12-ft. sidewalks. In this issue we shall describe and illustrate the principal design features of the substructure of this bridge as an introduction to an exceptionally complete contributed article which will be published in a

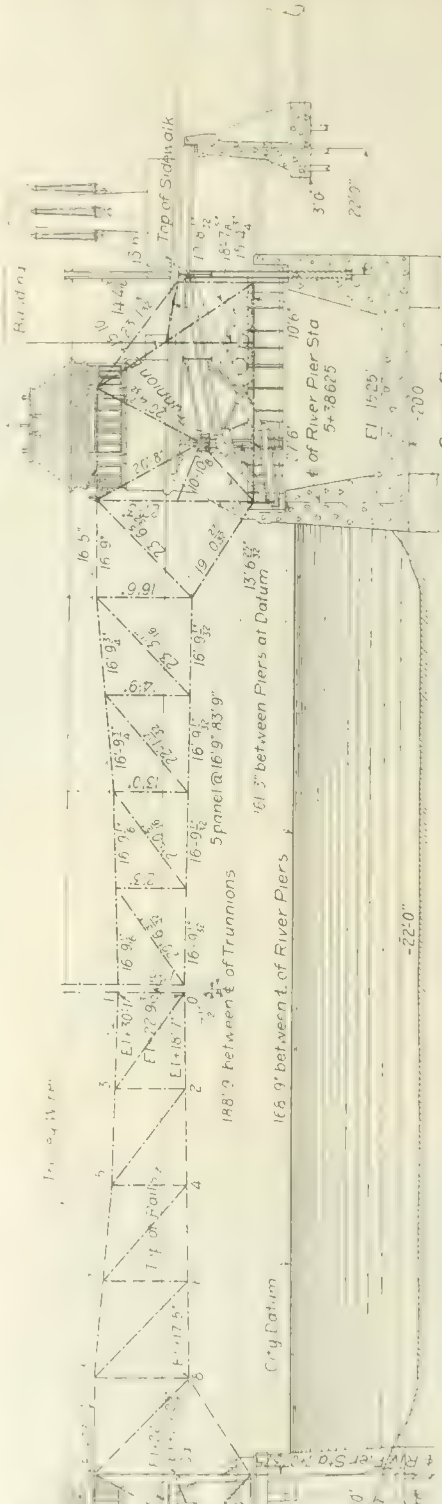


Fig. 1. Side Elevation and Longitudinal Section of the Chicago Ave. Trunnion Bascule Bridge, Chicago, Ill., Showing General Design Features.

later issue on the methods and costs of constructing the bridge substructure.

An outline of the structure indicating some of the design features is shown in Fig. 1. The trusses are spaced 39 ft. 6 ins. on centers, each leaf having a depth at the free end of 11 ft. 6 ins. and a depth over the river piers of 27 ft. 3 ins., center to center of chords. The sidewalks are carried outside of the trusses on cantilever brackets, which extend 10 ft. 3 ins. beyond the center lines of the chords. The four concrete piers which support each leaf of the bridge are carried to bed-rock, which lies about 83 ft. below city datum. The bottom of the counterweight pits is at elevation —20.0, which is about 2 ft. above the bed of the river at the site. The foundation work was somewhat complicated, as it was necessary to design and construct it so as to provide for a 6-ft. water pipe tunnel and for a future double-bore subway tunnel, the locations of which are shown in Fig. 3.

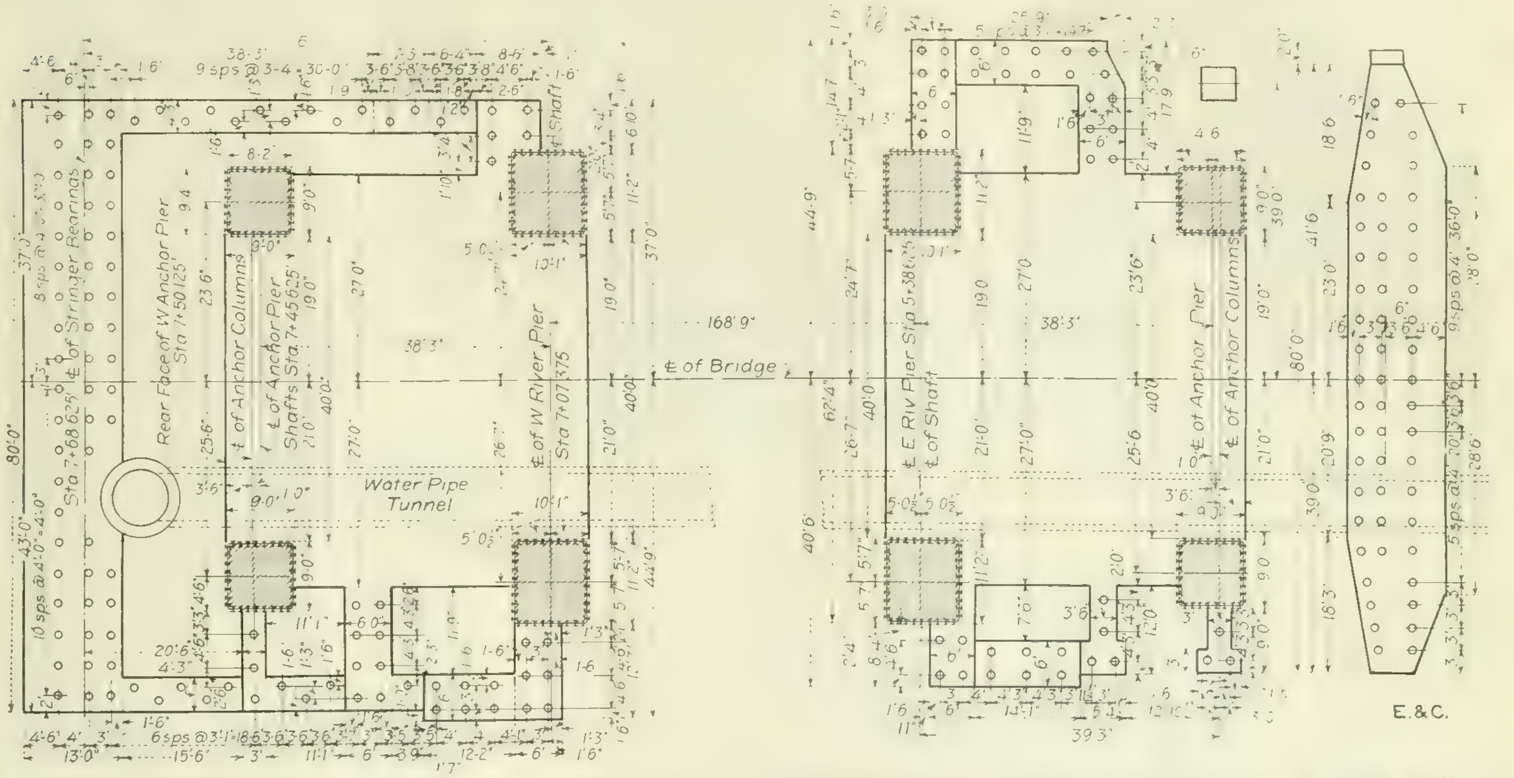


Fig. 2. Foundation Plan of the Chicago Ave. Trunnion Bascule Bridge, Showing General Dimensions and Location of Piers, Walls and Abutments.

SPECIFICATIONS.

Concrete.—The concrete used in the construction of the substructure was a 1:3:5 mix. The fine aggregate consisted of torpedo sand, it being required that the sand should pass a screen having ¼-in. diameter holes, and that not more than 6 per cent of it should pass a sieve having 100 meshes per linear inch. The coarse aggregate was required to be that retained on a ¼-in. mesh screen and passing a screen having 1½-in. diameter holes.

Steel Sheeting.—The prismatic portions of the pier shafts were required to be enclosed with steel sheet piling weighing not less than 35 lbs. per square foot, the sheeting extending from elevation —20.5 to elevation —45.0. This sheeting was to be left in place after the piers were completed.

Facing and Waterproofing Courses.—In order to waterproof the pits and to prevent disintegration of the concrete surfaces, they were required to be finished as follows: A horizontal layer of cement mortar at least 6 ins. thick was to be placed in the tail pit floors, as noted on the plans. The remainder of the floor surface in the tail pits was to be laid continuously until completed, the top of the floors being finished with a 3-in. layer of cement mortar. The faces of all piers and tail pits were also required to be finished with a layer of cement mortar, the thicknesses of this layer being 6 ins. for the outside of piers and tail pit walls below elevation + 2.0 and 2 ins. above this level. The inside faces of the tail pit walls were to be finished with a 2-in. layer of cement mortar. These waterproofing and finishing layers were placed inside of wooden forms and boxing, and were carried up with the concrete.

DESIGN FEATURES.

Figure 1 illustrates some of general features of the foundation design. The weight of each movable leaf of the bridge and its operating machinery is carried on four concrete piers resting on bed-rock. The abutments of the short approach spans and the enclosing walls of the structure are supported on pile foundations.

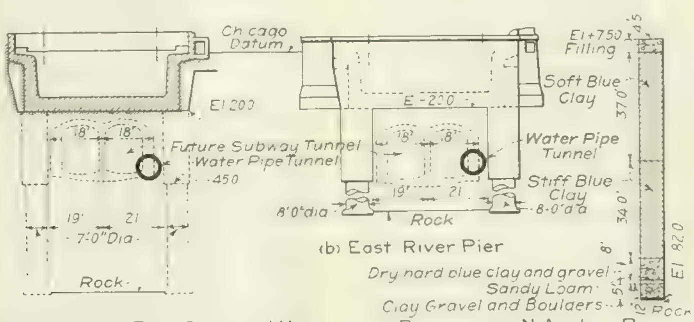


Fig. 3. (a) Cross Section of Pit and Elevation of East Anchor Pier of Chicago Ave. Bridge. (b) Front Elevation of East River Pier. Note Locations of Future Subway Tunnel and Water Pipe Tunnel.

Figure 2 shows a foundation plan of the bridge and gives the size and arrangement of the walls and piers. (During construction the dimensions of the upper portions of some of the piers were changed slightly, although the cross-sectional areas were not essentially changed.) The west anchor piers are spaced 49 ft. on centers, while the west river piers are spaced 51 ft. 2 ins. on centers, the center to center distance of the river and anchor piers being 38 ft. 3 ins. The spacing of the east river and anchor piers is the same as for the west piers.

Figure 3 (a) shows a cross section of the east pit and an elevation of the east anchor piers. This drawing also shows the location of the future subway tunnel and of the water pipe tunnel. It will be noted that below elevation —45.0 the piers are circular, the diameter being 7 ft. A boring taken near the north anchor pier (see Fig. 3, c) showed the subsoil to consist of the following strata:

Elevation + 7.5 to elevation + 3.0, filled in material.
Elevation + 3.0 to elevation — 34.0, soft blue clay.
Elevation — 34.0 to elevation — 68.0, stiff blue clay.
Elevation — 68.0 to elevation — 76.0, dry, hard, blue clay and gravel.
Elevation — 76.0 to elevation — 77.0, sand and loam.
Elevation — 77.0 to elevation — 82.0, clay, gravel and boulders.
Elevation — 82.0, bed-rock.

The elevation of bed-rock at various points of the site varies from about — 80.5 to — 83.0.

Figure 3 (b) shows a front elevation of the east river piers, the diameter of the pier shafts below elevation —45.0 being 8 ft. This drawing also shows the locations of the future subway tunnel and of the water pipe tunnel.

Figure 4 shows details of design of the west abutment and the west piers and pit. These drawings show the arrangement of the reinforcing bars and the reinforcing trusses and anchorage used. The east abutment, piers and pit are of similar construction. Figure 4 (a) shows a cross section of the west abutment and gives the principal dimensions, together with the location of the foundation piles. This abutment has a total height of 22 ft. 0¼ in., a length of 80 ft., and a width of footing of 13 ft. The abutment is of the counterfort type, the thickness of the counterforts being 2 ft. and their spacing 13 ft. on centers. Figure 4 (b) shows a longitudinal section of the west piers and pit and gives the position of the reinforcing trusses and the anchorage for the superstructure. Figure 4 (c) shows a cross section of the pit, and indicates the position of the reinforcing trusses, machinery girders, etc. The heavy girder in connection with Fig. 1 will give a clear conception of the relation between the substructure, the superstructure, and the operating machinery. The structural steelwork shown in Fig. 4 was furnished by the con-

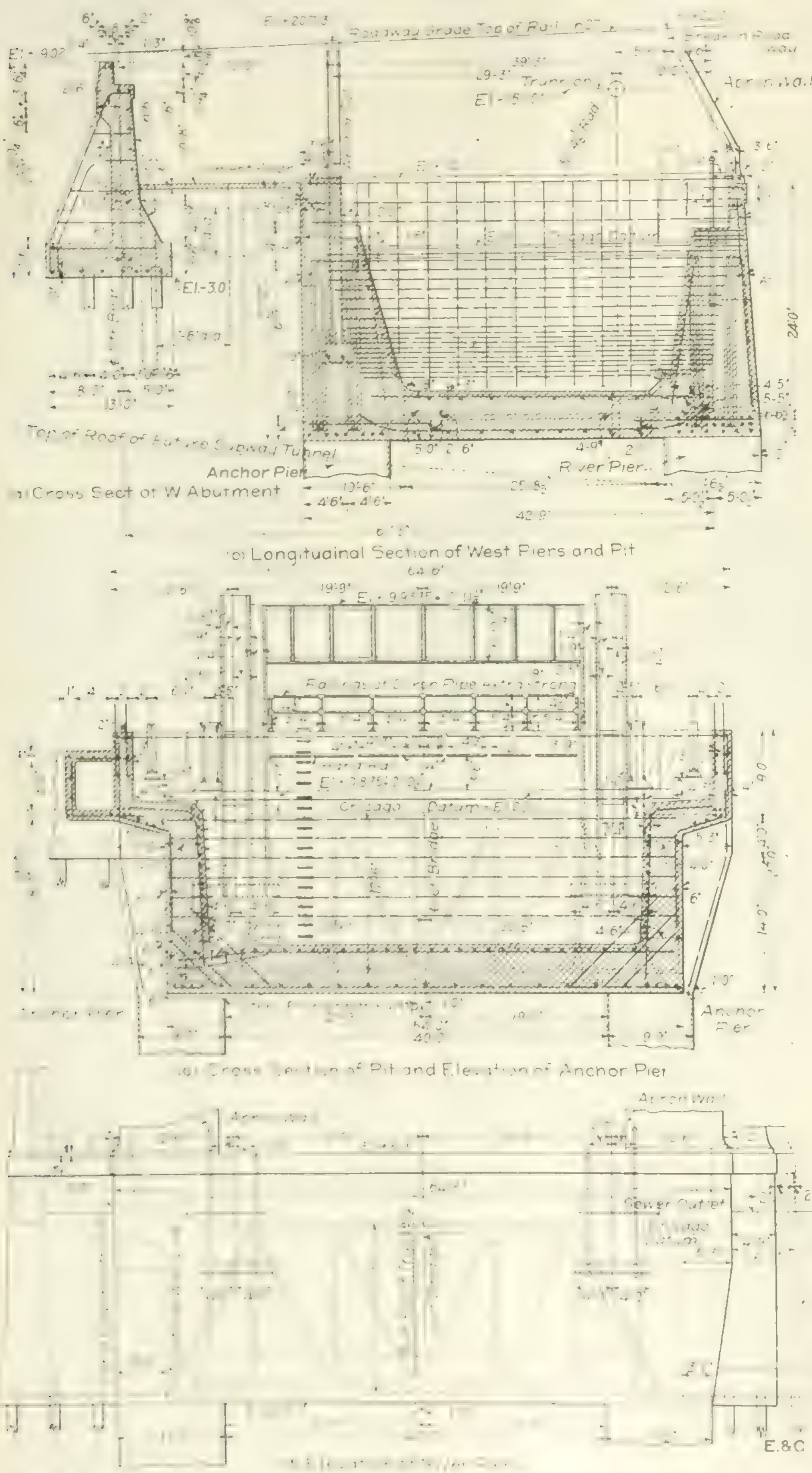


Fig. 4. Cross Section of West Abutment, Longitudinal and Cross Sections of West Piers and Pit, and Elevations of Piers of Chicago Ave. Trunnion Bascule Bridge, Showing Type of Construction and Reinforcement.

"A I" is the anchor column [illegible], which carries the stringers of the approach span. Figure 4 (d) shows an elevation of the west river piers and indicates the system of reinforcement used. A study of these drawings

tractor for the superstructure, but it was set by the contractor for the substructure.

The design of this bridge was in charge of John Ericson, city engineer of Chicago, Thos. Pehlfeldt, bridge engineer, and Alexander von Babo, designing engineer.

General Specifications for Bridge Work of Illinois State Highway Commission.

There is at present a tendency on the part of writers of highway bridge specifications to show more originality in the preparation of such specifications and to depart somewhat from the stereotyped form so long followed. Marked changes have been necessary due to heavier loads and to the advisability of building permanent types of bridges. The following extracts from the "General Specifications for Bridge Work" now in use by Illinois State Highway Commission are of interest. These specifications govern all bridge work which requires the approval of a County Superintendent of Highways or of the State Highway Commission. They are intended for bridges located outside of the corporate limits of cities and villages and which are expected to carry heavy highway traffic only.

Types of Bridges.

CONCRETE BRIDGES.

Culverts.—For culverts requiring an area of waterway of 12 sq. ft. or less; plain or reinforced concrete arches, reinforced concrete boxes, reinforced concrete pipe or double strength cast iron pipe.

Small Bridges.—For culverts having a waterway of more than 12 sq. ft., and for bridges having spans up to 30 ft.; reinforced concrete slabs, plain or reinforced concrete arches.

Bridges.—For bridges having spans of 30 ft. to 65 ft.; reinforced concrete through or deck girders, plain or reinforced concrete arches.

For bridges having spans greater than 65 ft.; plain or reinforced concrete arches.

Arches.—Plans for plain or reinforced concrete arches will not be approved unless founded on solid rock or unusually firm hardpan, except for paved culverts and very small paved bridges.

STEEL BRIDGES.

Use Of.—In general, plans for steel bridges will not be approved except for structures spanning drainage ditches, navigable channels, and for locations where the cost of concrete structures would be prohibitive.

Bridges Over Drainage Ditches. — For bridges spanning drainage ditches where it is anticipated that it will be necessary to remove the bridge superstructure periodically to permit the passage of a dredge, those having pin or bolted field connections shall be used.

Types for Various Spans.—For bridges having spans of 12 ft. to 45 ft.; steel I-beams.

For bridges having spans of 30 ft. to 100 ft.; plate girders or riveted pony trusses.

For bridges having spans of 90 ft. to 160 ft.; riveted trusses with parallel chords.

For bridges having spans of 160 ft. or more; riveted or pin-connected trusses with parallel or inclined upper chords.

ROADWAYS.

In general the clear width of roadways for bridges shall be not less than:

On Designated State Aid Routes.—Bridges up to and including 25 ft. in length, 24 ft.

Bridges 25 ft. and up to and including 60 ft. in length, 20 ft.

Bridges over 60 ft. in length, 18 ft.

Length is understood to mean the distance face to face of abutments.

Bridges located on designated state aid routes at a distance of more than 10 miles, by direct route from any city having a population of 5,000 or over and at a distance of more than 3 miles from any city having a population of less than 5,000 may provide roadways 20 per cent narrower than above prescribed, with a minimum of 18 ft.

No bridge shall, however, when located on a designated state aid route within 50 miles of any city having a population of over 200,000 have a clear roadway of less than 24 ft.

On Principally Traveled Roads.—On principally traveled roads, other than designated state aid routes:

Bridges and culverts of 8 ft. or less in length, 24 ft.

Bridges over 8 ft. and up to and including 60 ft. in length, 18 ft.

Bridges over 60 ft. in length, 16 ft.

No such bridge or culvert shall, however, when located within 3 miles by direct route of any city or village having a population of 10,000 or over, have a clear width of roadway of less than 24 ft. for bridges having a length of 25 ft. or less; nor less than 18 ft. for bridges having a length of more than 25 ft.

On Secondary Roads.—On lesser traveled or secondary roads:

Bridges and culverts of 8 ft. or less in length, 20 ft.

Bridges over 8 ft. in length, 16 ft.

Culverts Under Fills.—When culverts are built under fills, the length of the barrel of the culvert shall be such as to permit of side slopes of 2 horizontal to 1 vertical, and in addition, a top width of fill not less than as follows:

On designated state aid routes, 25 ft.

On principally traveled roads, 25 ft.

On secondary roads, 20 ft.

Loadings.

DEAD LOADS.

Concrete Floor and Wearing Surface.—All members and details of steel bridges shall de designed to carry a concrete floor slab, assumed to weigh not less than 48 lbs. per square foot of roadway surface and a pavement or wearing surface assumed to weigh not less than 50 lbs. per square foot.

Weight of Materials.—In computing the dead load, the following unit weights shall be used:

	Lbs per cu. ft.
Steel	490
Concrete	144
Earth fill	100
Gravel, tamped	120
Macadam and gravel rolled	140
Brick	150
Yellow pine and Douglas fir	42
Oak	54
Creosoted pine or fir	60

LIVE LOADS FOR CONCRETE BRIDGES.

Concrete bridge structures shall be designed to sustain in addition to the dead load, a uniform live load of 125 lbs. per square foot of roadway surface, or the concentrated live load indicated in Fig. 1. In all cases the load used and its position on the bridge shall be that which will produce the greatest stress in that part of the structure under consideration.

LIVE LOADS FOR STEEL BRIDGES.

Concentrated Load.—All floor systems shall be designed to sustain, in addition to the dead load, a concentrated live load of not less than 15 tons, which shall be considered as supported on two axles, spaced 10 ft. apart, the rear axle to carry 10 tons and the forward axle 5 tons.

Concrete Floor Slabs.—Concrete floor slabs shall be designed to carry, in addition to the dead load, a uniform live load of 1,440 lbs. per square foot of slab.

Longitudinal Stringers.—Stringers arranged parallel to the axis of the roadway shall be spaced not more than 2½ ft. apart and each stringer shall be designed to carry not less than 20 per cent of the rear axle load indicated above, considered as concentrated at the center of the stringer span.

Transverse Stringers. — In plate girder bridges when the stringers are arranged at right angles to the axis of the roadway, and are spaced not more than 2½ ft. apart, each stringer shall be designed to carry not less than 40 per cent of the rear axle load indicated above, considered as uniformly distributed over the middle 10 ft. of the stringer.

Uniform Load on Stringers.—Stringers of spans under 50 ft. in length shall be designed to carry a uniform live load of not less than 125 lbs. per square foot of roadway surface, and of spans over 50 ft. in length, 100 lbs. per square foot of roadway surface when such uniform live load would produce stresses in excess of those produced by the above concentrated load.

Floorbeams.—Floorbeams shall be designed to carry, in addition to the dead load, the full concentrated live load specified, arranged in such position as to produce maximum stresses in the floorbeams, or for spans of 50 ft. or less a uniform live load of not less than 125 lbs. per square foot of roadway surface, and for spans of ~reater length than 50 ft. a uniform live load of not less than 100 lbs. per square foot of roadway surface, provided such uniform live load would produce greater stresses than the concentrated load described.

Loads for Trusses.—In designing all trusses and plate girders, the following uniform live loads, arranged in such position as to produce the maximum stress in the member or detail under consideration, shall be assumed:

For spans of 50 ft. or less, a uniform live load of 125 lbs. per square foot of roadway surface.

For spans over 50 ft., up to and including 150 ft., 100 lbs. per square foot of roadway surface.

For spans longer than 150 ft., 85 lbs. per square foot of roadway surface.

Wind Load.—To provide for wind strains and vibrations, all lateral bracing shall be designed for a wind load of 25 lbs. per square foot of the vertical projection of the exposed surface of both trusses and the floor system. No such wind load shall, however, be figured at less than 300 lbs. per lineal foot of bridge for the loaded chord nor less than 150 lbs. per lineal foot of bridge for the unloaded chord.

No permanent lateral bracing need be provided in the plane of the loaded chord, provided a solid concrete floor, reinforced in both directions, is immediately to be placed on the structure. In this case, however, a

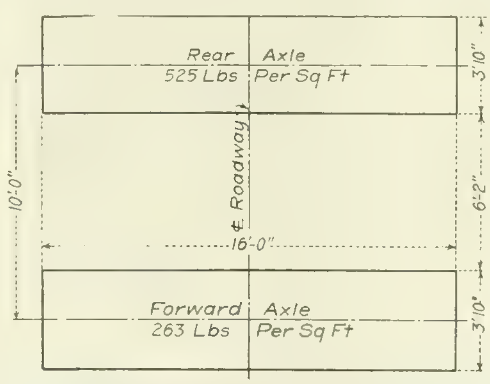


Fig. 1. Diagram Showing Concentrated Live Load Used in the Design of Highway Bridges by Illinois State Highway Commission.

temporary lateral system consisting of adjustable rods must be used to hold the chord in line while the floor is being placed. After the floor is completed, such temporary laterals must be removed.

Allowable Stresses.

The modulus of elasticity of steel shall be taken as 30,000,000 lbs. per square inch.

The modulus of elasticity of concrete shall be taken as 2,000,000 lbs. per square inch.

The coefficient of expansion of concrete, plain or reinforced, shall be taken as 0.000,006.

The total combined stress in the various parts of any bridge structure shall not exceed the following:

CONCRETE ABUTMENTS AND PIERS.

	Lbs. per sq. in.
Tension in concrete	00
Compression in Class A concrete	700
Compression in Class B concrete	450
Tension in reinforcing steel	16,000
Bond between reinforcing steel and concrete (surface area)	80

REINFORCED CONCRETE SLAB SUPERSTRUCTURE.

Restrained Slabs.—Slabs restrained from free expansion or contraction due to temperature changes, by friction on abutments or otherwise:

	Lbs. per sq. in.
Tension in concrete	00
Compression in concrete (Class A)	800
Shear (diagonal tension) in concrete when no shear reinforcement is provided	40
Shear (diagonal tension) in concrete when reinforcement is provided for shear in excess of 40 lbs. per sq. in.	120
Bond between reinforcing steel and concrete (surface area)	80
Tension in reinforcing steel	12,000

Simple slab bridges when the slab is merely separated from the foundation walls by means of a paper, felt, tar or asphalt joint will not be considered as free to expand or contract.

REINFORCED CONCRETE GIRDERS.

Provision for Free Expansion.—Reinforced concrete girders shall in every case be provided with rocker or other efficient bearings of nominal friction and all parts of the superstructure shall be completely separated from the foundations by an open joint or a joint not less than ¼-in. thick, filled with bituminous felt.

	Lbs. per sq. in.
Tension in concrete	00
Compression in concrete (Class A)	1,000
Shear (diagonal tension) in concrete when no shear reinforcement is provided	40
Shear (diagonal tension) when reinforcement is provided for shear in excess of 40 lbs. per sq. in.	120
Bond between reinforcing steel and concrete (surface area)	80
Tension in reinforcing steel	16,000
Bearing on expansion rockers (cast iron) resting on steel plates (d = diameter of rocker in inches.)	300d

PLAIN AND REINFORCED CONCRETE ARCHES.

Except for three-hinged arches, temperature stresses induced by a range of 40° either way from the normal shall be provided for in addition to dead and live load stresses.

	Lbs. per sq. in.
Tension in concrete	00
Compression in concrete (Class A)	800
Shear (diagonal tension) when no shear reinforcement is provided	40
Shear (diagonal tension) when reinforcement is provided for shear in excess of 40 lbs. per sq. in.	120
Bond between reinforcing steel and concrete (surface area)	80
Tension in reinforcing steel	16,000

RESTRAINED STRUCTURES.

The stresses specified for "Restrained Slabs" shall not be exceeded in the design of all reinforced concrete slabs, beams, girders and other parts of the bridge superstructure, when such part is restrained from free expansion and contraction by sliding friction on supports or otherwise.

Simple slab structures and T-beam superstructures resting directly on the foundations are of this class.

UNRESTRAINED STRUCTURES.

The stresses specified for "Reinforced Concrete Girders" shall not be exceeded in the design of all reinforced concrete girders, beams, slabs and other parts of the bridge superstructure, when such part may expand and contract practically without restraint by friction on abutments or otherwise.

Reinforced concrete girders resting at one end on rockers, floorbeams and floor slabs of through or deck girders are of this class.

STEEL SPANS.

	Lbs. per sq. in.
Tension.	
Medium steel and castings	16,000
Wrought iron	13,000
Compression.	
Compression but not to exceed 14,000 lbs. per sq. in.	16,000–70 l/r
l = unsupported length of member in inches.	
r = corresponding radius of gyration in inches.	
The greatest l/r shall not exceed 120 for main truss members nor 148 for lateral or other secondary members.	
Forked ends and extension plates	10,000–300 l/t
l = distance in inches from center of pin hole to first rivet beyond point where full section of the member begins.	
t = thickness of plates.	
Cast steel	16,000
Bending.	
Extreme fiber of rolled and built up sections and steel castings, tension and compression	16,000
Extreme fiber of pins	24,000
Shear.	
Shop rivets and pins	10,000
Bolts and field rivets	8,000
Webs of rolled or built up beams, gross section (average)	10,000
Bearing.	
Pins and shop rivets	20,000

Bolts and field rivets........................ 16,000
Expansion rollers 600 times diameter of roller in inches, per linear inch.

Reversing Stresses.

Connections of members carrying reversing stresses shall be proportioned for a stress found by adding ¾ of the lesser to the greater stress.

Wind Load, Chord Stresses.

The stresses in the truss members or trestle posts from the assumed wind forces need not be considered except as follows:

When the wind stresses in any member exceed 25 per cent of the maximum stresses due to the dead and live loads upon the member, the section shall be increased until the total stress per square inch will not exceed the combined dead and live load stress by more than 25 per cent.

When the wind stress alone or in combination with a possible temperature stress can neutralize or reverse the stresses in any member.

Concrete Proportions and Uses.

Unless otherwise specially indicated on the plans, there will be three classes of concrete known as Class X, Class A and Class B.

Class X concrete shall consist of 1 part cement, 2 parts sand and not to exceed 3½ parts stone, which will be retained on a ¼-in. screen and which will pass a 1-in. ring. At least 40 per cent of the stone shall be retained on a ½-in. screen.

Class A concrete shall consist of 1 part cement, 2½ parts sand and not to exceed 4 parts tsone which will be retained on a ¼-in. screen and which will pass a 1½-in. ring. At least 50 per cent of the stone shall be retained on a ½-in. screen.

Class B concrete shall consist of 1 part cement, 3 parts sand and not to exceed 5 parts rock broken to pieces which will be retained on a ¼-in. screen and which will pass a 2½-in. ring. At least 50 per cent of the stone shall be retained on a 1-in. screen.

Class X concrete shall be used for all parapet walls, bridge seats and floors of steel bridges.

Class A concrete shall be used for all reinforced concrete work and for all plain concrete measuring less than 10 ins. in thickness, except for the floors of steel bridges.

Class B concrete shall be used in all plain concrete abutments, piers and wing walls, and shall also be used elsewhere as may be provided for on the plans or by the written directors of the engineer.

Foundations.

Stone masonry and plain or reinforced concrete abutments, piers, and end walls for all bridges and culverts shall be designed in accordance with the best modern practice. So-called tube and leg foundations will not be considered.

Abutments to Be Self-Supporting.—Abutments, except for reinforced concrete slab bridges, shall be designed as self-supporting with the approach fills complete in place.

Drainage.—Adequate drainage for the backs of abutments and wings shall be provided by tile or other pipe drains running through the walls at the lowest elevation possible which will provide free outlets.

Batter.—The batter on the face of all plain concrete walls and piers shall be not less [illegible] and reinforced concrete abutments, wing and retaining walls shall be not less than one-third of the height of the wall.

Depth of Footings.—Footings shall be car[illegible]tance of 4 ft. unless rock or unusually compact hardpan is encountered at a lesser depth, except for spans under 12 ft. in length in which case a heavily reinforced floor may be used to distribute the load on the foundation.

[illegible] paved culverts and abutment walls shall be so designed as to distribute the loads over the full length and width of the main wall foundations.

The pressure on ordinary soils shall not exceed 1.5 tons per square foot average, for abutment wing or pier footings. Wing footings shall not be considered as taking any of the superstructure load.

Cofferdam.—Suitable and practically watertight cofferdams preferably of steel sheet piling or tongue-and-groove wood piling shall be used whenever water bearing strata are encountered above the elevation of the bottom of the excavation.

Pumping.—Pumping will not be permitted from the inside of foundations forms while concrete is being placed, and if necessary to prevent flooding, a seal of concrete shall be placed through a chute or by means of bottom-dump buckets, and allowed to set.

Placing Concrete Under Water.—Concrete shall not be placed in running water and shall only be placed in still water with suitable appliances and under the direction of the engineer.

Dimensions of Pits.—The inside dimensions of pits and cofferdams shall be sufficiently large to give easy access to all parts of foundation forms.

Reinforced Abutments.—In properly designed reinforced concrete abutments when the wing walls are located at an angle of 45° or more with the face of the abutment wall, advantage may be taken of the mutual support afforded by the main and wing walls when properly tied together by reinforcing steel.

All parts of such abutments shall be designed to resist a pressure, imposed upon the vertical projection of all walls, figured as that which would be caused by a fluid having the same depth as that of the earth fill and considered as weighing not less than 15 lbs. per cubic foot.

Retaining walls, reinforced concrete abutments and wing walls, when the wing walls make an angle of less than 45° with the face of the abutment, shall be designed to resist a pressure, imposed upon the vertical projection of such walls, figured as that which would be caused by a fluid having the same depth as that of the earth fill and weighing not less than 21 lbs. per cubic foot. In this case the width of footing shall not be less than 0.4 of the height of the wall.

In designing reinforced concrete abutments for slab bridges, it may be assumed that the slab of the superstructure supports the top of the main wall as regards the overturning effect of earth pressure.

Footings of reinforced concrete abutments, wing walls and retaining walls shall be so proportioned that the resultant of all forces, including the weight of concrete, weight of earth fill directly over the footing, weight of superstructure (on main wall only) and the horizontal equivalent fluid pressure, shall fall at or back of the forward edge of the middle third of the footing base.

EXTRA FOUNDATION WORK.

Should it be found necessary to carry the foundations to a depth exceeding 3 ft. below that shown on the plans, the contractor shall be paid, in addition to the contract prices stated in the proposal, an amount equal to the cost of completing that part of the foundation below said 3 ft. plus 15 per cent of said cost.

Cost.—Cost shall be considered as made up of the cost of materials plus the actual cost of the labor, but shall not include any charge for overhead, rental, depreciation of equipment, or other expense.

Maintaining Batter.—Should it be found necessary in the judgment of the engineer to increase or decrease the depth of the foundations from that shown on the plans, the thickness of the wall, where said wall joins the footing, shall be increased or decreased the same amount per foot as the main wall increases per foot of its height as shown on the plans.

General Features.

Pony Trusses.—The top chord of low trusses shall be securely braced at the panel points by means of knee-braces or solid ribbed vertical posts rigidly connected to the floorbeams. The outstanding legs of angles used for vertical posts shall not be less than 3 ins.

Clearance.—For all through bridges there shall be a clear head-room of not less than 15 ft. above the crown of the finished roadway.

Transverse Bracing.—Intermediate transverse frames shall be used at each panel of through spans having vertical truss members where the clearance will permit. Transverse frames shall be capable of transferring at least 0.7 of the wind load from the top to the bottom chord.

The top and bottom lateral systems shall, however, be designed to carry the entire wind load to the shoes in the usual manner.

Rigid Members.—Hip verticals and similar members, and preferably the two end panels of the bottom chords of pin-connected trusses up to 300-ft. spans, shall be rigid.

Reconstruction of Pontoon Bridge at Prairie du Chien, Wis.—The Chicago, Milwaukee & St. Paul Ry. has completed a 209-ft. pontoon bridge at Prairie du Chien, Wis., which is one of the few structures of this type now in use on railroads. This span is over the east channel of the Mississippi River, and is one of four spans in service across the two channels. The new span is designed for Cooper's E-50 loading. There is a fixed span at this point over a part of the channel adjacent to the pontoon span. The hull of the pontoon is 209 ft. 4 ins. long, 55 ft. wide and 6 ft. 3 ins. deep. It is constructed of creosoted material, and is sheathed with 4-in. planks. The deck of the pontoon is also of 4-in. planking, being provided with three 20x30-in. inspection holes and eight ventilating hatches of the same size. The bracing of the hull consists of 2-in. rods extending from the ditch to the bottom, and it is divided into six compartments by five 8-in. timber bulkheads. The track is carried on ten steel floorbeams spaced 12 ft. on centers and a 60-ft. approach girder at each end. In order to provide for the variation of water level (the greatest known variation between high and low water loads being 22.5 ft.) the floorbeams are supported on blocks and can be moved vertically between four guide posts at each end a distance of 18 ft. to maintain the tracks at a uniform elevation. These guide posts form the vertical posts of two of the four longitudinal trusses which extend along the outside of the pontoon for its full length, the opening of the trusses in each pair being 9 ft. 2 ins. and of the pairs, 19 ft. 2 ins. These trusses give stability to the structure. The floorbeams are provided with hooks at each end, to which are attached cables which in turn run over sheaves in the tops of the guide posts and then to the drum in the machinery boat. In adjusting the track level the floorbeams are raised one at a time, the blocking being inserted or removed as required. It requires about one hour to raise or lower the track. The 60-ft. approach girders project beyond the pontoon, the projecting ends resting on steel shoes and roller nests which in turn rest on pile abutments. An average of ten trains cross the span each day, the maximum permissible speed being ten miles an hour.

Dust Laying Methods in Memphis, Tenn. The Bulletin of the United Gas and Electric Corporation states that the dust of city streets is one of the most discomforting conditions of city life, and there are many cities not adequately equipped to keep their streets properly sprinkled. In Memphis the street railway company has for a number of years been under contract with the city to sprinkle such streets as the Memphis Street Railway Co.'s lines cover.

To take care of this work the Memphis company has in operation five sprinkling cars, four of which are of 4,000-gal. capacity and one of 3,000-gal. capacity. The four large sprinklers distribute water by air pressure and are able to efficiently cover streets up to 80 ft. in width. Very good service is being rendered by this equipment, and a total of 50 miles is covered twice each day. In the month just passed approximately 3,000 miles were covered by the sprinklers and nearly 14,500,000 gals. of water distributed.

ROADS AND STREETS

The Design and Construction of a Concrete Pavement in the Village of Glencoe, Ill.

(Staff Article.)

The following article describes the design features and details of construction used in connection with the improvement of a section 560 ft. long of Wentworth St. in the village of Glencoe, a suburb of Chicago, Ill. The methods of preparing plans and drawing the ordinance for this work are described and illustrated.

DESIGN FEATURES.

Drainage.—The soil on which the pavement was to be constructed was of a clayey nature and it was decided to use tile drains throughout the work. Vitrified clay pipe 6 ins. in diameter laid on a grade of not less than six-tenths per cent was adopted for this purpose. These drains led into brick catch basins, Fig. 2, placed at suitable intervals, into which the surface drainage from the road also flows. Two outfalls for the drainage system were provided, one at each end. The ends of the outfalls were protected with concrete end walls. Drain pipe were laid at least 2 ft. 6 ins. below the ground surface and in a bed of cinders. Cinders were also used for backfilling in all trenches over which pavement was placed and for a width of 6 ins. from the pipe to the ground surface in all other trenches. Figure 1 shows the usual location of drain pipe.

Brick catch basins, Fig. 2, were provided, the brick being laid in 1:2 Portland cement mortar. The iron curb on these catch basins is adjustable to any height of concrete curb, permitting their use with curbs of variable heights when variation in the curb heights is desirable to secure gutter grades that drain well.

Cross Section.—It was decided to pave but 19 ft. (including the curbs) of the central portion of the 60-ft. street. The pavement was of reinforced concrete, having armored expansion joints at 25-ft. intervals. The cross section, Fig. 1, was such that the curbs were combined with the pavement proper. A thickness of 8 ins. at the center and 6 ins. at the sides was used. A crown of about 2 ins. was provided, being at the rate of ¼ in. to the foot, the surface curve being an arc of a circle. The curb exposure was 3 ins. and its width 6 ins. Longitudinal reinforcement of wire fabric having a sectional area of .038 sq. in. for each foot of width, and transverse reinforcement of a sectional area of .049 sq. in. for each foot of length was required. Reinforcement lapped 2 ins. and, of course, did not cross expansion joints.

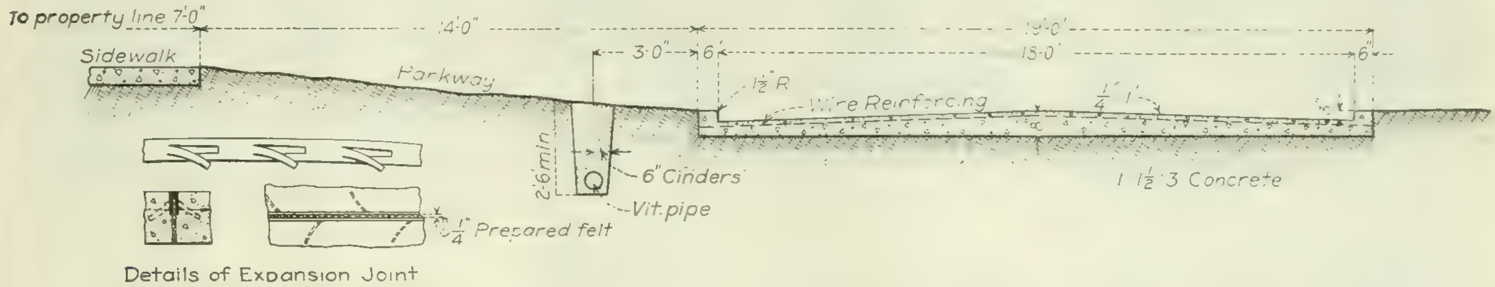


Fig. 1. Cross Section of Concrete Pavement in Glencoe, Illinois. Note Parkway and Arrangement for Drainage.

Width and Alignment.—The street paved is a residential street and carries no through traffic, hence the narrow width of paved way adopted. An interesting feature of the alignment is the curve at the end of the street, terminating in a paved circle having a radius of 20 ft. 6 ins. to the outside of the curb, the circle being tangent to one property line of the street.

PLANS AND ORDINANCE.

Plans prepared for this work were unique in that the ordinance, specifications and detailed plans were bound together, providing a complete record for filing purposes and convenient size plans for field use.

The ordinance was of the form required by the Illinois Local Improvement Act, which requires the accurate location of the improvement, being so worded that it read continuously with the specifications, obviating the necessity of duplicating descriptions. All terms used were clearly defined, for instance:

> The word parkway wherever used in this ordinance or upon drawings shall be construed to mean the spaces between the respective street lines and the back of the nearest curb; the term sidewalk line wherever used shall mean the line in the street 7 ft. from and parallel with the street line.
>
> When the diameter of a drain is mentioned or referred to it shall mean the internal diameter.
>
> Wherever the word sub-grade is used in this ordinance it shall mean that portion of the roadway which has been prepared or hereinafter specified upon which the pavement and curb is to be constructed.
>
> The term roadway shall mean the portion of the street where the pavement and curb is to be constructed.

Plans were prepared on sheets 8½x13 ins. in size to be bound with specifications and ordinance, typewritten on paper of that size. Both the plan and profile, Fig. 7, and construction details were placed on these sheets. The scale used for the plan was 1 in. to 100 ft. and for the profile 1 in. to 100 ft., horizontal, and 1 in. to 6 ft. vertical. Each sheet was complete in itself and was referenced by means of a file and order number.

Colored areas were freely used on the blue line prints of the plans. In coloring an ordinary colored Blaisdell paper pencil was used, a little gasoline being rubbed over the colored area with an artist's stump to fix the color and prevent its smearing or becoming erased.

The following abstract from the specifications for the grading of parkways is interesting:

> The parkways along both sides of the roadway between the curb and sidewalk shall be

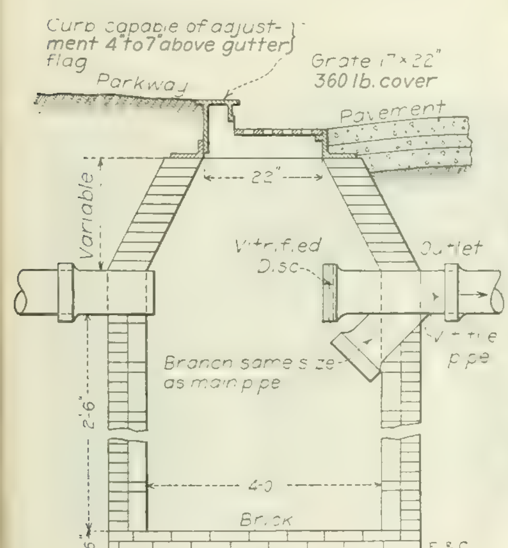


Fig. 2. Catch Basin with Adjustable Curb.

Fig. 3. Construction in Progress Showing Method of Handling Striking Board and Mixing Plant.

smoothed and graded between the curb and the sidewalk line so that the parkway shall at no point be below the top of the adjoining curb except, however [where specifically instructed at point where the roadway is on a decided fill.—Editors], the parkway shall be so graded that the earth at the back of the curb shall be even with the elevation of the top of the curb and the surface therefrom shall naturally and gradually slope and conform to the natural topography of the land.

Item.	Quantity.	Bid price.
Earth grading, cu. yd.........	3,000	.33
Gravel concrete pavement, including curb, sq. yd.........	4,000	$1.47
18-in. drain, 3 ft. deep, lin. ft.	150	.75
12-in. drain, 6 ft. deep, lin. ft.	1,190	.65
10-in. drain, 3 ft. deep, lin. ft.	240	.45
8-in. drain, 5 ft. deep, lin. ft.	340	.45
6-in. drain, 6 ft. deep, lin. ft.	1,730	.40
4-in. drain, 4 ft. deep, lin. ft.	380	.25
Brick catch basins, each.....	14	30.00
Concrete, cu. yd..............	2	7.00
Total amount of contract..................		$9,238

Fig. 4. Side Forms and Expansion Joint in Place. Striking Board in the Foreground.

CONSTRUCTION FEATURES.

The sub-grade was rolled with a 10-ton roller and all soft spots excavated and filled with gravel or firm material.

Concrete.—Concrete was mixed in the proportion 1:1½:3 in a Koehring paving mixer shown in Fig. 3. Wire mesh reinforcing was placed as the paving progressed, being laid transverse the pavement, allowing the 2-in. lap at the edges previously mentioned. In placing the reinforcement it was spread on a previously laid layer of concrete somewhat in excess of the width of the reinforcing and another layer of concrete placed over it immediately. In this way a plane of weakness between the layers of concrete was avoided. After striking the surface was further worked with a wooden hand float, the finisher working from a bridge immediately behind the striking board. The finishing was such that at no point would it vary more than ¼ in. from a 2-ft. straight-edge placed upon the surface in any position.

Metal protection for the edges of expansion joints was used. The details of this joint are illustrated in Fig. 1. The felt extended through the entire thickness of the pavement, the metal edges being only 2½ ins. deep.

The curb was constructed simultaneously with the pavement and was finished on the roadway face and top with a ½-in. coat of mortar mixed in the proportions of 2 parts cement, 2 parts finely crushed granite and 1 part sand. It was evenly troweled and well broomed after removing the face forms.

The sand used in concrete was of a quality which when briquettes molded from a 1:3 cement and sand mortar were tested they showed a tensile strength equal to that of similar briquettes in which standard Ottawa sand was used, proportion being by weight. Gravel was an evenly graded mixture of clean pebbles varying from ¼ to 1½ in. in screen size.

Curing.—After the pavement was completed it was covered with earth, Fig. 6, to prevent it from drying out too rapidly and to ensure thorough curing. This earth cover was kept moist.

COST

Bids were opened on March 26, 1914, and the work was completed on July 1, 1914. The bid prices were as follows:

PERSONNEL.

W. J. Walter, Glencoe, Ill., was the contractor. The pavement was designed and construction supervised by Windes & Marsh, civil engineers, Winnetka, Ill., to whom we are indebted for the data presented here.

Some Notes on the Development of Street Cleaning Methods.

Present street cleaning methods are of comparatively recent development in this country. In 1895 Col. George E. Warring began to apply the principles of engineering to street cleaning in New York City and thereby placed the work on a substantial scientific foundation. Many efficient machines and appliances for facilitating such work have since been devised. [illegible] in a paper before the American

Fig. 5. Distribution of Sand and Gravel on Subgrade Ahead of the Mixer.

Society of Municipal Improvements discussed the general features of the problems and a portion of his paper is given here.

[illegible]ENT.

The first pavement recorded in America, according to Tillson, was discovered by a farmer who, while plowing at Pemaquod, Maine, in 1625, found the point of his plowshare caught against a curb of a buried paved street. In New York the first pavement was laid in 1676, Boston about the same time, Philadelphia in 1719, Baltimore 1781, and Chicago in 1855.

It is a matter of surprise that while pavements were laid so long a time back, effective methods of cleaning were not considered until very lately and only just now have serious studies been made to solve the problem of efficient and economical street cleaning by competent men. This was in a measure due to the fact that but recently have paving experts considered the cost of cleansing to be an item to be considered in the cost of maintenance of pavements. In the main, however, it was due to the fact that the cleansing operation was considered to be an elementary proposition and therefore not worthy of consideration in its details by technical men.

The work of street cleaning is in the majority of cities performed by a bureau of some department whose other activities are considered of greater importance and the men employed are usually those who would otherwise be on the rolls of the poormaster for relief because they have attained old age without having laid by sufficient means for their support and find themselves worthless as wage earners.

Up until 1882 this work in New York City was performed by a bureau of the Police Department and in that year there was instituted by legislative act a department for the specific work of street cleaning and waste disposal. The first Commissioner in 1889, after seven years of endeavor to put the work on a proper basis, reported to the Mayor that he found his task an impossible one because he had only partial control of street conditions and because the appropriating bodies were unconvinced of the necessity to provide sufficient funds for the institution and operation of adequate systems of cleaning. In 1895 Col. George E. Waring Jr. caused considerable merriment by taking the job of Street Cleaning Commissioner seriously and in attempting to apply engineering methods to the solution of the problems involved.

Col. Waring's studies were incomplete when his term expired but he had arrived at the conclusion that the real problem of street cleaning was the removal of the fine dust, the removal of the other materials being without comparative difficulty. The fact is that the streets are cleaner today than ever, though this is not generally believed, but the fine dust problem has become more prominent in that the low-bodied fast-moving automobiles are raising and spreading the dust which before was not so raised and spread except by high winds. Formerly ordinary traffic only raised and spread the dust to about the height of the infant in the perambulator and not until it was raised and blown into the faces of grown-ups was the knowledge of the continuous existence of this menace generally recognized.

In almost every large city we today find men of the caliber of Waring who are giving this matter of street cleaning serious consideration and who are applying to this work the principles of efficient and economical performance which obtain in other lines of business activity. Unfortunately the efficiency principle seems to be to get more work done for less money cost rather than better work done for the same cost with later endeavor to reduce the cost. The bookkeeper is the efficiency referee whereas the practical operator observing practical results should be.

Fig. 6. Completed Pavement Covered with Earth for Protection to Concrete While Curing.

Then, too, the systems are founded on hand labor with machine auxiliary. In city development large street areas are being added annually and each city now may be described as "one of magnificent distances" with the result that systems must change to meet the new requirements and the basis of these should be machine work with hand labor as auxiliary.

New York has 2,173 miles of paved streets, Chicago 1,899, Philadelphia 1,371, and the other large cities have from 100 to 500 miles. As under average conditions a man can clean but about one-half mile per day, even could he perform the work with thoroughness the cost of such an army of labor for this work would be prohibitive. Machines at present used can do from two to four miles each and this work is done with a good degree of thoroughness, making daily work on the same area unnecessary, and economical arrangement of mechanical plants may be made with good results. The man then merely polices the streets as a litter gatherer and can cover a very considerably larger route.

MACHINE CLEANING.

Some look to the automobile machine to increase the workable area, but I question if there is not a limit of speed wherein effective work may be done. The benefit to be derived by the use of motor vehicles in my judgment will come rather from the ability to adapt better mechanical method to the operation, thereby improving the quality of the work, rather than increasing the amount done per machine unit.

The machine broom as today operated leaves much to be desired purely, I think, because the action of the broom is dependent upon the speed of traction, whereas should a motor vehicle be substituted, power may be transmitted to the axis of the broom independent of the tractive power of the machine, a proper rate of revolution of the broom may be found and maintained irrespective of the speed of progress of the machine. At present if you find a particularly filthy spot and endeavor to slow up to give it more careful attention, the speed of the broom lessens and the effect is lost, whereas were the power independent the result would be that required.

A mechanical device attached to the sweeping machine to pick up the stroke of sweepings from the broom and empty into a receptacle or receptacles would avoid a considerable amount of extra work now performed because the materials are spread by the wind or tracked by vehicle tires. Such machines are in evolution now but most of the devices are complicated or otherwise ill adapted to the performance of this work.

Flushing machines and squeegees are now constructed which do effective work but their adoption has been greatly retarded by the claims that the water has an adverse effect upon paving materials. Clifford Richardson, the asphalt expert, in his work on "The Modern Asphalt Pavement," says, "in a properly constructed pavement no important deterioration from water action should ensue within the life of the pavement, and, as a matter of fact, in the author's experience, the deterioration of asphalt surfaces laid under his supervision has in the last ten years become an item which is hardly worth consideration, where the form of construction has provided satisfactory drainage." As to the ill effects of water on the joints of stone and block pavements, tests made some years ago in Detroit and Cleveland showed that such action was not necessarily adverse where good construction methods had been adopted. My judgment is that the paving engineer should design his exposed surfaces to resist water action for the great benefit to be derived from water cleansing. In flushing machines there is a distinct advantage in having an attached pump to express the water so as to get positive action continuously and on all the water which the storage tank may carry. A motor flushing machine has the advantage of carrying larger quantities of water with a consequent conservation of the time now lost in frequent filling of small tanks.

The machine squeegee or rubber scrubber is effective on smooth pavements where not too filthy, but the present method of delivery of water so close to the scrubber renders the work imperfect through the ineffective action of the water on the filth before the scrubbing action is applied. This may be remedied by preceding the squeegee machines by a sprinkling cart sufficiently in advance to give the water action full effect. And here let me say that that sprinkling wagon should not be used, in my opinion, except as an adjunct to other cleansing devices, as by itself it is but a temporary makeshift and ineffective for good result. There is no reason for the filth removed by washing machines being deposited in sewer basins, but arrangements should be made for picking up the materials and carting them away as soon as they become sufficiently dry.

Dry cleaning machines are greatly to be desired and good progress is being made in the development of such. I know of two types of vacuum cleaners which are doing good work and need but little to make them practical and effective. These of course should be designed to pick up and place in receptacles the filth removed. When dry cleaning machine work is adopted there must still be periodical wet cleaning and such a combination should insure the best results. The system of cleansing must be so operated as not to interfere with the traffic conditions as the important factor of street use is for transportation. For this reason, in congested districts the machine work should be done at night.

METHOD OF MAKING A BEGINNING IN SMALL CITIES.

Where I have proposed a system based on the foregoing lines, it has been said that such a system applies to large cities only, while I feel that it is a basis upon which a small city may start and develop as the city's needs grow.

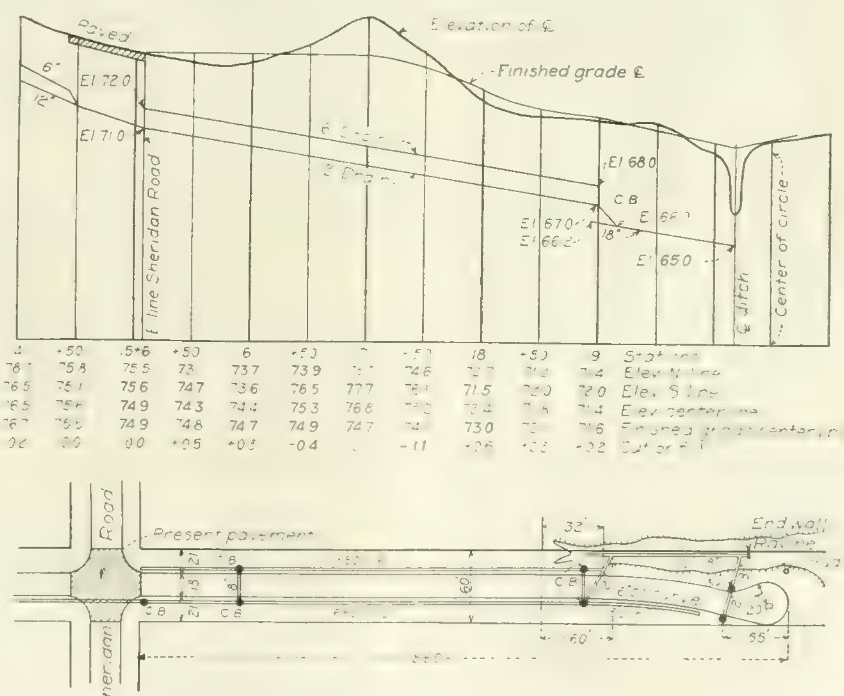


Fig. 7. Portion of Plans Showing Type of Data Placed on Construction Drawings.

My proposition then is: to district your city and arrange for periodic machine work so that machine units may be changed from one district to the other avoiding necessary duplication of plant; make combinations of machine cleaning adapted to different requirements; use dry and wet methods alternately as necessity demands; have patrolmen work as litter gatherers where accumulations develop quickly which do not necessitate machine work; in congested districts or parts of districts do machine work at night and patrol work in the day time; do not employ old men on patrol, put them on park work or other less strenuous duty; keep cost data but do not regulate your work by that but by real efficiency; count cleansing necessities as items to be considered in the choice of pavements and on any type of pavement design to resist action of cleansing devices; and give serious and continuous study to the needs of this department of municipal activity. So may cleanliness be attained in city streets and health and comfort result.

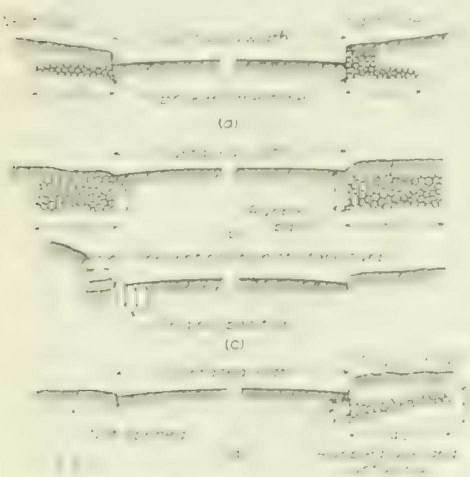


Fig. 1. Typical Example of the Effect of Foundation Movement.

The Prevention of Foundation Movement in Roads.

A subject that is becoming of great importance on account of the rapid increase in the weight of loads hauled over country roads is that of securing a firm and stable foundation for road surfaces at a reasonable cost. While it would appear that proper sub-drainage or a heavy rubble base would be a good solution of the problem in this country occasions arise where neither of these remedies are satisfactory. E. S. Sinnott, county surveyor of Gloucestershire, England, discussed some observations on the action of poor foundations in a paper before Institution of Municipal and County Engineers which paper is given here in part.

From time to time the writer has had under his observation the behavior of roads subjected to heavy traffic, and he has recently initiated certain experiments to determine, as far as possible, the lateral and longitudinal movement of material forming the sub-crust of highways. As a preliminary to more precise investigations, he has arranged, during the past few months, for various sections of grass verge adjoining the principal roads in Gloucestershire to be opened, and some of the results of such openings are shown in Fig. 1, *a, b, c, d*.

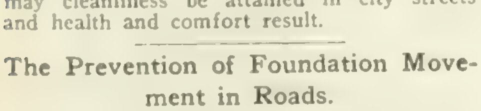


Fig. 2. Instrument for Measuring the Movement of Foundations.

Speaking generally, these indicate that lateral movement, in some instances to a considerable extent, has been taking place, and such movement seems to call for special consideration.

The following examples, Fig. 1, will serve as illustrations: (a) At this point the grass verges were opened on both sides of the road, and on the west side there was found, at a depth of about 4 ins., a bed of broken limestone having a width of 3 ft. from the metalled surface, varying in thickness from 7 ins. nearest the road to 4 ins. at the 3-ft. distance. On the east side a somewhat similar state of affairs was found to exist, the width in this case being 2 ft. 6 ins. from the verge. The subsoil is a hard, red clay.

(b) In this instance the lateral thrust is shown in the movement of a line of channelling formed of three stones on edge put in to protect the foot of a bank; originally laid with a horizontal face of 13 ins. in width, the stones have been forced into a vertical position, having moved through an angle of 90°. In several instances the upper stones have been forced completely over, having rotated through an angle of 180°, and have fallen backwards into the highway, their movement away from the road being arrested by the bank they were put in to protect. The subsoil is clay.

(c) In this case metalling and pitching have been found from 3 ft. to 5 ft. from the metalled surface, and about 18 ins. in thickness. The subsoil consists of clay and sand.

(d) The opening at this place showed that the original pitching and limestone macadam had been forced for a distance of 4 ft. from the edge of the metalled surface, the outer end showing an upward movement. The subsoil is clay.

The results above described indicate generally the position of affairs upon roads where traffic is heavy, although in a small minority of cases investigated little or no movement could be detected. It appears evident, however, that where lateral spread does occur it cannot fail to be ultimately destructive, as without substantial and immovable support the top coating cannot be kept up to its work, particularly after some wear has taken place, and it becomes a question how far new surfaces of an expensive character are justified before the subcrust movement referred to has been arrested.

With a view to obtaining more accurate information, a number of iron bolts have been lately placed below the surface of certain roads, and has fixed their position exactly by steel-tape measurement; so that any movement can be definitely ascertained on a future occasion. He has also inserted measuring instruments, as illustrated in Fig. 2, at various places, which consist of two rods arranged to slide one over the other, having iron plates at their outward extremities, which will enable any movement, either lateral or longitudinal, to be recorded.

As a means of preventing the action above described, the writer has recently designed and put into use a rigid framing, the essential feature of which is that longitudinal and cross members placed at a suitable depth below the surface preclude any movement of the subcrust, and at the same time provide a means of constructing an impervious arch of tarred macadam to carry the traffic, great additional strength being provided by the longitudinal members for the support of the heaviest road vehicles. For various reasons it was thought best to construct the frames in reinforced concrete, although timber or other material could, if preferred, be used. The form adopted after one or two experiment frames had been made was that shown in Fig. 3. The longitudinals are 12 ins. by 3½ ins. (average thickness) by 12 ft. long, and are slightly tapered from top to bottom. The ends of the longitudinals are securely housed at the extremities of the cross ties. The reinforcement consists of expanded metal, 3-in. mesh, ¼ in. by 3/10 in., weighing 11½ lbs. per square yard, cut into strips 9 ins. wide—in the case of the cross-ties this is supplemented in a minor degree with wrought-iron round bars of small section at the ends of the same. A further modification consists of making the longitudinals slightly curved on the inside in order to withstand lateral thrust more effectively; also, where difficulties due to traffic may be anticipated, in placing the cross-ties in position they can be made in two parts and connected in the center with a bolt or bolts.

In the experimental frames which have been put down the width between the longitudinals has been fixed at 8 ft.; but there is no particular reason for adopting this dimension other than that it appears suitable for dealing with the traffic conditions of rural roads in Gloucestershire, where most of the heavy weights are carried in the central portion of the highways. If considered advisable, a greater width than 8 ft. may be adopted between longitudinals, or, where traffic conditions justify it, the central pair can be supplemented by longitudinals on either side, connected thereto by cross-ties similar to those previously described. The frames have been placed as shown in Figure 4, the top edges of the longitudinals being 6 ins. below the finished road surface, which, from the information at present at the writer's disposal, appears the most suitable position for arresting the lateral movement; also, at this depth there would be little or no interference with pipes, and with 6-in. cover over the top side of the longitudinals, there appears to be sufficient cushion to avoid any damage thereto.

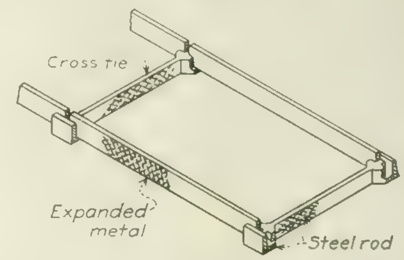


Fig 3. Details of a Concrete Frame for Confining and Preventing the Movement of Road Foundations.

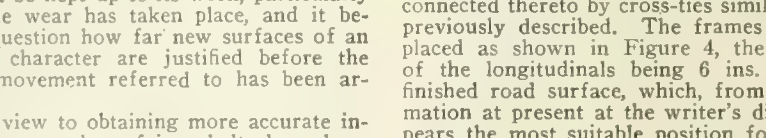


Fig. 4. Cross Section of a Road With the Concrete Frame in Place.

Exhibit of Street Cleaning Appliances.—The department of street cleaning of the city of New York will hold an exhibition of street cleaning appliances during the week beginning Nov. 23, 1914. The apparatus exhibited will include all sorts of brooms, brushes, sweeping machines, flushing machines and other equipment or appliances used in cleaning streets and removing snow; garbage, ash and rubbish cans or receptacles used by householders; carts, wagons, motor trucks or other vehicles used for collecting garbage, ashes, rubbish and street sweepings; conveyances for transporting refuse by water or by rail; methods used for the disposal of garbage, ashes, rubbish and street sweepings, including garbage reduction works, garbage crematories, incinerators, destructors, etc. The city of New York is considering the adoption of improved methods and appliances for street cleaning refuse storage, refuse collection and refuse disposal. An appropriation amounting to $250,000 has been made for the purchase of apparatus to be installed in a "model district" of the Borough of Manhattan. The object of the exhibit will be to give manufacturers or inventors an opportunity of bringing to the attention of the city officials their various appliances, likewise providing a chance for examining and testing these appliances during the period while the exhibit is being held.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design and to Methods and Cost of Construction

Volume XLII. CHICAGO, ILL., OCTOBER 28, 1914. Number 18.

Stream Gaging by Titration.

To the Editors: Referring to the editorial entitled "A Chemical Method of Gaging Stream Flow" appearing in your issue of Sept. 16, one might easily infer that the tests mentioned are being conducted by the writer. The tests at Massena are in the charge of Mr. B. F. Groat, M. Am. Soc. C. E. I am working with Mr. Groat as one of his assistants.

R. E. Ward.

Massena, N. Y., Oct. 8, 1914.

The Work of the Wisconsin Highway Commission.

The topography and intensity of settlement of Wisconsin are so varied and the types of construction and methods suitable to meet these conditions so different that the successful working out of any general plan should emphasize those factors which are fundamental in the organization and administration of a state aid system of road construction. In this connection, several features which have developed in the work of that state are worthy of note. Among them are the use of the day labor system of construction, the independent organization of counties, and the excellent morale of the state organization.

Wisconsin furnishes perhaps the best example of the extensive and organized use of day labor in highway construction, and contrary to the rather general experience of other states, the results obtained have been not unsatisfactory. Important factors in this success have been, no doubt, the lack of experienced contractors capable of undertaking first class work, the enforced divison of the work into small parts to meet the demands of numerous political units, the widely varying types of construction used, and the rather independent and individual citizenship of the state as a whole. As portions of the state become more thickly settled, the general average of improved roads bettered, and the construction of more permanent, and relatively expensive, types of surfacing become economical, it is quite probable that construction may be carried out more economically by contract work.

This condition is of course recognized by the commission. In fact in the more thickly populated sections of the state the commission is now letting contracts on much of its work. With this transition an important result which should not be overlooked is the opportunity afforded of building up an efficient system of maintenance with the various county force account organizations as a basis. A fault of the contract system of state aid road construction and a difficulty hard to overcome is the lack of efficient local organizations for proper maintenance after the construction work is completed.

The independent organization of counties we believe to be essential to successful road administration in all but very small states. It is necessary from the standpoint of future maintenance. Also, in large counties and thinly settled regions it possesses marked advantages for the building of new roads.

An interesting feature of the county organization work is the instruction of county officials in the details of first class construction. This is a thankless task at best and may lead to unfavorable criticism by some "pupils" who perhaps consider themselves sufficiently well informed. A course of instruction and personal supervision for county superintendents has, however, been found necessary in several states.

A noticeable characteristic in the employes of the commission is the enthusiasm shown in their efforts to accomplish economical work. This condition is usually a result of efforts in that direction on the part of executives, and this is possibly the determining cause in Wisconsin. But whatever the cause, the good results obtained are evident.

Interrelation of Sun Spots, Rings of Sequoia Trees, and Rainfall.

That all forces are interrelated is a generalization that often receives exemplification in a startling manner. Who, for example, would ordinarily suspect any relation between the spots on the sun, the thickness of the rings of the giant redwood of California, and the amount of annual rainfall? Within a few months there have been published two sets of data that seem to show a definite relationship between these three classes of phenomena.

Careful measurements of the thickness of the woody rings of the California sequoia—a tree that lives to be 30 centuries old—have disclosed a cyclic variation in the thickness of the woody rings. The ordinary cycles have averaged about 11 years. It was surmised that the thickness of a tree ring bears some relation to the amount of water in the soil, hence that a year of heavy rainfall would correspond with a thick ring. A comparison of rainfall records by years disclosed that in years of heavy rainfall the rings of the sequoia were thicker than in years of light rainfall. Also the rainfall records of California disclosed a rather definite cycle of 11 years.

A recent study of rainfall in Norway for a period of 35 years has disclosed a "remarkable correspondence between rainfall and the sunspot period of about 11 years." In general, when the sun spots were at their maximum, rainfall was also at a maximum, although this was not invariably the case.

Thus in two far separated parts of the earth two very different sorts of investigations serve to establish a quantitative relationship between certain phenomena that previously had not shown evidence of relationship. Should further investigations confirm the accuracy and generality of the conclusions as to the 11-year rainfall cycle, some very important applications of these data to engineering designs may follow.

Of course it is to be noted that while rainfall over large areas may show a fairly definite correspondence with the sun-spot cycles, it by no means follows that local rainfall will follow the same graph.

Since rain results from the cooling of moisture-laden air to its dew point, and since moisture in the air is caused by a rise in temperature of water, it would seem to follow that sun spots must act either by increasing evaporation or by producing dew point conditions by lowering the temperature. Spectroscopic observations indicate a lower temperature within a sun spot than elsewhere on the sun. Hence it may be inferred that a year of maximum sun spots is a year of minimum heat radiation to the earth. If this is so, the heavy rainfall of such a year is caused by an abnormal lowering of temperature that serves to abstract from the atmosphere much of the moisture accumulated in preceding years of higher average temperature.

The earth's atmosphere is a reservoir holding a vast volume of water. The gate that opens the reservoir is lower temperature. Water lifted years ago by the sun from oceans and lakes flows back into these lower reservoirs when the gates of the upper reservoir of the air are opened. The sun spots appear to be instrumental in opening the gates.

The Effect of the Illinois Architects' License Law.

The report of the secretary of the State Board of Examiners of Architects, presented at the Illinois State Convention of Licensed Architects, is an exceedingly interesting statement. Although the law requiring that architects practicing in Illinois must be licensed has been in effect for 17 years, few attempts have been made to enforce rigidly its provisions until the present Board of Examiners assumed office. Before that time there had been no Supreme Court decision touching upon the legality of the act and the board, whose duty it is to enforce it, had no precedent to follow. It is interesting to note that since the legality of the act has been established the board proposes to enforce the provisions of the act on its broad interpretation. In view of the importance of some of the statements contained we are publishing the report in full in the "Buildings" section of this issue, and we recommend that it be given careful study by engineers.

In connection with the agitation which has arisen in several states to secure the passage of laws requiring that engineers obtain a license in order to practice their profession, it is interesting to note some of the practical effects of the Illinois Architects' License Law. On Dec. 1, 1898, 744 architects were granted licenses, of whom 701 obtained their licenses because they were practicing architecture before the law became effective, and 43 passed the examination of the board. Today there are only 863 licensed architects in Illinois, 410 of whom secured their licenses because they were practicing their profession before the passage of the law, and 453 passed the numerous examinations held by the board. During this period of 17 years 500 licenses have been revoked. There are today only 119 more licensed architects than in 1898 and only 17 more than were practicing two years ago. Since the publication of the last biennial report on Dec. 1, 1912, *104* new licenses have been *granted* and *87 revoked*, this being due largely to the activity and new policy of the present board. The ruling of this board is that "only a licensed architect can practice in this state, or from this state, and that his license is not transferable nor negotiable. Any combination formed for the practice of architecture, except between licensed architects, is illegal, and any licensed architect who assists others to practice, who have no license, is guilty of dishonesty, as provided in the act, and should have his license revoked."

We wish to call attention particularly to the statements in the report affecting engineers who wish to engage in any phase of building design. Further, we desire to call to the attention of engineers the fact that *by legal right* architects have established themselves as supreme in *every* class of building design, and that they now seek to exercise the same authority over the *supervision* of building construction, by legislation. All this has been made possible by *close organization* and *united action* on the part of architects throughout this country.

The Cost of Draining Overirrigated Lands on U. S. Reclamation Service Projects.

That real danger as well as extravagance lies in overirrigation is a truth which the irrigation water user is slow to perceive. The average irrigationist uses almost habitually more water than the soil can contain within the zone of plant growth. A part of this water consequently sinks into the lower soil strata. Gradually these subsoils become saturated and finally water comes to the surface on low areas or on the lower portions of the slopes down which the surplus water is percolating. This rising water brings the alkali to the surface, or at least the soil is waterlogged, and crop growth is destroyed. This phenomenon is elementary knowledge to the irrigation engineer and manager, and they have repeatedly warned the water user that it must occur except when some unusually favorable condition prevents. Disaster seems, however, to have been essential to bring conviction, and disaster has come to wide areas of the land put under irrigation within a few years by the U. S. Reclamation Service. It will by the end of 1914 have cost the service nearly $2,500,000 for drainage works to care for seepage and overirrigation. The figures, including authorized expenditures for the year 1914, are:

Project.	Total estimated expenditures to Dec. 31, 1914.
Salt River	$ 1,191.7[illegible]
Yuma	185,608.36
Orland	1,000.00
Grand Valley	2,000.00
Uncompahgre	6,498.93
Boise	309,518.43
Minidoka	700,639.63
Huntley	279,692.42
Lower Yellowstone	65,330.00
North Platte	133,221.67
Truckee-Carson	67,709.01
Carlsbad	67,780.00
Rio Grande	50,[illegible]00.82
Umatilla	54,067.96
Klamath	194,697.20
Belle Fourche	2,000.00
Strawberry Valley	11,118.80
Shoshone	366,183.00
Total	$[illegible]

[illegible] the drainage works there have been a general lowering of the water plain over an area, reclaimed and protected, of [illegible]. Assuming the higher figure, the cost has been $62.50 per acre, quite as much as the cost of the original irrigation. The condition indicated conveys its lesson without explanation, but a touch of absurdity is lent by the comment of the Reclamation service engineers that "in some cases it has been found, where drainage works have been effective in lowering the ground water so as to permit lands being brought under cultivation, that the excess of water applied to the soil through irrigation has been sufficient to again raise the water table too near the surface during the peak of the irrigation season, and to produce temporarily a condition injurious to crops."

The Employment of Experts by Cities.

After enumerating 14 of the major accomplishments of the Philadelphia Department of Public Works in carrying out its constructive program, Director Morris L. Cooke, in his [illegible]

[illegible]

while others have to be retained on a definite retainer.

Mr. Cooke's arguments favoring the employment of the man of exceptional qualifications are convincing. He extends the principle involved to cover the handling of routine as well as exceptional activities. His theory is that every position from the lowest to the highest should be filled by one expert in the duties appertaining to that position. He gives numerous examples of the employment of experts, both for handling the highly complex sporadic problems to which large cities are subject and for handling less pretentious matters of more frequent occurrence and the accompanying routine duties.

Experts of the first type have advised the city in matters pertaining to architecture, landscape work, electric lighting, highway engineering, building, street and steam railways, mosquito extermination, management, water, printing, gas and gasoline lighting, acoustics, time study, municipal finance, markets, construction work, street cleaning, sewage disposal, power plants, garbage disposal, slow sand and mechanical filters, and law.

Mr. Cooke calls attention to the fact that the average person seems to believe that anyone can fill a city job acceptably. He sees clearly that the business of cities ought to be handled with the same efficiency exhibited in the transactions of the business world. Commenting on this matter, with special reference to the employment of experts, he says:

> In the industries it is becoming more and more the rule to go to experts—to the men who know. The routine work of an establishment will always be best done by those who are schooled to it. But if progress is being made, questions are constantly arising which can only be answered by those who have had the peculiar experience in which the answering facts are developed. All such questions are being answered today more and more by the logic of facts rather than by personal opinion. It is to the expert that we must go to get these facts.

The quotations given relate to the employment of experts for short time engagements for the study of matters of highly complex nature. As stated previously, however, the department has extended the principle of employing "men who know" to the engaging of men to give all their time to the city. These latter men fill the various steady positions in the department and are selected for their special fitness for their positions. Many of the more important steady positions were filled by men called to Philadelphia from similar positions in other cities. Thus such important positions as the chiefship of the Bureau of Highways and the Bureau of Water were filled by men from other cities.

The following quotation states the conviction of the department relative to the calling in, from other cities, of men in the lower salaried grades:

> A study of the German system of municipal administration shows that it is only a matter of a few years until this country will have progressed to the point where we will exchange as between cities not only those who hold higher positions but men and women in the [illegible] of municipal administration and engineering becomes more and more of a science it will become more and more usual for the officials of one town to seek assistance of all grades in any other town or city where it may be found.
>
> The rise of the "commission-manager plan" of city administration will hasten this movement. It seems quite clear that those in responsible charge of municipal administration must more and more watch the work being done in towns and cities other than their own, to secure the people with whom to build up and maintain an efficient administrative staff. That town or city which entirely depends upon its own will find out sooner or later that too much inbreeding in this field is as bad as it is in any other.

That Mr. Cooke is a consistent advocate of the exchange between cities of municipal employes is shown by the following quotation:

> If there is any town in the country of over 5,000 inhabitants in need of a city manager or one which wants to fill any other position and if they are willing to pay sufficient salary we will be glad to point out the best man in our service to take the place. We have good men and women working in our departmenet and we want to see them get ahead even if we lose some good ones in the process. In other words, we feel it is quite as important to provide means of advancement for our people either in or out of the city service as it is to fill vacancies satisfactorily as they occur. Ninety-nine out of every hundred promotions will probably come through the city service and in Philadelphia, yet nothing would act as a greater tonic for our own municipal service than to constantly have a certain percentage of our best employes given better positions in outside cities.

The several quotations given contain many valuable suggestions. Clearly, if the handling of city business is to be raised to the plane of economy and efficiency exhibited in private business enterprises the employment of competent men for all municipal positions will be a necessary step in the transition. We desire especially to commend the employment of recognized experts for the handling of the larger and more complex problems. Many city councils will not now appropriate money for the employment of such experts, but where the need for expert services is well defined various civic organizations and public-spirited citizens are usually ready to make generous contributions for that purpose. In fact the City Council of Philadelphia has placed every possible obstacle in the path of securing expert advice. In all but a few cases the fees for experts called to Philadelphia have been paid from funds privately subscribed.

The engineer can do much to promote a better understanding and a higher appreciation on the part of city officials and private citizens of the value of expert services. It is true that some engineers need to learn in this matter as well as laymen. Too often the self-sufficient engineer has declined to ask for the help of expert advice. Again some engineers consider it a reflection on their own capabilities to admit that they could profit from another man's advice. Happily the employment of experts is now sufficiently common to make the last-named objection of little weight.

ROADS AND STREETS

The Organization and Standards of the Wisconsin Highway Commission.

(Staff Article.)

The [illegible] Highway Commission [illegible] to that time the state geological survey furnished engineering aid to the va[illegible] in cases where roads were constr[illegible] the [illegible] In 1911 the state aid law [illegible] the legislature and its scope materially broadened.

[illegible]

The highway commission consists of five non-paid members; three appointed by the governor and ex officio; the dean of the college of engineers of the State University; and the state geologist. The executive officer of the commission is the state highway engineer, appointed by the commission. The necessary engineering assistants are appointed by the

In each county of the state the board of supervisors elects a county highway commissioner, as he is called, who has the direct supervision of all road work accomplished in the county, under state aid. The candidate must satisfy the state commission as to his fitness, his appointment being subject to their approval. In case a county fails to appoint a road commissioner one may be appointed by the state highway commission.

Funds are derived from a direct tax levied by the state and are apportioned to the various counties in direct ratio to the percentage of state tax paid by the county. In addition to the money appropriated for state aid there is

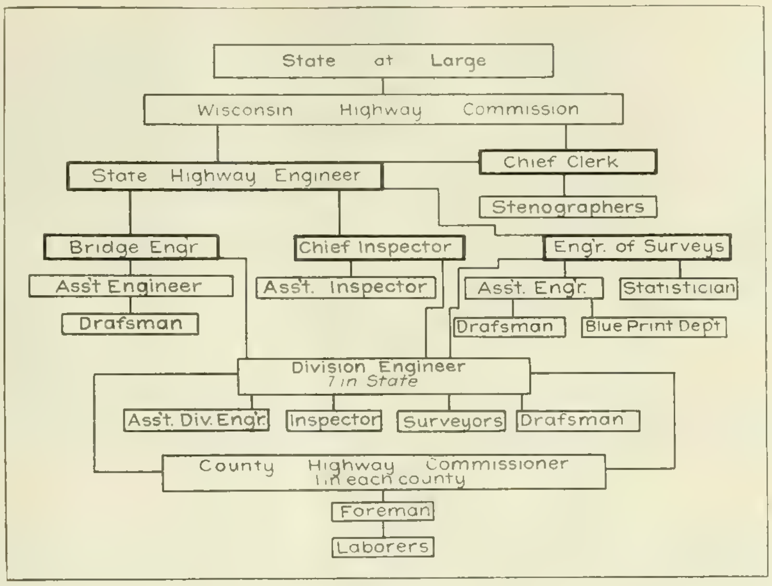


Fig. 1. Diagram Showing the Organization of the Wisconsin Highway Commission.

also provided money for the support of the commission. Funds are apportioned for state aid on a basis of one-third each of the cost of the proposed improvement to be paid by the township, county and state; or, two-thirds of the cost by the county and one-third by the state; provided, that where bridges exceed 6 ft. in span the state pays only one-fifth of the cost. On this basis state aid road improvement funds amount to about 4¼ million dollars in 1914, of which the state pays about 1¼ million. The initiative in appropriating money for state aid work lies with the local unit—the county or township—the state meeting requests for aid up to the limit of the funds available for the use of the local unit. Counties and townships may also issue bonds for road improvement.

State aid is granted only for the construction of roads included in a previously selected system of county highways. These roads selected by the various county boards and their selection approved by the commission, comprise about 15 per cent of the total mileage of public roads in the state. It is proposed to complete the improvement of this system of roads before undertaking the building of roads not included in the system.

It is readily seen that the work of the commission is of a supervisory nature, direct ad-

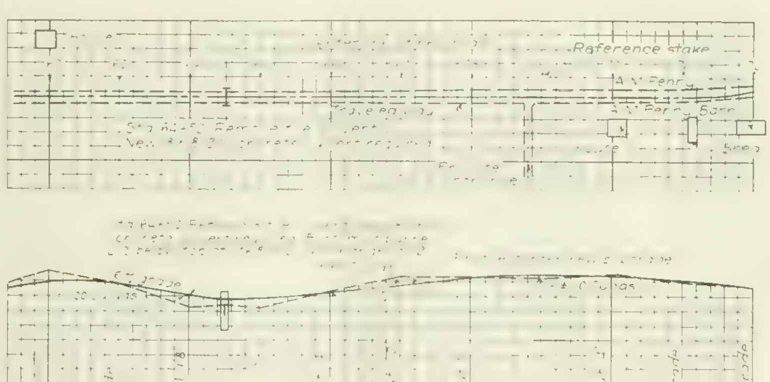


Fig. 2. Portion of Road Plans Showing Type of Information Given.

ministration being in the hands of the local unit. However, the commission has power to assume direct charge wherever such action is necessary in order to secure a quality of work which will conform to the standards established by the commission. The chart, Fig. 1, shows graphically the organization of the commission and the relation of the employees one to another. After roads are completed the expense of maintenance must be met by the counties and townships.

GRADE SHEET.

STK. NO.	DIST. FROM STAKE TO CTR. OF NEW ROAD	DIST. CTR. OF NEW SUBGRADE ABOVE OR BELOW TOP OF STK.		CUT OR FILL
39+30		BELOW	0'-0"-Stk. #39	CUT 0'-7"
40	46'- 8"	ABOVE	0'- 8"	FILL 0'-4"
41	28'- 7"	BELOW	1'- 3"	GRADE
42	27'-11"	"	1'- 2"	CUT 0'-2"
43	27'- 2"	"	0'- 9"	GRADE
* * *	* * *	* * *	* * *	* * *
62	25'-11"	BELOW	4'- 7"	CUT 1'-7"
63	25'-10"	"	0'-11"	GRADE
64	28'- 6"	ABOVE	1'- 3"	FILL 1'-8"
65	31'- 6"	"	0'- 4"	" 1'-2"
66	33'- 5"	BELOW	4'- 4"	CUT 0'-4"
67	32'- 4"	"	3'- 6"	" 1'-2"
68	31'- 0"	"	3'- 2"	" 0'-5"
69	32'- 9"	"	2'- 2"	FILL 0'-5"
70	32'- 1"	"	3'- 4"	CUT 0'-4"
71	34'- 6"	"	4'- 3"	GRADE
72	25'-10"	"	1'- 9"	

NOTE:—In grading check the height of finished grade by the last column of the grade sheet which shows the distance the new subgrade is above or below the top of the numbered stake.

Fig. 3. Grading Data Placed on the Plans for the Use of Foremen and Inspectors.

THE PRACTICAL WORKING OUT OF THE PLAN.

The plan of the state highway law is comprehensive and farsighted. In putting the plan in operation various interesting expedients were resorted to in order to establish the undertaking on a firm basis.

Perhaps the most interesting of these expedients is the almost universal adoption of the day labor system of executing work. It was found that by no other system could many short isolated stretches of state aid road be improved at an equally low cost. Also the wide variation in the wealth and density of population of the various counties in the state made economical a similarly wide variation in the type of improvement advisable to secure the greatest benefit to the residents.

In turn, these conditions of sparse settlements and short, isolated road improvements and the resultant economy of day labor methods of road construction made advisable the appointment of many county highway commissioners who were not engineers or surveyors, but rather competent road foremen or superintendents. The work of making surveys, therefore, devolved upon the commission. These surveys are made by engineers in the office of each division engineer and are checked and approved by the state highway engineer.

An important and far-reaching saving effected by the state commission was in the matter of freight rates on road materials. Due to the efforts of the commission, a material reduction was secured, the amount saved by the reduction amounting to about $75,000 during the current year.

Another interesting phase of the work was the institution of schools for road commissioners during the winter months, and the planning of a campaign of public education in the details of the work of the commission and the difficulties with which it was contending. At the road schools, or meetings of road officials, details of construction are discussed and experiences exchanged. The effect of these meetings has been to bind together and create a feeling of working support and co-operation between the various county officials. Numerous speeches and lectures are delivered by employes of the commission and a spirit of frank criticism has been developed on the part of both the press and people at large and the commission, which, although heated at times, has resulted in great good. Newly acquired technical knowledge is often displayed in curious and sometimes amusing opinions from critical citizens. And yet while possibly annoying and misleading in some of its features, this interest evinced by the public will eventually make easier the work of the commission. The policy of directness and publicity is to be commended.

COST OF WORK ACCOMPLISHED.

A recent bulletin of the commission states that the roads constructed may be roughly divided into five types: (1) *Roads of the first class:* Concrete, sheet asphalt, sandstone for granite blocks, or vitrified brick. (2) *Roads of the second class:* Stone macadam roads consisting of stone which has passed [illegible] Comprised in this class are also those roads composed of a first course of uncrushed rubble stone or gravel and a second course of crushed stone. (3) *Roads of the third class:* Gravel macadam roads; those roads constructed of gravel, as it occurs in the pit or of screened gravel, including roads built of rubble stone for the first course and a gravel surfacing. Roads of crushed gravel are considered as stone macadam and are thus classified. (4) *Roads of the fourth class:* Roads surfaced with shale, clay or sand. While these may be called surfaced roads, the surfacings in general are not of such character as to make permanent improvements, and are not turned over to the county for maintenance as in the case of classes (1), (2) and (3). (5) *Roads of the fifth class:* Dirt roads, consisting of roads which are graded but not surfaced. Roads in this class on which a small amount of gravel has been placed to

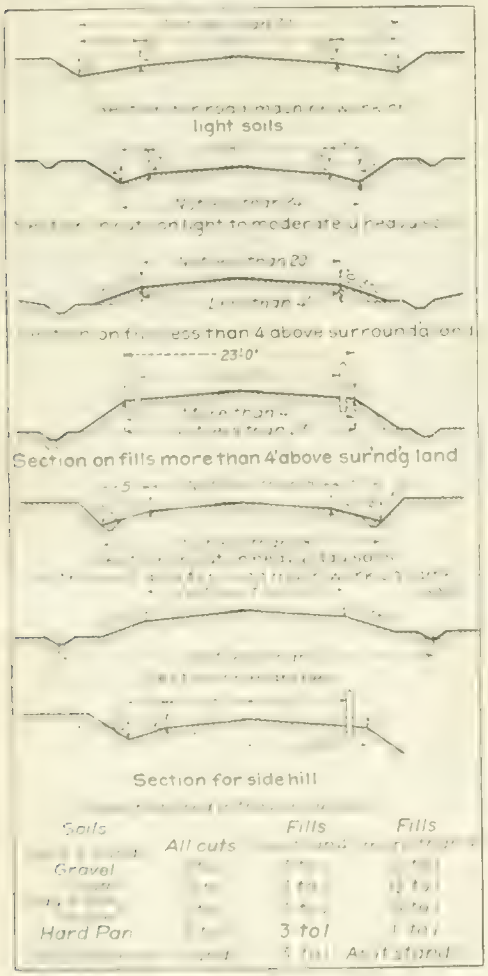


Fig. 1. Standard Cross Sections Used in Grading All Types of Roads Except Those Paved with Concrete.

[illegible] considered to be in class (3).

The approximate average cost per mile for the whole state reduced to a surfaced width [illegible] as follows: Class (1), $8,500; Class (2), $3,083; Class (3), $2,123; Class (4), $2,131 The average cost per mile for roads graded [illegible] grading and culverting the total mileage, in- [illegible] $750 per mile. Reduced to a basis of square [illegible] $1.30 per square yard, stone macadam 44 cts., gravel 26 cts., exclusive of grading and culvert work.

The unit costs of work completed in typical counties of the state are given in Table II.

The percentage of the total cost expended for state supervision has been reduced in three years from 3.53 per cent to 2 per cent, an average [illegible] 2.72 per cent. In addition to the above supervision cost paid by the state each county has a supervision cost varying from 1½ to 8 per cent, averaging for the state probably a little less than 4 per cent. The joint cost of the supervision of state road construction by county and state is therefore about 6½ per cent. No part of the expense of supervision is paid out of tax moneys raised for construction purposes.

ROAD PLANS.

The interesting feature of the plans prepared by the commission is the including of all information necessary for use in the field in the laying out of the work on the original plans. Surveys are made, under the direction of the division engineers, of sections of road ordered surveyed by the commission. Center line, profile and cross sections are determined and a reference stake driven near the road property line at 100 ft. intervals. The distance of these stakes from the center line and the elevations of the tops of the stakes are noted.

In preparing plans continuous transparent profile paper 10 ins. in width is used. The plan and profile are platted in the manner shown in Fig. 2, information being placed on the plans as shown. Ordinarily scales of 1 in. to 80 ft. horizontal and 1 in to 8 ft. vertical are used for the profile and 1 in. to 80 ft. for the plan. At one end of the sheet a tabulation of reference distances and cuts and fills is placed. The tabulation is made up as shown in Fig. 3. This grade sheet enables the road construction foreman to set his own grade stakes without the aid of an engineer. To this end foremen are frequently furnished with hand levels and instructed as to their use. Other information on the plans is made quite detailed for the convenience of foremen.

GENERAL CONDITIONS AFFECTING CONSTRUCTION.

The topography of Wisconsin is quite varied. Some prairie sections exist, but for the most part it is broken and irregular, abounding in streams. Likewise local conditions vary. In the northern section settlements are few, the country is timbered and farming has not developed to the same extent as farther south. In the southern portion of the state is found one of the most prosperous farming regions in the United States. Several large cities are found in the state but for the most part the towns are small and are settled mainly by retired farmers. In the eastern portion of the state there is much manufacturing.

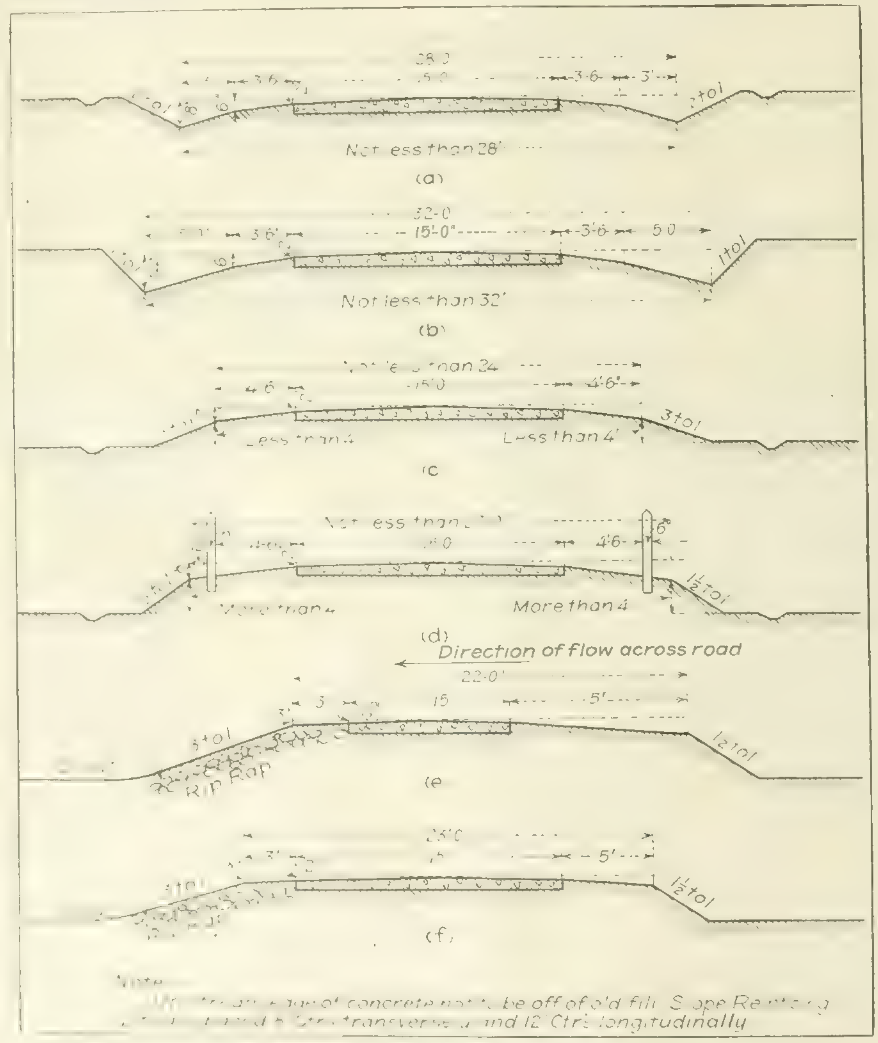


Fig. 5. Standard Cross Sections for Roads Paved With Concrete.

[illegible] Section in cuts on heavy clay soils. (c) Section on fills [illegible] 4 ft. in height. (e) Section for overflowed roads [illegible] ft. wide.

Road materials of good quality abound in all parts of the state. Glacial gravel of excellent quality is found in many sections both mixed with clean sand and with clay. The southern portion of the state has much limestone, and granite of a good quality is also found.

CONSTRUCTION DETAILS.

Cross Sections.—A number of typical standard cross sections for grading and paving, Fig. 4, are used characterized by the facts that (1) no road is built less than 20 ft. wide on fills and 24 ft. wide between ditches in cuts, (2) the finished cross sections for earth and paved roads are ordinarily the same, and (3) a 9-ft. width of paving is advocated for ordinary rural roads.

In Fig. 5 various cross sections for concrete roads are shown. Where this type of pavement is constructed the width is ordinarily greater than 9 ft. ENGINEERING AND CONTRACTING for May 19, 1914, contains an article discussing the methods and cost of concrete road construction in Milwaukee County, Wis. Further discussion of methods of construction of all types of roads will be found in the issue of April 22, 1914.

Types of Pavements.—A large portion of the roads improved have been graded but not surfaced. There is, however, an increasing tendency shown toward the construction of more paved roads as shown in Table I.

TABLE I.—SHOWING TOTAL FUNDS AVAILABLE AND ACTUAL CONSTRUCTION IN MILES IN 1912 AND 1913, AND ESTIMATED CONSTRUCTION IN 1914.

	1912.	1913	1914.*	Totals.
Funds available	$1,305,930	$2,606,004	$4,362,706	$8,274,640
Stone macadam	164	290	400	854
Gravel macadam	54	150	300	504
Concrete surfacing	10	26	75	111
Shale and other surfacing	18	65	100	183
Graded but not surfaced	223	465	700	1,388
Totals, miles of road	469	996	1,575	3,040
State aid bridges built	140	300	305	745

*Estimated.

The principal types of paved surfaces used are earth, gravel and crushed stone macadam, and concrete. But little oiled or bituminous surfaced road has been constructed.

Guard rails are constructed on all fills over 4 ft. in height, the details of construction and method of placing being shown in Fig. 6.

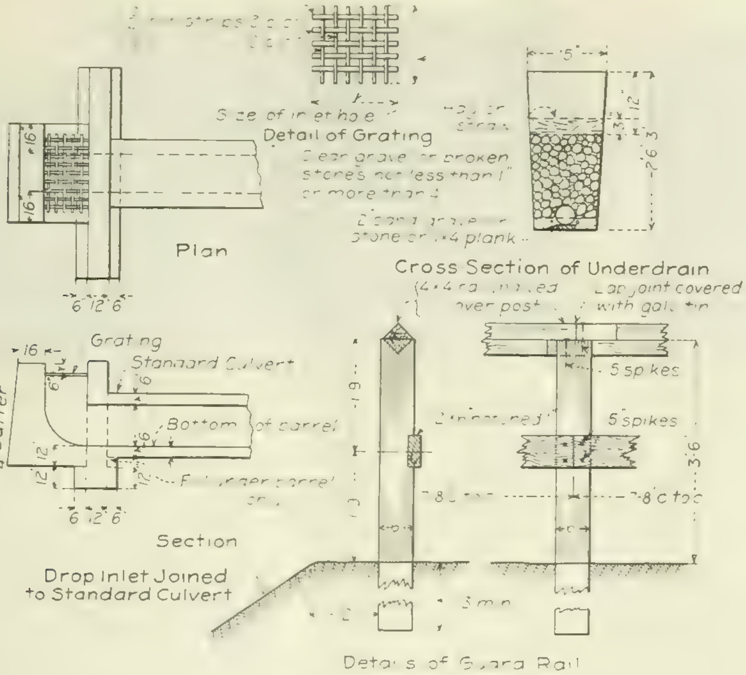


Fig. 6. Details of Guard Rail, Underdrain and a Drop Inlet.

CULVERTS AND BRIDGES.

Types of small culverts and bridges in use are illustrated in the accompanying figures. The following description of culvert and bridge construction in Wisconsin by A. R. Hirst, chief engineer, abstracted from a paper before the American Road Congress discusses the subject fully. All waterway structures 6 ft. and under in span are classed as culverts, and are built out of the funds available for the construction of the road. All waterway structures over 6 ft. in span are considered as bridges and must be provided for by separate appropriations, the state paying 20 per cent of their cost instead of 33⅓ per cent as in the case of culverts on roads.

Wisconsin has a drastic bridge law which provides that culverts under 18 ins. in span must be so constructed or reconstructed as to stand without planking a load of 18 tons, and waterway structures over this span must be designed, "in accordance with standard engineering practice," to stand a load of 15 tons without planking.

In addition to the duty of designing all state-aid bridges and culverts, the state highway commission has imposed upon it by law the duty of approving as to their safety and engineering sufficiency the plans of all bridges constructed with county aid. In the seven seasons of the existence of the Wisconsin highway commission and its predecessor in highway work, the Wisconsin geological and natural history survey, about 1,000 highway bridges of a span exceeding 10 ft. for counties, and about 400 bridges exceeding 6 feet in span, and innumerable culverts under this span have been designed. In addition to this, plans for probably 500 bridges have been checked as to engineering sufficiency. Actual bridge construction under the commission plan has cost about $2,000,000.

Bridge superstructures are roughly divided into five classes, as follows:

Class 1. Spans from 18 ins. to 10 ft.—Almost invariably constructed of reinforced concrete of the slab type. Even the smallest culverts are of reinforced concrete, which are found in Wisconsin to be serviceable, easily constructed, and in probably 50 per cent of the cases cost less than any other type of culvert except wood, which is not allowed on any of the work. The smallest concrete structure is 18 ins. by 12 ins., smaller sizes being equally as expensive and very easily blocked with ice and debris.

In some cases where the foundation conditions are extraordinarily soft and difficult, or concrete materials are not reasonably available, culverts of corrugated metal with concrete or stone end walls are used but only where it is impracticable to use concrete, and probably not 1 per cent of culverts are now built of this material. Vitrified clay pipe are not used at all on state-aid work, as they have been found almost invariably to crack in cold weather by filling and freezing. Concrete pipes molded in place and afterwards moved to the job are not used for the same reason. Under conditions favorable to their use, both make excellent culverts, and may be recommended in less severe climates. Cast iron water pipe has not been used on account of its cost, which invariably exceeds that of concrete.

Small concrete culverts ranging from 18 ins. to 10 ft. in size cost per cubic yard, complete, including the excavation and backfilling, about $8, very seldom more than $10 per cubic yard, sometimes as low as $6.

Class 2. Spans 10 ft. to 40 ft.—Either reinforced concrete, the slab type up to 18 ft. and the through girder type up to 40 ft., or I-beam structures with concrete floors are used. Many true reinforced concrete bridges are built, but, with public lettings open to any bidder, the workmanship and finished appearance has been so poor in many cases that it was found desirable to use more I-beam structures. In the case of I-beams, a 5-in. concrete flat slab is placed on top of the I-beams. The corrugated arch type of support for the concrete between the I-beams is but seldom used.

Class 3. Spans from 40 to 80 ft.—Warren riveted pony trusses are used practically exclusively, though a few plate girders are being used where the conditions of hauling are favorable. All of these structures have concrete floors.

Class 4. Spans from 80 to 135 ft.—Riveted Pratt high trusses are used with a horizontal top chord, also with a reinforced concrete floor.

Class 5. Spans Over 135 ft.—Pratt riveted high trusses with a curved top chord

Fig. 7. Typical Limestone Macadam Road 9 Ft. Wide in Argyle County.

Fig. 8. Granite Macadam on a Sand Subgrade in Portage County.

are used. Practically all of these larger spans are also built with a reinforced concrete floor. Very seldom is a pin connected truss used, either for Class 4 or 5, probably not once in 25 cases.

Costs.—From cost figures on all bridges constructed up to 1911 for any span, the price erected (including substructure and superstructure) figures out very close to $40 per linear foot of the overall span. Reinforced concrete floors average about 20 cts. per square foot. Steel in plate girder and truss spans averages from $65 to $70 a ton erected, and I-beam spans figure from $50 to $60 a ton erected.

Widths for concrete culverts and bridges follow the standards recommended by the Association of State Highway Departments, which are as follows:

	Feet
First class roads—	
Culverts under 12-ft. span minimum width	24
Slab bridges over 12-ft. span, minimum width	20
All other concrete spans, minimum width	24
Very long bridges less if necessary	
Second class roads—	
Culverts, less than 12-ft. span, minimum width	20
Slab bridges over 12-ft. span, minimum width	18
All other concrete bridges, minimum width	18
Third class roads—	
Culverts less than 12-ft. span, minimum width	20
Slab bridges over 12-ft. span, minimum width	18
Longer bridges than 60, minimum width	16

Steel bridges are built almost invariably with a 16-ft. roadway; that is, 16-ft. clear distance between trusses or rails, no matter what the class of road, although for spans under 80 ft. some 18-ft. and 20-ft. clear roadways have been built.

Abutments under practically all structures are plain concrete, since with concrete materials as cheap as in Wisconsin, and with the difficulty of getting first-class workmanship in reinforced concrete foundations without constant inspection, this is the cheapest type of abutment. Occasionally cement rubble masonry abutments are used, also steel I-beam piles surrounded by a concrete wall. The last type of abutment has been found to be very satisfactory and economical for high abutments on sandy bottoms, and has largely displaced the use of cylinders with steel backing. Steel backing is not allowed on any state-aid structure. The price of concrete in bridge abutments and piers averaged last year about $8 per cubic yard.

PERSONNEL.

The present members of the Highway Commission are John A. Hazelwood, chairman; W. O. Hotchkiss, secretary; F. E. Turneaure, John S. Owen, and J. H. Van Doren. A. R. Hirst is chief engineer for the commission. His staff consists of M. W. Torkelson, bridge engineer; J. T. Donaghey, chief inspector; A. L. Luedke, engineer of surveys; William Dawson, chief clerk, and 7 division engineers, as follows: No. 1, Madison, F. M.

Fig. 9. Type of Earth Road Grading in Northern Wisconsin.

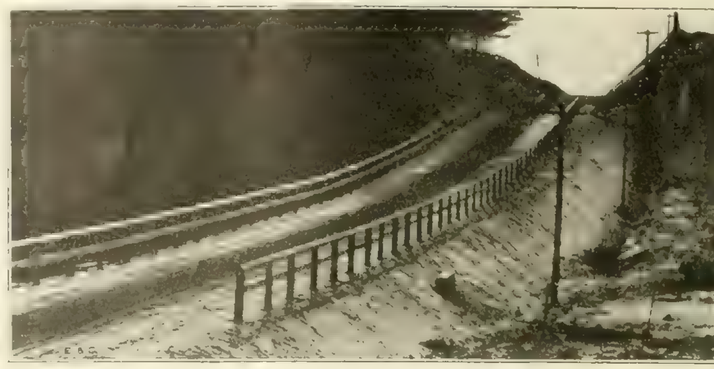


Fig. 10. Fill, Culvert and Guard Rail on a Road in Pittsfield County.

TABLE II.—COST OF MACADAM AND GRAVEL ROADS CONSTRUCTED IN TYPICAL WISCONSIN COUNTIES IN 1913.

Item.	Brown County.	Dane County.	Dodge County.	Monroe County.	Racine County.
Grading, miles	13.01	31.84	18.28	14.14	6.06
Cost of grading, per mile	$707.00	$793.00	$650.00	$775.00	$987.00
Grading, cu. yds.	....	....	41,239	....	10,215
Cost of grading, per cu. yd.—					
Average	$0.31	$0.30	$0.29	$0.33	$0.58
Maximum	.52	.97	.74	.67	1.09
Minimum	.16	.11	.08	.21	.28
Concrete culverts, No.	33	97	43	55	5
Concrete, cu. yds.	320	942	350	4	34
Cost of concrete, per cu. yd.	$9.83	$8.21	$7.90	$9.79	$11.29
Cost of culverting, per mile	$242.00	$242.00	$152.00	$356.00[2]	....
Macadam surface, miles	9.31	16.4	7.41	9.78	3.01
Macadam surface, cu. yds.	14,723	20,338	14,103	14,445	4,382
Macadam surface, per cu. yd—					
Average	$1.91	$2.33	$1.36	$2.50	$2.13
Maximum	2.47	3.26	3.10	3.60	2.94
Minimum	1.45	.91	.65	1.95	.85
Macadam surface, sq. yds.	52,842	89,883	35,450	51,216	16,082
Macadam surface, per sq. yd.—					
Average	$0.53	$0.53	$0.54	$0.71	$0.57
Maximum	.98	.75	.78	.87	.68
Minimum	.41	.32	.33	.47	.35
Gravel surface, miles	3.37	3.55	4.44	...	0.59
Gravel surfacing, cu. yds.	5,076	4,835	8,652	...	895
Gravel surface, per cu. yd.—					
Average	$0.99	$1.76	$0.74	...	$1.45
Maximum	1.51	3.22	1.79	...	1.45
Minimum	.68	1.30	.54	...	1.45
Gravel surface, sq. yds.	17,845	18,800	23,500	...	3,100
Gravel surface, per sq. yd.—					
Average	$0.28	$0.45	$0.28	...	$0.42
Maximum	.43	.53	.59	...	.42
Minimum	.17	.30	.22	...	.42
Miscellaneous	$628.69	$8,841.42[1]	$1,091.61	$215.00	$25.00
Total cost of work	$16,097.16	$99,452.84	$41,497.28	$56,950.39[3]	$17,091.82

[1]Material on hand. [2]Including 14 metal culverts at a cost of $570.07. [3]Including 0.16 miles of concrete paving at a cost of $1,001.50. [4]Tile drain, 700 lin. ft. at a cost of $25.

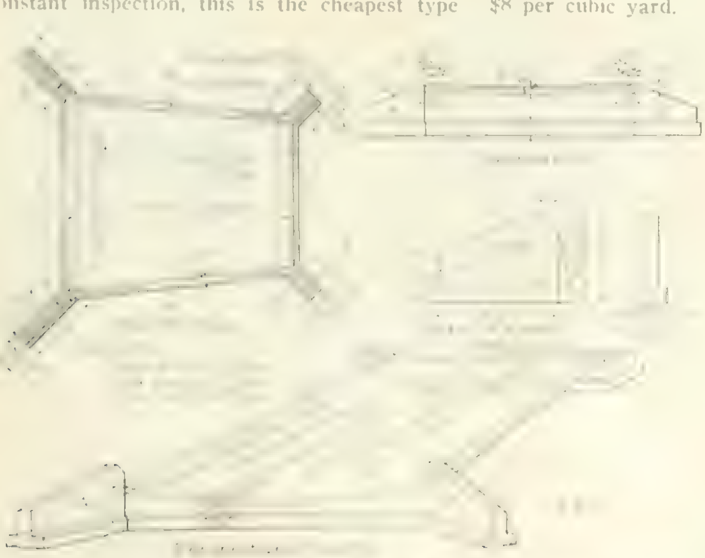


Fig. 11. Type of Chute or Paved Ford Successful Under Special Conditions.

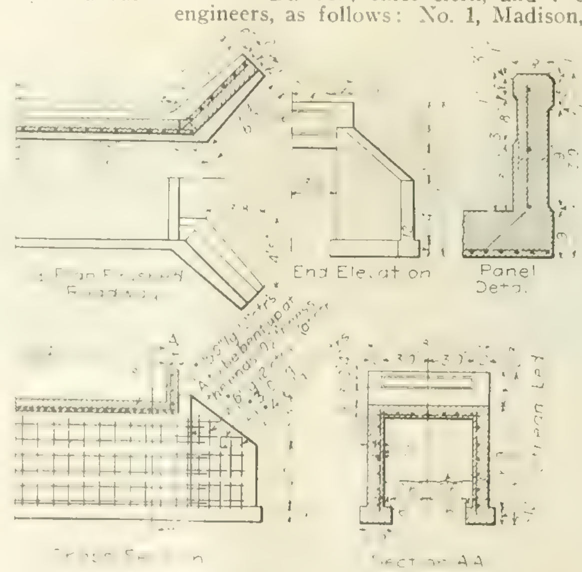


Fig. 12. Standard Type of Concrete Culvert with Unpaved Bottom.

Balsley; No. 2, Milwaukee, W. M. de Berard; No. 3, Green Bay, William Conway; No. 4, Grand Rapids, J. E. Gillespie; No. 5, La Crosse, W. C. Buetow; No. 6, Eau Claire, S. P. Hall; and No. 7, Ashland, F. M. Sargeant.

The Traffic Limits of Various Types of Pavements.

The economic traffic limits of various types of pavements used for street and country road surfaces have not yet been fully determined. By traffic limit is meant the amount and type of traffic a pavement will carry without undue wear. W. deH. Washington in the report of the New York State Commission of Highways for 1913 briefly summarizes European ideas upon this subject.

THE TRAFFIC LIMIT OF WATERBOUND MACADAM.

It has been thoroughly proven that, after traffic reaches a somewhat variable degree of speed and intensity, ordinary waterbound macadam becomes dusty, muddy, is worn into ruts and holes, and disintegrates so rapidly as to require constant mending and resurfacing.

Just what this economic traffic limit of waterbound macadam is has not yet been fully determined. But it is very essential to know it, and in many places tests and censuses are being made to ascertain it more exactly. Both in England and France the opinion is that if simple waterbound macadam does not last two years without resurfacing it should be changed to another type of construction, or at least be superficially tarred to increase its life, if possible, to or beyond this period. Another criterion suggested by experience in England is that if the maintenance of waterbound macadam costs more than a third of a penny (two-thirds of a cent) per ton mile of traffic this type has become uneconomic and another should be substituted. The average cost of maintenance in Midland is just about this limit; in Norfolk it is only half as much.

English experience indicates that 120,000 tons of traffic per yard width will wear out ordinary waterbound macadam in a year. At Sidcup a traffic of 506 tons a yard width a day wore out the macadam section in a year. It is believed there that the traffic limit at which this type becomes useless is 50,000 tons a yard width per annum, or 137 tons a yard width per day.

The French estimate the traffic limit of waterbound macadam to be 1,000 "collars" a day for the whole width of the road. A "collar" means a single horse-drawn loaded vehicle, and an automobile is figured as equivalent to three "collars."

It is evident from the figures above mentioned that the traffic limit of waterbound macadam is far from being definitely established, while a knowledge of it is highly important. The writer recommends that censuses be taken on all the state roads which seem to be yielding too rapidly to traffic in order to determine the traffic limits of our various types of roads under the climatic conditions in the different parts of our commonwealth. The writer also urges that traffic censuses be taken as well on roads about to be improved, as a guide to the type of construction to be adopted.

Construction and Varieties of Macadam Types.—The methods of construction of waterbound macadam prevailing with us are so well understood that they are not dwelt upon. But a few variations common abroad and certain special methods and details may be chronicled with profit.

Although macadam was originally a course of uniformly broken stone, without other foundation, nevertheless on the Continent one or another kind of a foundation is more frequently present than absent. In France, Tresaguet foundations of stones set on edge are commonly employed and highly recommended, and types of Telford or variations of macadam are common in England and elsewhere. On the Continent any road with a waterbound macadam surface is still called a macadam road, no matter what the foundation. The sturdy foundation may account for the high wearing power and long life of many European so-called macadam roads.

In regard to the broken stone aggregate, although macadam was, as is well known, originally one course of angular broken stone of uniform size, there is a European tendency to subdivide and make use of a lower layer of 2½-in. to 2¾-in. stone, with an upper layer of 1.6-in. to 2-in. stone, somewhat as that preferred in tar macadam.

The softer the stone, the larger the size which is recommended. Abroad, for example, the lower layer in such a road may be 2¾ ins. if soft stone or 2.4 ins. if hard stone. European road builders generally believe that stones of a uniform texture are superior to those which are part hard and part soft. For this type of roads limestone as used by us is largely employed by the French, who consider that it binds well and stands fast, light traffic well, but admit it does not stand up under heavy crushing vehicles or too great motor traffic.

In Europe it is almost everywhere maintained by engineers that round gravel does not make a good road. The Italians, however, have found that broken gravel is satisfactory if not of too varied a texture, and use it extensively. Many of the English authorities believe that selected slag is more

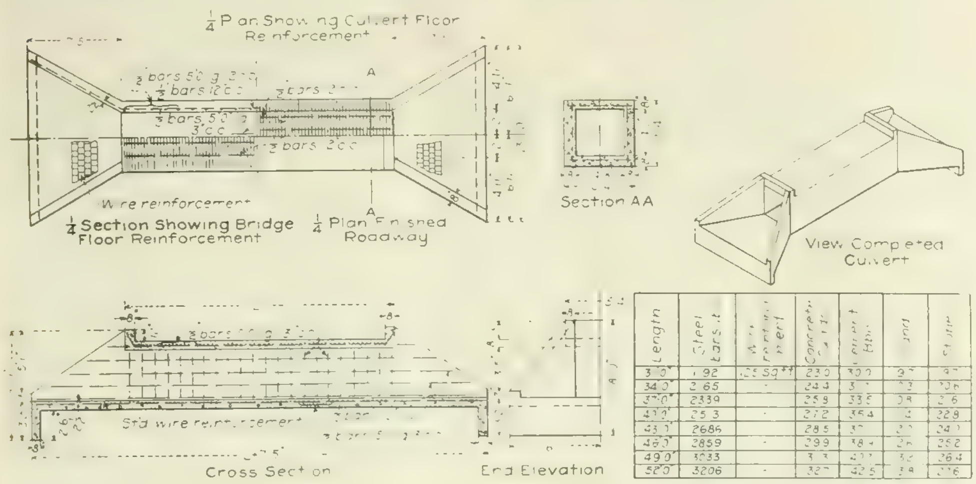


Fig. 13. Standard Type of Concrete Culvert with Paved Bottom.

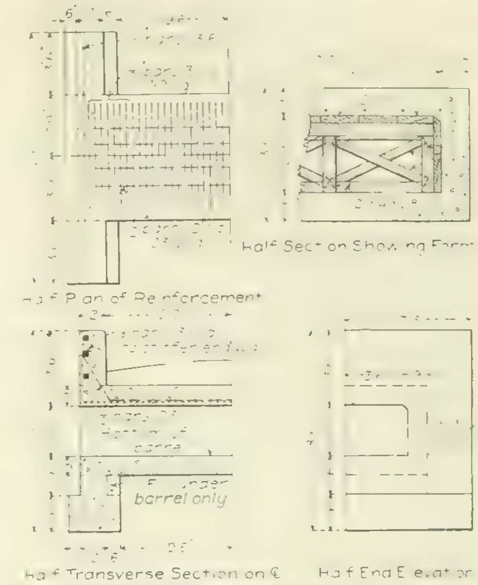


Fig. 14. Type of Slab Culvert Used When Head Room Is Limited.

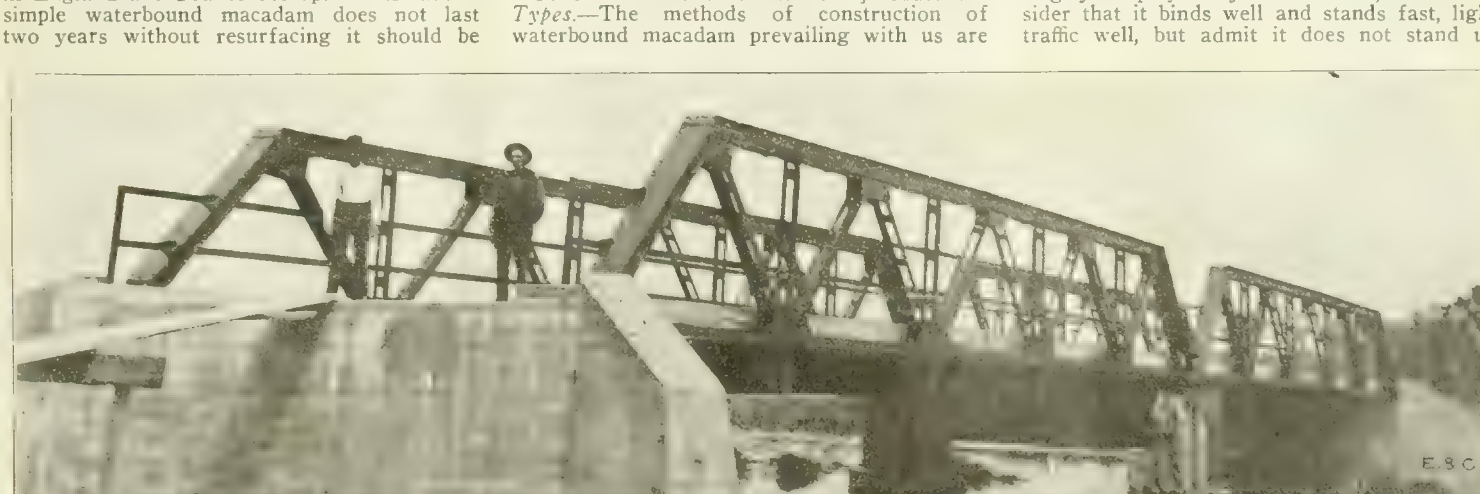


Fig. 15. Typical Riveted Low Truss Steel Bridge.

valuable than even granite for road material, as it crushes more under the roller and it is better molded under traffic pressure, so as more completely to fill in the interstices and become interlocked and welded into a solid mass.

PRESERVATION OF WATERBOUND MACADAM.

When waterbound macadam becomes too dusty or shows signs of giving out the first question is whether it can be preserved, or must be superseded by another type of construction. Examination of the road itself and a census of the traffic it bears are useful to give data to determine this. Bituminous treatment of several varieties may be applied to advantage. The chief forms of dust layers and surface preservers are many kinds of tar or tarry oils and calcium chloride.

Calcium Chloride.—Calcium chloride is recognized to be a useful preventive of dust in summer, but after test in England was rejected by a majority of the county engineers who tried it, as it produced an extra amount of sticky mud in wet weather.

The writer believes that this should not be applied save where the road is naturally dry or sandy.

Superficial Tarring.—The standard preservative of macadam is some form of surface tarring. Inasmuch as we have thousands of miles of waterbound macadam in the United States, the result of tests of this method elsewhere is important.

Before tarring the road should be repaired, leveled, freed from caked mud and well brushed. The tarring should be done while the road is dry. If a heavy tar is used the road should be well warmed, either by the sun's rays or otherwise. The tar should be applied uniformly by machinery and under pressure. It should be well brushed after application. For a first coating the English use one imperial gallon of tar to from five to seven square yards of surface.

Cost of Superficial Tarring.—The cost of such surface tarring in England runs from one and one-third cents for crude tar to four cents for distilled tar per square yard, or, taking as a unit a mile of 16-foot road, the cost would be $125 to $375 per mile. The cost in France is about two cents a square yard. The cost is greater here, where more tar is used per square yard, although with doubtful advantage.

The tarring under moderately heavy surface usually lasts only one year, but greatly increases the life of the macadam, and is cheap in proportion to the cost of resurfacing the macadam, which they estimate at twenty cents a square yard for a two-inch top. In some cases it may retain effectiveness from two to five years.

Increasing the Life of Roads.—The English experience indicates that tar surfacing increases the total life of waterbound macadam from 120,000 tons to 240,000 tons per yard width, a gain of 100 per cent.

In Germany nineteen road authorities stated that surface tarring increased the life of the road a full year; forty-six other authorities stated that it reduced the wear on the roads, and twenty-six thought it made little difference, but in some of these cases crude and not refined gas house or distilled tar was used.

The French estimate that surface tarring increases the life of the roads by 50 per cent or more. Surface tarring of the Bois de Bologne, the most traveled pleasure street in Paris and the one leading to the park, has reduced the maintenance expense from 2.10 to 1.32 francs a square meter. Thousands of automobiles, taxis and horse-drawn vehicles use this boulevard daily and, though heavy commercial vehicles are forbidden, it stands very well the traffic of thousands of heavy iron-tired vehicles which cross at street intersections.

Annual superficial tarring prolonged the life of one road in France to eight years, whereas an adjacent untarred section wore out in three years. The [illegible] that surface tarring is very effective to prevent dust and to preserve the road where it is being worn through and torn up by fast traffic. Superficial tarring alone does not seem effective against very heavy crushing traffic, which apparently wears out or crushes the road underneath the tar film.

It is recommended that superficial tarring be employed upon waterbound macadam surfaces where they wear out in from one to four years, or where a traffic census shows a traffic of from 80 to 300 tons per yard width a day. And also that roads be thus built economically in many cases for light traffic and light motor traffic. Many roads of macadam should be given a bituminous carpet if possible and should thus give excellent service.

The Traffic Limit of Superficial Tarring.—The point at which superficial tarring ceases to be economic is far from final determination. A German estimate of the economic traffic limit a day of superficially tarred macadam is 400 single horse-drawn vehicles, 40 automobiles and 20 heavy motor wagons, which would be equivalent to only about 640 "collars" per day. How long they expect it to endure this traffic is not given. This is much less, however, than either the French or one English estimate, which, as stated, is a total life of 240,000 tons per yard width.

At Sidcup superficial tarring showed a probable total life of 18 months for a traffic of 506 tons per yard width per day. This would imply that a surface superficially tarred annually would stand for two years, which is the economic minimum, under a traffic of 377 tons a yard width per day, but this is many times the traffic on our ordinary country roads.

On the other hand, Mr. Dryland of Surrey considers a more permanent type more economical in the end as soon as the traffic reaches 20,000 tons a yard width a year. These estimates, however, differ so materially that the writer recommends that a census be taken on our own surface tarred roads to determine the traffic limit of superficial tarring.

MORE PERMANENT TYPES OF CONSTRUCTION.

Where the traffic is too heavy for superficially tarred waterbound macadam more permanent types of road become necessary. Besides asphalt, granite, brick and small stone set paving, the expense of which confines them largely to cities in Europe, the two types most highly considered at present are concrete and bituminous or tarry bound macadam roads.

Concrete Highways.—Europe is far behind the United States in experiments with Portland cement bound concrete roads. Abroad objections are made to concrete roads as being too hard, insufficiently resilient and too liable to crack from the weather.

Where tried abroad, they have as yet found no certain method of giving them a resilient, waterproof surface which will stick and endure. In fact, their experiments point out quite clearly to them the necessity of such a surface. The author believes that special tests should be made to find a surfacing for concrete with these necessary qualities. As regards concrete roads in general, he believes their many advantages and best types are to be seen rather in the examples in this country than abroad.

Bituminous Macadam.—The English experience with bituminous or tar bound macadam dates back on certain roads over thirty years. In general this type is considered satisfactory to meet the traffic within the limits where waterbound macadam, even when surface tarred, fails, and where traffic becomes so intense as to require asphalt or paving.

Two distinct methods of construction have been developed, namely, grouting and mixing.

Bituminous Grouted Roads.—Their grouting method consists, in brief, of laying a 2 or 3-in. crust of 1¼-in. to 2½-in. broken stone, rolling, and while the surface is dry pouring hot pitch, softened with oils, and sometimes mixed with an equal part of hot sand, into and filling the interstices. This crust is closed with a surface of ½-in. to ¾-in. chippings, rolled and firmly compacted. The cost in England is about 72 cts. a square yard.

While Engineer Brodie of Liverpool has attained good results with grouting for twelve years, it is admitted to require great care and has often failed, even in England. France reports adversely on grouting and Germany considers its results doubtful.

The fact seems to be that such pains are required completely to fill the interstices between the stones, on account of the difference in temperature in different hours of even the same day, or of the skill and experience of the laborer doing the pouring, that hand grouting can be entrusted to no one who is not personally interested in the life of the road. The writer is not disposed to recommend it for use under contract.

The Mixing Method.—The other principal method in vogue is the mixing process, many varieties of which are employed.

In England and Switzerland the mixing method is highly commended. It consists in general of heating granite, hard stone or slag until all moisture is driven off and mixing it with hot tar at the same temperature, usually 225 to 350 degrees.

What is designated as the Aeberli system of mixing heated stones with crude tar, without a filler, and allowing the stones to lie in piles for the tar to set before laying on the roads, is frequently used on the Continent. Although employed in a number of places with good results, the writer finds that there has been such a large percentage of failures that he believes this method to be too uncertain to be recommended. The fault seems to lie in using too light tar without a filler and in letting the mixture cool before laying.

The Praed-Hines system, which has given very good results, adds hot coarse sand and some hydraulic cement to the hot tar while mixing, and mixes all at an even heat in a simple and mobile machine.

In the Tarmac system, where hot selected slag is used, only a little filler is employed, but the Tarmac process is largely used and giving excellent results at reasonable cost. It is a hot mix at the plant but laid cold when away from the vicinity of the plant. The fact seems to be that granite or hard stone requires a filler in the tar, whereas slag, to which the tar clings better and which is also more readily further compressed by traffic, filling up the interstices, does not require such a filler.

The writer considers the two methods last mentioned of preparing bituminous macadam material as the best he found in Europe. The advantage, if not necessity, of thoroughly heating and drying the stone aggregate before coating seems established logically and by successful practice. A lower 2½-in. course of 2½-in. stone or slag is laid, over which is added a second 1½-in. layer or cover coat of 1½-in. stone or slag finally surfaced with chippings. The cost in England is from 70 cts. to $1 a square yard, and it is frequently guaranteed for five years under heavy traffic at these prices.

Efficiency of Bituminous Macadam.—The efficiency of bituminous and tar macadam seems well established by many examples which have been under hard wear for years in England. There are a number of tar macadam roads which have endured, with only occasional resurfacings, from 30 to 35 years. One estimate of the life of pitch macadam is a total of 1,320,000 tons per yard width, or of 11 years with a traffic of 120,000 tons per yard width each year.

In Sheffield a tar macadam road with a traffic of 36,000 tons per yard width per annum has been down for 21 years at an average annual cost, including the original expenditure, of 8 cts. a square yard. They claim it can be maintained indefinitely for 6 cts. a square yard a year.

Another tar macadam road with a traffic of 70,000 tons a yard width a year has been laid seven years and can be maintained indefinitely for 12 cts. a square yard a year, and this under very trying traffic.

The British contention is that where a superficially tarred waterbound macadam road lasts only two years before it requires retarring, it will be more economic to substitute tar macadam or concrete. If waterbound macadam lasts three years with surface tarring, tar macadam will still apparently be more economic in the end.

The traffic limit at which tar macadam seems to become uneconomic may be estimated as high as 1,000 tons a yard width per day, at which rate the top layer would probably wear out in four years. This is a very heavy traffic, and five or ten times that on our ordinary country or rural road. For such very light traffic roads it seems unnecessary to dwell extensively upon any types of construction more expensive than concrete and bituminous bound macadam, or even carpeted waterbound macadam.

ECONOMIC FEATURES

Good roads will solve a great economic problem, at present found in the decadence of the small farm, which is due in a large measure to the rebellion of the rising farming young folk against the isolation caused by impassable roads, which are too muddy to be traversed by wheel and not sufficiently liquid for navigation. In better roads lies the hope of the renaissance of agriculture.

We are on the eve of an era of road building comparable only to railway building between 1840 and 1880. In the writer's judgment, we will be able to obtain roads suitable for a moderate amount of traffic at a reasonable cost. We need not be driven always to the most costly types of road, the building of which would tremendously reduce the new mileage possible under reasonable appropriations, for to the fullest economic extent we should seek to extend the greatest good in the way of roads to the greatest number of people.

Fixed Charges on Roads.—The fixed charges on the most expensive types of roads in the way of interest and sinking fund make them more costly and sometimes less economical in the end than the cheaper road with a moderate cost of maintenance. For instance, a road costing $14,000 should be capably maintained under the English system of constant care for not to exceed $200 a mile per annum.

On the other hand, a road costing $24,000 or more would necessitate an interest charge on the $10,000 extra cost, which at 4½ per cent would mean $450 a year. To this must be added a sinking fund of 1 per cent, which would make $550, as well as further expense for maintenance, say $50 per annum, making $600 as the annual excess upkeep cost per mile of highway.

Roads Will Wear.—Road builders have to bring their public to understand, and in fairness to them the people should realize, that nothing in this world can be used without wear. Even a steel rail upon a railroad has an average life of only about ten years.

The maintenance of some of the more expensive types of roads is much like that of steel rails, which, when the surface is worn, must be entirely replaced, whereas some types of roads which have a renewable or replaceable top need not be entirely reconstructed, but merely resurfaced or repainted, as it were, at necessary intervals.

The Advantages of a Readily Replaceable Road.—If a railroad could find a type of rail the surface of which, when worn, could be made substantially as good as new without removing it from its place, it would consider it an economic advantage from both the fixed charge and maintenance standpoints.

We should congratulate ourselves that in the past few years, since the advent of the new traffic conditions to which the motor vehicles have subjected our roads, we have developed types whereon not the entire road, but merely the surface actually worn, needs to be repaired, and in which we can save our base and confine our maintenance to the limited zone of actual wear. It fact, maintenance should be so handled and constant that wear should never be allowed to come upon or reach the second course or foundation of any road.

Brick, concrete, bituminous macadam and durax will play important parts in the roads of the future and each will have its sphere of proper usefulness.

The Effect of Leaking Illuminating Gas on Bituminous Pavements.

An important question in connection with the life of bituminous pavements is the effect of leaking illuminating gas upon the composition of the pavement. In a paper before the recent meeting of the American Society of Municipal Improvements George C. Warren discussed that subject and his paper is given here in part:

To everyone who has had long experience in bituminous pavement construction and maintenance it is well known that illuminating gas escaping from subsurface gas mains is very injurious to all forms of bituminous pavements, and yet we frequently find representatives of gas companies and city officials ridiculing the idea that gas can have any effect on such pavement surfaces.

The fact is that illuminating gas, a bituminous substance, and hydro-carbon, quite similar in its chemical composition to light tar and oil distillates, is lighter and more penetrating than the distillates and even more rapidly attacks and liquefies a bituminous cement, no matter whether the bitumen be a tar or an asphaltic product.

In my thirty years of experience with asphalt and other bituminous pavement construction this matter has come up many times, and generally the persistent claim of the gas companies is:

1. That it is ridiculous to suppose that gas will attack an asphalt pavement.

2. That the gas mains have been carefully tested by plugging holes through the pavement and into the ground and that there are no leaks.

3. When the main is on the side of the street opposite a portion of the street affected, it is too far away.

I would comment on these points as follows:

1. There can be no question but that coal gas, being a hydro-carbon in gaseous, and therefore its most penetrating form, will attack any form of bitumen even more rapidly than oil, just as gasoline, being more penetrating, will attack bitumen more rapidly than heavier oil, such as machine oil, for instance.

I particularly remember one extremely aggravated case on West End Avenue, New York City, about fifteen years ago, which became reaffected, and the asphalt pavement again ruined within three months after it was resurfaced, the gas company having claimed that they had thoroughly tested the mains and repaired the leaks. Shortly, subsequent to this, the gas company was obliged to entirely relay the gas main and the pavement was again relaid.

2. Even if they dislike to, or will not admit it, the gas companies well know that the "plugging" test only sometimes locates the leaks. They naturally dislike to go to the expense of tearing up and relaying pavements and excavating to and uncovering the mains unless the conditions become such that they are absolutely required to do so.

3. The leaking gas naturally follows the lines of least resistance, which may be in either a nearly vertical line and show the result almost immediately above the leak, or, what is extremely hard to find, may follow a long distance through a vein of porous earth or along the space formed by some old settled trench and reach the pavement surface a long distance from the leak. In such cases the pavement is generally affected over a large area, because the earth has become quite generally saturated with the gas, which gradually works up to the pavement surface.

I remember a case in about 1888, on Rutger St., Utica, N. Y., where the first indication of trouble in the gas main was the effect on the asphalt pavement. The gas company claimed they had repaired the leak and a considerable area of pavement was relaid. About a month later an elderly couple, while asleep in one of the residences nearly a block away, was nearly asphyxiated by gas leaking from the street. Probably the gas had first found a partial outlet, following along a vacancy in some trench settlement, and then through the pavement. Subsequently the leak became worse, with the serious result referred to above.

Frequently, if not generally, the first indication of leak in the gas main where the street has a bituminous pavement is the effect on the pavement, in which case it is good fortune that the pavement is of a character which is affected by gas and will permit the gas to permeate the surface, thus lessening the chance of serious casualty as above outlined in Utica.

The visible effect of leaking gas on a bituminous pavement is a serious "shifting" or "rolling" of the pavement in its softened condition, accompanied by a breaking up of the surface into a "crackled" appearance not unlike the folds or cracks in an alligator's back.

Generally, when this condition is noticed, a perceptible odor of gas will be found in the pavement surface, but sometimes the leak may have been repaired or the gas taken another course and the gas escaped so that its odor cannot be detected, yet the pavement is left in a seriously damaged condition. On the other hand, under certain subsoil conditions, the earth below the pavement may retain the escaped gas and continue to have its damaging effect for months, if not years, after the gas main has been repaired and the cause of the trouble probably removed.

The effect on the pavement may extend for a considerable period beyond the repairs to the gas main unless steps are taken to provide frequent vents, left open for several days, if not weeks, after repairs to the gas mains are made.

In such cases, before making repairs to the pavement, its entire surface over the main and where the surface shows the effect of gas should be removed and the gas main should be thoroughly repaired (renewed if necessary). After the leaks are repaired vents or openings at least 1 ft. square, extending from the surface to the level of the gas main, should be left open for two or three weeks or longer, if there is still any odor of gas, and the openings then refilled and thoroughly tamped and the pavement surface relaid. With this precaution the trouble will probably be overcome if the gas company has not been too parsimonious, inefficient or incomplete in making of repairs to the gas mains.

In conclusion it may be well to call attention to a pernicious but quite general custom of the gas companies in their "hunt" for leaks. I refer to the custom of having two or three men, one of them selected for his "smelling" efficiency, go along the street where there are indications of gas leaks, hammering a cone-shaped bar through the pavement into the ground below. These holes are made at intervals of a few feet. The "smeller" man puts his nose to the holes, and if he discovers a sufficient odor of gas to attract his attention an opening is dug to try to discover the leak. If the "smell" test does not locate a leak apparently below the hole it is quietly filled with dirt. A few weeks later the city official, or the "poor devil" of a paving contractor, if he happens to have the pavement under "guaranty," finds serious holes along the center line of the pavement and, not being able to trace the trouble to any cause, he makes the repair, if his "guaranty" is good, or "lets it slide" if he and his "guaranty" are no good, and perhaps at the end of a lawsuit the city makes the repairs.

The simple, but so far as the writer knows never enacted, cure for this evil is an ordinance requiring gas companies, whenever they want to test their mains, to cut openings of sufficient size through the pavement to enable excavation to the gas main, absolutely prohibiting the promiscuous drilling holes through the pavement as above described, and to pay for making a proper repair to the pavement. With the enactment and enforcement of such ordinances the chances of locating the leaks are greatly increased and the unfairness to the city and contractor of making holes in the pavement without adequately repairing them will be eliminated.

BUILDINGS

The Design and Operation of the New Shop for Building Steel Freight and Passenger Cars of the Canadian Pacific Ry., at Montreal, Que.

The Canadian Pacific Ry. has recently constructed, at its Angus plant, Montreal, Que., a shop for the construction of steel passenger and freight cars having a capacity of 10 passenger cars per month and 8 freight cars per day. Before the construction of this shop thousands of steel frame cars had been built by different firms in the United States and in Canada for this company to the same design as the cars for which the new shop is intended. The new shop, which is divided into two parts, one for the construction of freight cars and the other for passenger cars, contains excellent facilities for the rapid and economic construction of steel cars. The following article on the layout and operation of the shop is based on a paper by L. C. Ord, assistant master car builder, Canadian Pacific Ry., in the Proceedings of the Canadian Society of Civil Engineers. The original article gave few data on [illegible] Canadian Pacific Ry., for the structural drawings shown in this article and to Mr. Ord for [illegible] cars have the floors, the lining and the roofs [illegible] all-steel roofs. The existing large blacksmith [illegible] be handled in the new shop, the rivet furnaces being the only ones required in it.

As it was anticipated that the greater number of the cars built would be steel frame box cars, the design and layout of the freight shop was considered solely with reference to this type of car in order that the work might be done as cheaply as possible. In designing the shop 2,750 sq. ft. of floor area were allowed per car per day (total under cover) as the average for the existing shops. To prevent overcrowding, which is common in most steel freight car shops, and to allow for the greater amount of space taken up by the design of spacing punches, a larger amount of machine space was provided. The floor area finally adopted for the freight shop was 41,785 sq. ft., the area of the machine shop being 22,069 sq. ft. less 7,265 sq. ft. (which was set apart for machining and assembling steel center sills on repair work), giving a total area of 14,795 sq. ft. available for machines. The area of the assembling portion of the freight shop is 9,170 sq. ft., while the erecting area is 17,820 sq. ft.

DESIGN FEATURES OF SHOP.

The steel shop proper consists of two 100-ft. bays running parallel with the front of the shop, and at right angles to this part one 72-ft. bay, 405 ft. long (see Fig. 1). The erecting section of the passenger shop is composed of four 27-ft. 6-in. bays 202 ft. 6 ins. long, running at right angles to the 100-ft. bays and parallel to the freight section.

The crane service comprises a 10-ton traveling crane of 96-ft. 3-in. span, on a runaway 309 ft. long, which is located in front of the shop. This crane serves the materials section, and is parallel to the cranes in the shop. Inside the shop there is a 10-ton crane of 96-ft. 3-in. span in each of the 100-ft. bays. These cranes all have a head room of 27 ft.

In the erecting section on the freight car side there is a crane of 10 tons capacity with a span of 67 ft. 3 ins., and a head room of 35 ft. 6 ins. In the passenger shop four traveling cranes of 24-ft. 10-in. span and of two tons capacity, with a 20 ft. head room are used. They are employed for the handling of material for the passenger cars and are operated from the ground. A noticeable feature is that these runways are carried into the main shop for a distance of about 8 ft., under-running the overhead traveling crane to assist in the transfer of materials from one crane to another.

[illegible] and brick [illegible] a large amount of light, the window area being approximately 30 per cent of the total wall area.

Figure 2 shows a framing plan of the shop and sections through various parts of it. These drawings show the varied types of construction adopted for different parts of the shop, the general dimensions, and the sizes of members. A study of these drawings will give a clear idea of the care which has been given to the design of the various sections of the shop to facilitate the economic construction of steel passenger and freight cars.

The shop floor consists of two ⅝-in. layers of an asphalt mastic (a wearing layer and a cushion) laid on a 4-in. concrete base. After considerable use of this type of floor in various parts of the plant a floor of harder consistency than usual was selected, and although marks will occur if heavy weights are left on it for some time, yet when these weights are removed the marks gradually work out. This floor does not crack nor break if anything is dropped on it, as in the case of cement. It is also much easier for walking or working on; it is waterproof; and is easily kept clean and free from dust.

The crane runaway on the midway runs the full length of the plant and extends far enough in front of the steel car shop to load and unload materials from the first two material tracks running through the storage space. This space is so arranged that, should it be found advisable to extend the shop in order to afford a cover for the material (to provide for additional machine space or for any other purpose), the crane runway can be transferred to the opposite side of the midway and still be used for the storage of material.

Supply tracks run in an east-and-west direction through the storage space at intervals suited to the arrangement of the machines inside and to the storage of the material outside. Through tracks for the handling of loaded freight cars are located at the north of the bay for passenger car material and at the south for material for freight cars. The material is unloaded direct from the cars to the proper piling space, which is conveniently located with relation to the supply tracks. With this arrangement the handling of mate-

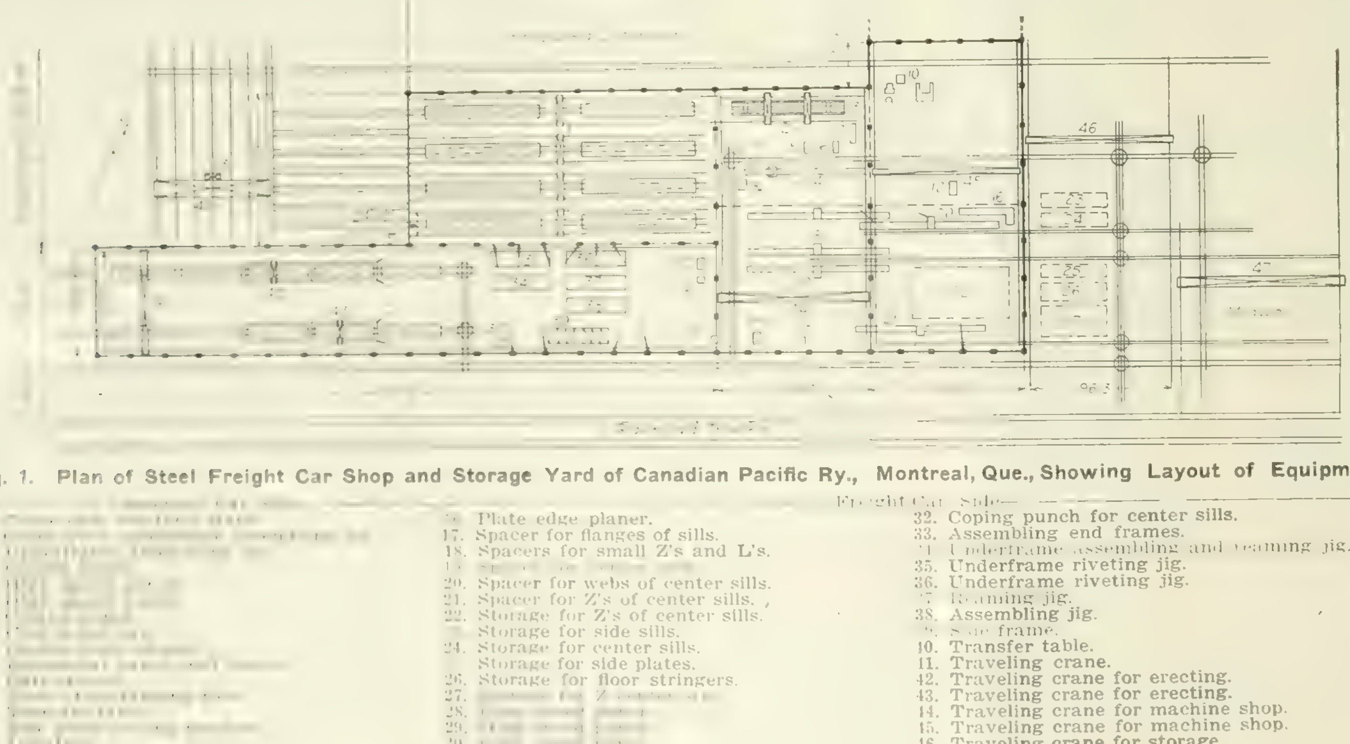


Fig. 1. Plan of Steel Freight Car Shop and Storage Yard of Canadian Pacific Ry., Montreal, Que., Showing Layout of Equipment.

[illegible]

16. Plate edge planer.
17. Spacer for flanges of sills.
18. Spacers for small Z's and L's.
19. [illegible]
20. Spacer for webs of center sills.
21. Spacer for Z's of center sills.
22. Storage for Z's of center sills.
23. Storage for side sills.
24. Storage for center sills.
25. Storage for side plates.
26. Storage for floor stringers.
27. [illegible]
28. [illegible]
29. [illegible]
30. [illegible]
31. Coping punch.

Freight Car Side—
32. Coping punch for center sills.
33. Assembling end frames.
34. Underframe assembling and reaming jig.
35. Underframe riveting jig.
36. Underframe riveting jig.
37. Reaming jig.
38. Assembling jig.
39. Side frame.
40. Transfer table.
41. Traveling crane.
42. Traveling crane for erecting.
43. Traveling crane for erecting.
44. Traveling crane for machine shop.
45. Traveling crane for machine shop.
46. Traveling crane for storage.
47. Traveling crane for midway.

rial is reduced to a minimum and the supply to the shops is for the most part independent of the overhead traveling cranes, which are

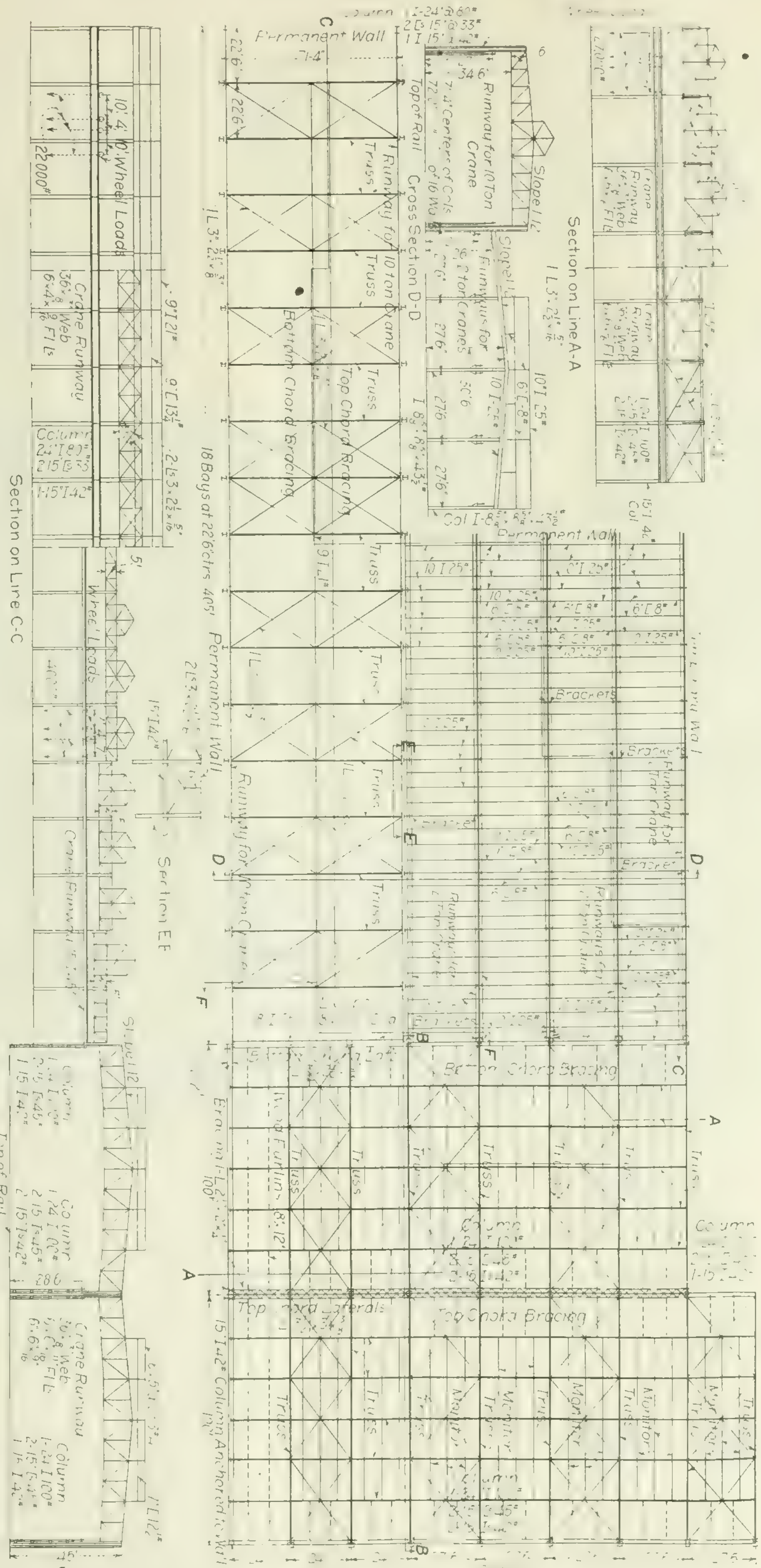


Fig. 2. Structural Framing Plan and Sections of Steel Car Shop of Canadian Pacific Ry., Montreal, Que.—Note Different Types of Construction Used.

used principally for unloading cars and for handling large quantities of specially heavy material.

In the front portion of the steel shop there are two long bays each 100 ft. wide (see Fig. 1). The first bay is 209 ft. 6 ins. and the second, 182 ft. in length, one bay being longer than the other in order to allow for unloading, directly into the shop, any material which it is necessary to keep under cover.

The arrangement of the two long bays with traveling cranes running crosswise of the tracks in the shop was particularly suited to the spacing tables and types of machines installed. The crane in the front section was used to supply the material to the machines, through which it was carried automatically into the next bay where the second crane distributed it from the machines to the various points in the shop.

Figure 3 (P.408) shows details of a part of the shop, indicating the roof and side construction, and framing details. This drawing is a section taken along *F-F*, Fig. 2.

Figure 4 (P.408) shows details of the type of monitor outline in Fig. 3.

An effort was made in the layout of the shop to use machines with relatively small capacities, but sufficient in number to prevent the expense and delay of changing dies and setting up and the large accumulation of material necessary to feed the shop without delay, which is required when one large machine is used for several purposes.

Steel Freight Car Shop.

In the freight shop four spacing punches are used. One of these is fitted especially for punching 6-in. Z-bars for the steel center sills used for repairs on the line, and it is not concerned in any way with the steel car work. This machine is fed by an independent lorry track. The Z-bars require to be punched twice, once for the flanges and once for the webs. The material is run in beyond the punch and is handled through it to the front of the shop. Specially arranged air jacks are used for lifting the Z-bars to place on the rollers which feed the machine, and also for unloading the punched Z-bars from the machine. Skids are so arranged that the Z-bars are lifted and skidded from the machine without being touched by hand. It has been found cheaper and quicker to run a quantity of bars through in one operation, and when a considerable number have been punched to move them back for the second operation, which of course requires a change in the set-up of the machine. On the last operation, instead of unloading at the material space at the side of the machine, the Z-bars are skidded in the opposite direction onto the rails where they are required for assembling for draft gear, etc. From these skids they are slid onto the trucks, which take them to the front of the shop where the couplers and draft gear are fitted. They are then either loaded directly on cars for shipment or are used in the freight car shop where heavy repairs to the existing equipment are made. The second spacing punch is used for the side plate of the car, which is a 4-in. Z, and for the floor stringers, which are 3-in. Z's. The capacity of this machine is well above the present requirements.

One spacing table is used for punching the webs of channels (center sills 15-in. and side sills 8-in.), while another machine punches the flanges of these channels. The machine which punches the webs of the channels does the work in one operation and has, therefore, double the capacity of the flange machine. With the present output of the shop the web punching machine is used on the above work for only one-half of the time, it being employed for the remainder of the time on cover plate and similar flat work. An additional punch for the flanges has been installed and the foundation for the spacing table has been laid out so that without disarranging the handling of the material (by adding a spacing table to the existing punch) it will be possible to double the output of these machines, the spare punch being used for hand work at the present time. By providing a spacing table on this coping punch and an extra coping punch, practically no other additions would be necessary to increase the machine capacity of the shop to 25 cars per day

In the event of a breakdown of any of the punches employed on this spacing table work, serious delay to the output could not possibly be avoided. As additional heavy punches for coping, slotting, etc., were necessary it was arranged to purchase machines which are duplicates of those used on the spacing tables. If at the present time any one of the punches on the spacing tables should become totally disabled, it would be possible to substitute another punch, or ... without brake pipe. These lots are made on a coping punch, which is so arranged as to require no backward movement of the material. The the exception of the diaphragms which are punched on a horizontal machine.

This completes the machine equipment for

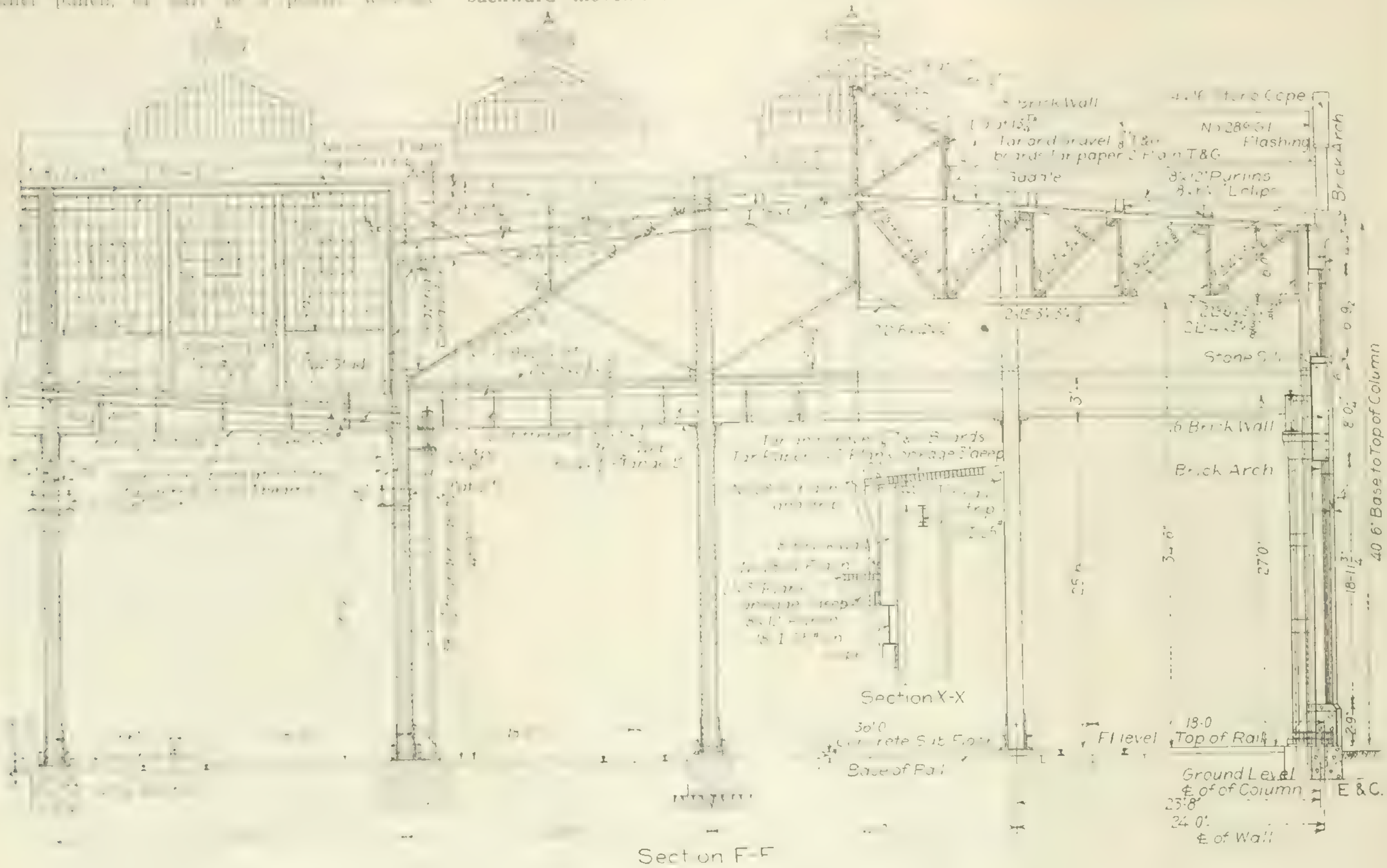


Fig. 3. Details of a Part of Car Shop of Canadian Pacific Ry., Montreal, Que.—Drawing Represents Section F-F of Fig. 2.

much delay and to keep the shop running, as in the case of punches for coping and similar work, substitutions could readily be made. On all the machines used in the shops the interchange of punches, gags and other jigs have been closely considered to prevent delay or the necessity of keeping on hand large stocks.

The piling space outside of the shop is arranged so that the material can be skidded onto the lorries by hand and taken to the spacing punches and other punches by hand, which in case of trouble with the overhead crane saves a great deal of time. Care was taken in the case of the heavy material, such as the center and side sills, to arrange that each movement should be short and easily handled. The first movement from the pile is to the web punching machine, from which the sills are handled by small cranes and rollers to the flange punching machine without any backward movement. On the flange punching machine, the sills are bolted together in pairs; they are then run through the machine; are brought back through the machine; [illegible] second time to punch the lower flanges.

The design of the spacing tables is par- [illegible] has two gage lines, and therefore the punch- [illegible] from th[illegible] the last movement on [illegible] machine the ma- [illegible] the floor, where a [illegible] and side sills, to pr[illegible] of delay on these very [illegible]

The small track on [illegible] center sills are taken [illegible] used to supply the [illegible] spacing machines. [illegible] arrangement prevents [illegible] be inevitable if all the [illegible] over one supply track.

[illegible] and flanges also require slots in them to accommodate the [illegible] and die of this punch is left set up on the machine all the time, but it is so arranged that small dies can be set up beside the large ones, to enable the machine to be used for light punching when not in use for punching slots. An-

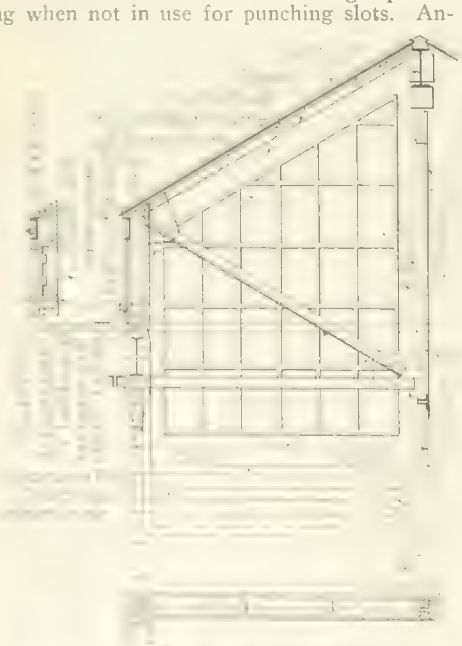


Fig. 4. Details of Monitor Used on Car Shop of Canadian Pacific Ry.—Positions of Monitor Shown in Fig. 3.

[illegible] for mak- ing the various mitered cuts and for coping the side posts and braces. The coping punch near the Z-bar machine is fitted with a special die for coping the side plates. Three high speed punches are used for all the various [illegible] with the steel shop, but it should be borne in mind that the hot forging, the upsetting and the bending work is done in the blacksmith shop, the material being brought into the steel shop already finished.

Special attention has been given to the handling of material to make this, as far as possible, independent of the overhead cranes, except for the movement of large quantities of heavy material. For this reason it will be noted that the shop is particularly well equipped with air jacks, skids, overhead fixed hoists, traveling hoists on runways and swinging jib cranes. To reduce the labor and the cost of handling and of repairing, ball bearings and roller bearings are used throughout on jibs, hoists, hand traveling cranes and material rollers. Special care has been taken to have definite space alloted for the piling of material outside the shop, for the storage of material around the various machines, and for the storage and accommodation of the finished material. Specially constructed racks are used throughout the shop. To maintain the orderly handling of the material, painted lines are used to define the boundaries of these piles and to mark the passage ways, which are always kept free of material. These boundary lines are repainted at the end of every week, at which time an absolute clean-up is made of any material which would otherwise tend accumulate.

The "Thomas" spacing tables are of the semi-automatic type. The movement of the carriage is controlled automatically, while that of the gags for bringing one or more punches into play at each stroke is controlled by hand. The tables are electrically operated, their movement being controlled by two templates, ⅞ in. x 3 ins., having a double row of steel pins. They are so arranged that when the trip on the moving portion of the table engages the pin it automatically stops and locks the material to be punched and at the same time, by means of an electric magnet, operates the clutch on the punch. The punch head, after coming down and punching the material, on its return stroke disengages the carriage and the clutch on the punch and automatically starts the movement of the material. The operation is repeated as each successive pin is encountered. The particular value of this type of machine is that templates may be made up of wooden strips with pins driven in, on very short notice and small cost, while permanent steel strips with inserted pins can be made up for permanent work. The strips can be changed in a very short time, and they are kept at hand for the various classes of material to be handled. The accuracy of the work practically depends upon that of the template, and the results obtained are very satisfactory.

The high speed punches were designed especially for this shop by John Bertram & Sons. They operate at the high speed of 60 strokes per minute and are entirely without gears, being belted from the motor direct to the flywheel of the punch. The clutch is of the six-point type; two punches are fitted in each head, both being controlled by a single gag lever which has three positions, one for each punch and a neutral position.

These high speed punches are not equipped with spacing tables, as it was found that, on account of the slower movement of the carriage on the spacing table and the consequent time lost in entering the piece, hand punching for small light pieces was cheaper, provided the same could be made sufficiently accurate. To do the punching more accurately the method adopted was that of using a drilled or punched template and of butting the piece against a gage inserted in each successive hole in the template. (The usual method is to make up a square template at one side of the machine, having one or more rows of holes punched in it on each of the four sides, some of which have as many as eight different rows of holes, each row being used as a template.) The material is moved along the top of the gage until it butts against a pin inserted in each hole in turn. It is found that for certain classes of work the gag is thrown in on the punch and the operator can move the material fast enough to catch every hole with the punch running at 60 strokes a minute. This is nearly three times as fast as the rate at which it would be punched on a spacing punch, while the time of setting up and the backward movement of the carriage is saved. Although only one piece is handled at a time the cost of punching light, short material per hole is fully as cheap as when done on the spacing punches. On some of the punches, where there is little variation in the work handled, single-sided templates are used with three or four rows of holes, which take care of the work satisfactorily. No marked-off work of any kind is used in the shop, except for irregular pieces which cannot be punched in the spacing punches or by the method described here, and most of the other work is punched to a gage, where it is possible to design one. This not only reduces the time in punching and saves the expense of marking off, but greatly increases the accuracy of the work done.

ASSEMBLING.

Special arrangements were made for the storing and handling of material, in relation to the assembling, to reduce the labor to a minimum. By means of underframes an important gain was made by using clamps instead of assembling bolts, which is the common practice elsewhere. (For an assembling bolt, it is necessary to get a full hole before the bolt can be applied; a wrench is required and the time spent in this way is greatly reduced by using a clamp with a hinged handle.) The clamp is applied between the holes to be reamed, and the time lost in removing the bolt from one hole and applying it in the adjacent hole, when reaming, is entirely avoided. The clamps are applied by the men who do the assembling. These clamps are not touched by the men who ream the material, but are removed by the riveters as they work up to the clamps. They are collected from the floor by a laborer and are carried back to the assembling positions for the succeeding frame.

In assembling the underframe a jig is used which accurately locates the center sills, bolsters and cross bearers. By this method the sills are assembled square, they are reamed in the same position, and are then transferred to the underframe riveting jig. This jig consists of a number of cast columns supported on I-beams, which are bolted to a concrete foundation. It securely holds the underframe in position while being riveted so that the underframe is constructed accurately and square in every way (see Fig. 5). A great deal of time is thus saved in the assembling, and the line of the car when finished is greatly improved. To rivet the underframe on the jig by compression riveters without turning it over, it was necessary to have a special type of riveter designed with a thin nose to permit the top row of rivets to be driven and to allow sufficient clearance for the bottom row, particularly on the bolsters, to be driven without moving the underframe. The makers considered that this would be an expensive type of riveter to maintain, but quite the reverse is the case, for even though a heavy block of cast steel is used for the top die, a small high speed steel insert is used. It is not possible to use a high speed steel snap for ordinary work, as it is extremely liable to break, but when it is inserted in the cast steel block it is well supported and does not fracture. When renewed, the amount of steel is so small that the cost is not considerable, and it has easily proved to be the cheapest type of die for maintenance of any compression riveter in the shops.

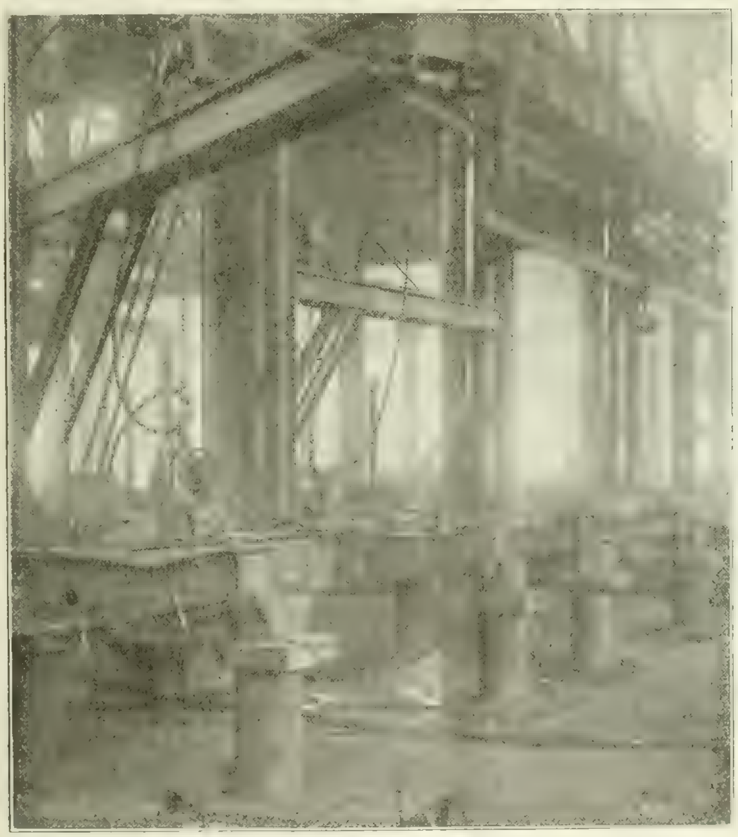


Fig. 5. View of Portion of Jig and Equipment for Constructing Underframes of Steel Cars—Canadian Pacific Ry. Car Shop.

Fig. 6. View of Portion of Erecting Section of Steel Freight Car Shop of Canadian Pacific Ry.

The movement of the steel sills from the point of assembling to the jig where they are riveted together is to be handled entirely by small air jacks running on trucks on narrow-gage rails, which move the underframe from position to position without requiring the use of the overhead crane. This arrangement saves considerable time and enables the movements to occur simultaneously, which would not be possible if it were necessary for the crane to move them all, as the crane can only handle one at a time. The only portion of the output which is fixed is that of riveting the underframe with the compression riveters, and the capacity for this work is about 14 or 15 cars per day. Provision is therefore made for the installation of an additional position for riveting the underframe along the side wall of the building where the jib cranes for handling compression riveters can easily be located. The other positions can easily handle

25 cars a day, by the addition of more help where required.

For assembling the side frame a jig was designed in which a considerable improvement has been attained over former practice. The side frames, for as many cars as are required, are built on one jig, all the parts being placed in templates, giving them a fixed location. The riveting on the lower side-sill is handled by two suspended compression riveters, the balance of the riveting being done by hand. The parts are at present clamped together by hand clamps, which will shortly be replaced by fixed pneumatic clamps in order to hold them more rigidly and to avoid the time lost in applying and removing the hand clamps. Special attention has been paid to the loss of time in locating and applying clamps, and pockets are arranged in the frame so that everything is as close as possible to the point where it is to be used. Even more care is taken in arranging the material in the racks, as each post and brace is piled directly opposite to where it will be used in the frame. A small "Gantry" crane is being constructed to handle this material from the jig to the piling space, so that no time will be lost in waiting for the traveling crane.

The same arrangement is made for the end frames as for the side frames, a small traveling crane being used to handle the finished end frames from the assembling sections to the place where they are piled.

ERECTING.

The erecting is commenced at a point where the trucks are brought in from the supply tracks at the side of the shop and handled across turntables into the proper position. The underframe is then brought down by the traveling crane and placed on the trucks. In the first position the floor stringers are riveted in place and one end frame is lifted on by jib crane specially constructed for that purpose. When this is done the underframe is moved down to the second position, and the side frames are lifted by two special hoists from the convenient piles where they have been placed by the overhead crane, and the other end frame is lifted to place by a jib crane (see Fig. 6). In the third position the carlines and the riveting up on top of the car is completed on an overhead scaffolding, which allows the men to work conveniently. There is also an upper floor and special racks for the accommodation of the carlines and other material.

An all-metal roof, designed by the Canadian Pacific Ry. for these cars, requires special handling. The usual practice was to allow the cars to go out of the shop with the carlines riveted in place, while the roof sheets and various wooden parts of the roof were applied in the wood freight-car shop. As this roof is all steel with outside carlines, it was necessary to apply the roof sheets before the carlines, and therefore the roof was assembled and erected complete on the cars in the steel car shop in two positions, and the cars were turned out of the steel shop with the roofs complete in every respect except for the application of the wooden running boards. This is a very unusual feature, and the fact that the entire output of the steel shop could be handled in this way is evidence of how readily the roof and carlines are applied. Figure 1 shows the arrangement of upper scaffold platforms which were used for the application of the carlines of the ordinary roofs and for the convenient storage of carlines and roof sheets for the all-steel radial roof.

Space for a fifth erecting position is allowed, but this is not required with the present output of the shop. The cars are moved outside by a motor-driven car pull situated in the lower end of the shop. They are sent over to the wood shop for lining, roofing and painting; and are then reported for service.

Steel Passenger Car Shop.

The method used in handling material in the passenger shop is the same as for the steel freight shop. The arrangement of the machines is similar (see Fig. 1), except that it is not possible to get enough work of the same class, with the relatively small output, to use spacing tables to advantage. A coping punch is therefore fitted up with rollers for the convenient handling of long material, and the work is punched to gage in much the same way as on the smaller punches, although considerable marked-off work has been used up to the present time.

The operation of the high speed and other coping punches is also similar to those used in the freight shop. The additional machines comprise plate straightening rolls for straightening or bending plates, and a plate edge planer for the outside sheets of the coaches. A cold saw and metal band saw are used for cutting accurately such material as could not be handled to advantage in the shears. A large gate shear, a double angle shear, a combination horizontal punch, and a bending or straightening machine completes the ordinary machine equipment in the passenger shop.

As it has been decided to drill the side plates instead of punching them, a special drilling machine was installed for this work. This is of similar construction to a locomotive-frame slotting machine, the plates being laid out on a long table and a traveling head provided, the latter being fitted with a number of electric drills. The head of this drill is very light in order to make it easy to handle. Templates, with hardened steel bushes for drilling, are used as in ordinary good practice. By laying out several sheets, one on top of another, and drilling a number of holes at the same time, the drilling costs are reduced to a minimum and compare very favorably with the cost of punching in an automatic machine, besides being necessarily much more accurate. Moreover, the drilling does not buckle or distort the sheets or leave a "rag" on the lower edge, as in punching. This is very important for the side sheets of the car, where the rivets are small and where small errors in the location of the rivet holes are relatively serious. It is intended to handle the drilling of the long side members of the car in the same way as that of the side sheets.

The drilling of the small, side cover plates will also be handled on a jig with a small sensitive drill, and it is intended to use rollers and stops in order to minimize the cost as far as possible.

ASSEMBLING.

The same arrangement of an assembling jig is used in the passenger shop as was adopted in the freight shop, but the number of operations was reduced to one, and the assembling, reaming and riveting of the underframe are done in one position. On account of the much greater length of the jig and the fact that the wall of the adjacent shop is considered as a temporary wall (in view of the probable extension of the shop), it was impossible to use a jib crane of any type because of the headroom which it would require. A low crane runway was therefore adopted, with small traveling cranes to carry the riveters. With this arrangement the number of riveters needed can be increased indefinitely in proportion to the output of underframes required, which certainly could not be done in the case of jib cranes; it further enables the underframe to be lifted out in either direction. The underframe jig is located centrally in the shop, in view of the probable future extension.

ERECTING.

The noticeable features of the erecting portion of the passenger shop are the arrangement of small bays running lengthwise over each track, the use of small traveling cranes just large enough to handle material or riveters as required, and an unique arrangement of scaffold posts, short on one side so that the cranes can operate over them, but stiff enough to carry rivet furnaces for the roof work, men's tools, etc., or to serve for a backing for light drilling work on the side sheets or framing. Where the posts of the shop are located, these are made to do double duty, to provide for the adjustable scaffolds so necessary in erection work of this kind (see Fig. 7).

Special rivet furnaces, which are designed to prevent the pitting so common in the ordinary oil furnaces, are used. These muffle furnaces have given very good satisfaction, and while they are slow in heating up, the burning and pitting of rivets is very materially reduced. In the passenger sections there are four tracks, with two positions on each track. The output in the original layout of the shop was based on 10 cars per month. It was thought that each car would stand 10 days in each position and that the underframe equipment would turn out a car for every 2½ working days, but it is evident from the progress already made that a better output than this can readily be obtained. In the first position the posts, the end frame and the complete framework of the car are erected and the side roof sheets and hood sheets are applied. As the car leaves the first position it is run beyond the second postion to the outside of the shop, where it is sand blasted and then returned to the second position for finishing. In the second position the center roof sheets and the flooring, including the vestibule trimmings, etc., are applied. The car is then sent over to the wood passenger shop for the inside trimming and finishing. A transfer table is provided at the outside of the shop for switching the cars from track to track or to the out-going track, while the sand blasting at the

Fig. 7. View of Interior of Passenger Car Section of Canadian Pacific Steel Car Shops, Montreal, Que.

present time is done outside of the shop, between the shop and the transfer table.

Figure 1 shows the arrangement and equipment which are used in the erection portion of the passenger car shop. Special attention to the convenience of the men is shown in the temporary side deck platforms for working on the center deck, in the scaffold posts which serve as ladders, in the convenient rivet supply, and in the location of air hose connections.

Report of the Illinois State Board of Examiners of Architects.

The report of Francis M. Barton, secretary of the Illinois State Board of Examiners of Architects, which was presented before the Illinois State Convention of Licensed Architects, held in Chicago on Oct. 7, 1914, contains some interesting data. In this report the general policy of the Board is outlined, and the practical effects of the architects' license law are noted. As some of the statements contained in the report are of special interest to engineers—particularly structural engineers—a careful study of the report by all engineers is recommended. The full report, not essentially changed as to tone or form, is herewith given. Parts printed in italics have been given this emphasis by the editor; they did not so appear in the original report:

It gives me great pleasure to have this opportunity to inform you of the work of the present Board.

We have today 863 licensed architects; 410 architects who were admitted because they were practicing when the law went into effect, and 453 architects who passed examination by the Board. There have been given three regular class examinations since the last biennial report was made, as follows: Apr. 15, 16 and 17, 1913; Oct. 14, 15 and 16, 1913; Apr. 14, 15 and 16, 1914; and preparations are now being made for the coming fall examination, which will take place Oct. 20, 21 and 22, 1914, all at the University of Illinois, Urbana. There have also been held four special examinations under the provisions of the "Board Rule No. 10" (adopted May 10, 1907) and an exception to the same (adopted June 18, 1909). These special examinations have been held at the Board rooms, in Chicago, at the following times: Apr. 7-8, 1913; Sept. 25-26, 1913; March 25, 1914; and June 18, 1914 (also at Urbana.) A total of 236 candidates have taken the regular class and special examinations (up to Sept. 30, 1914), and of these 105 have passed and received certificates. Thirty practicing architects of other states received certificates after examination before the Board, by exhibits, and in some cases, after personal attendance. The following table gives a summary of the licenses issued, revoked and in force:

Date.	Total issued.	Total revoked.	Total in force.	Licenses in force. *A	Licenses in force. †B.
Dec. 1, 1898..	744	...	744	701	43
Dec. 1, 1900..	797	140	657	574	83
Dec. 1, 1901..	828	159	667	564	103
Dec. 1, 1902....	858	173	685	555	130
Dec. 1, 1903...	875	192	683	546	137
Dec. 1, 1904...	914	226	688	526	162
Dec. 1, 1905...	942	238	704	517	187
Dec. 1, 1906....	967	263	704	484	[illegible]
Dec. 1, 1907...	995	288	707	484	223
Dec. 1, 1908....	1,029	326	703	467	236
Dec. 1, 1909....	1,067	341	726	[illegible]	270
Dec. 1, 1910....	1,131	377	754	445	309
Dec. 1, 1912....	1,259	413	846	431	415
Oct. 5, 1914...	1,363	500	863	410	453

*Practicing before license law went into effect.

†Passed examination by the Board.

A total of 104 new licenses have been issued since the publication of our last biennial report, Dec. 1, 1912, and 87 licenses have been revoked. It will be observed that there are only 17 more architects practicing now than two years ago.

The present Board is proud of the results it has obtained, and these results were possible solely because of the harmony that existed among its members.

This Board agrees, as a unit, in a broad interpretation of the wording of the Act, and in the strictest enforcement of the law. This Board proposes to enforce the Act on its broad interpretation and will only narrow down its interpretation by the decision of the Supreme Court.

The present Board holds that only a licensed architect can practice in this state, or from this state, and that his license is not transferable or negotiable. Any combination formed for the practice of architecture, except between licensed architects, is illegal and any licensed architect who assists others to practice, who have no license, is guilty of dishonesty, as provided in the Act, and should have his license revoked.

The Illinois State Board of Examiners of Architects has been in existence for 17 years, but there was no Supreme Court decision until the present Board assumed office, and practically no court action of any importance that would give to this Board a precedent to follow. For the past 17 years we have read a great many legal opinions from eminent lawyers and there has been much argument among lawyers and among architects and the public as to the meaning of the wording of the Act. This wrangle has existed for 17 years. The Act was never broadly enforced, but complaints were mostly settled on a compromise which still left the meaning in doubt.

This Board proposes that at least eight cases of different forms of violation of the Act shall reach the Supreme Court inside of the next year, at which time the courts will definitely settle forever any question as to the proper interpretation of the wording of the Act, that is, the rights of the architect and the rights of the public; and when this has been accomplished, then the enforcement of the Act will be a more simple matter. Most of these eight cases are in preparation and some are in court at present. However, it must be born in mind that the cases selected are cases where the violation is specific of its kind and not complicated by various other elements, so that when a decision is rendered in the Supreme Court it will be clean cut and will apply only to that particular form of violation. In other words, where a decision is made in a case where there are many points at issue the decision is of little value, except as applied to that specific case, but if the decision is based on a simple issue it will apply to all similar cases and will be of great value to the Board.

There have been attempts made by men and bodies of men, both inside and outside the profession, to dominate this Board. The members hold that their position is a judicial one and must be free from such influence. Any complaint received by this Board, either from an individual or a body of men, will receive the same consideration, and no architect will be cited before this Board without first having had an opportunity to appear informally before it.

It must be born in mind that under the decision of the Appellate Court in the "Kaeseberg" case, this Board cannot revoke a license unless two cases of violation are proven; hence, a great many single complaints are filed that never come up formally before the Board. This leads the public and the architects of the state to feel that a great many cases presented to the Board are never pushed through to a proper finish. Each single complaint is filed, if the indications are that it contains a violation. When information of violations of the law is received from any reliable source, while the personnel of this Board remains unchanged, all architects may expect courteous treatment and a square deal, whether they be black, white or yellow, and regardless of their creed.

A word regarding the "City Ordinance" being violated by any architect may be expedient at this time. This Board is a state board, and as different cities and towns have different ordinances, the Board cannot revoke an architect's license because he violated the city ordinance, when the same architect could have built the same structure in other towns of the state without violating any ordinance. Therefore all matters pertaining to *incompetence or recklessness* must be based *only* on *violations of good engineering practice.*

This Board holds that the purpose of the examination is to ascertain the qualifications of the candidate, and that when the candidate procures his license he must, in his practice, keep near to the standard set by the Board. The Board, recognizing the existence of a low standard of work turned out by some architects (drawings and specifications being incomplete, either as to construction or materials or both, and often incomprehensible and not in accordance with *good engineering practice,* and the buildings erected in accordance therewith not only being a financial loss to the owner but a constant menace to the public) notice was served last June, that this Board will cite before it for trial any architect who prepared drawings and specifications issued for use in this state which indicate gross incompetency or recklessness.

The Board trusts that every licensed architect will co-operate in assisting to eliminate the preparation of drawings and specifications which indicate gross incompetency or recklessness.

This Board has found its greatest work to be the elimination from the architectural field of various architectural firms, which operate under an alias, such as architectural engineers, civil engineers, industrial engineers, engineers, designers, builders, etc. Most of these violations are assisted by a licensed architect, who is either financially interested, a partner, or who secures a salary. This Board has eliminated at least 20 such *illegal* combinations in the last few months and expects to eliminate all others from the architectural field in the near future. These combinations are to a great extent the result of lack of enforcement of the law or improper interpretation of the meaning of the wording of the Act. *Attention is called to the fact that all structural engineering on buildings is part of the architect's work and cannot be performed by others, except under the direction of a licensed architect;* and that the architect is responsible for all engineering data shown on his sealed plans, whether performed by him or not.

The *present members of* the Board have thoroughly analyzed the wording of the Act and *all agree* it is the *best* Act creating any of the Boards of the state of Illinois. We do not say that the Act is perfect, but we do say that it is usual and customary not to modify a law until, through the courts, its strong and weak parts have been found. Practically no court decisions, interpretating the wording of the Act, have been had in 17 years. The present Board feels that no attempt should be made to make any changes in the wording of the Act until such time as through the courts it is found inadequate. *Any attempt made by anyone at this time to have the Act changed will be considered inadvisable.*

It takes but little study of the problem to find what is needed to give the *architects* and *public* the results that the Act intended. No act is of any value or effect, unless enforced. We have the best act on the statute book, but *we have not the full power* to enforce it properly.

The Supreme Court has just held that the Act is constitutional, which is the only litigation of any real value this Board has had in 17 years, and any change in the wording of the Act would render this decision worthless, as it would not apply to an act which has been changed.

The architects and public must then wait until the new or changed act is held constitutional in order to enjoy the same security they now have.

The Legislature should look with favor on any legislation that will assist the State Board in enforcing the law that has been held constitutional and the Board wishes the architects to assist it in procuring proper legisla-

[illegible] giving the Board legal power to [illegible] all buildings that are [illegible] in this state without plans drawn by licensed architects and all structures that [illegible] [illegible] *architect* [illegible] licensed architect, as provided by law.

If such an [illegible] procured the Board will guarantee to the public and to the licensed architects of the state the full benefit of the Act of which they have been deprived for 17 years.

DRAINAGE AND IRRIGATION

Comparative Steam and Electric Power Layouts for a Drainage Pumping Plant.

In the land reclamation project of the Muscatine Louisa Drainage District No. 13, in Illinois, alternate plans were prepared for steam operated and electric pumping plants. The accompanying drawings compare the general plant layouts and the machinery layouts for the two kinds of power. The comparison is further made clear by brief descriptions of the two plants and by the comparison of bidding prices which follow:

STEAM PLANT

[illegible]	Machinery	Structures	Total
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]
4	70,676	41,000	111,676
5	73,294	41,000	114,294
[illegible]	[illegible]	[illegible]	[illegible]

ELECTRIC PLANT

[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]
4	80,690	40,125	120,815
[illegible]	[illegible]	[illegible]	[illegible]

Such comparison as these figures justify would better be made following the descriptions of the alternative plans which are given below:

[illegible]

The general plan arrangement for steam pumping is shown by Fig. 1; the portions of structures shown in dash line are those forming essential parts of the steam plant, but which are not required for electric operation. On the other hand, it is to be noted that the transmission line structure shown is eliminated in the steam plant. Specifically, the boiler house and equipment and the coal storage and handling plant, required only for steam operation [illegible]

Fig. 1. General Plan of Drainage Pumping Plant. Muscatine Louisa Drainage District No. 13, Illinois.

and 3 compare the steam machinery and the electric machinery layouts. The machinery buildings are duplicates in character of construction and construction requirements except as differentiated on the plans. All general specifications, requirements for materials and workmanship are identical for the steam and the electric plant buildings, foundations, engineer's residence, etc.

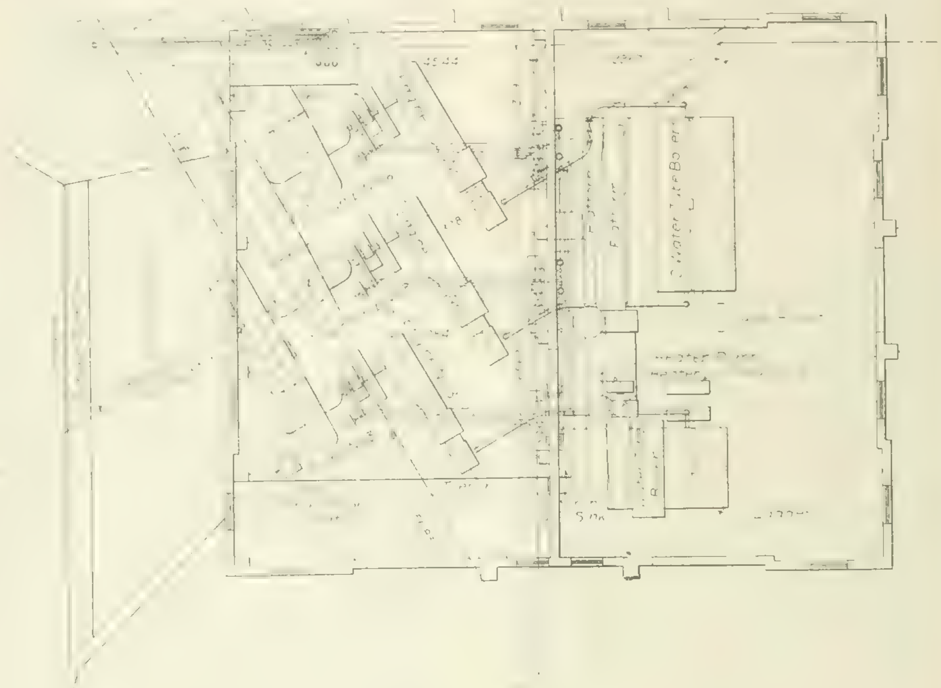


Fig. 2. Machinery Plan Steam Operated Drainage Pumping Plant.

PUMPING MACHINERY.

The main items of machinery required for steam pumping are: Two 54-in. low head, double suction, centrifugal drainage pumps, each having a capacity of 85,000 gals. per minute, against a maximum total pumping head of 16 ft., complete with suction and discharge piping, priming apparatus and water level maintaining system. One 36-in. low head, double suction, centrifugal drainage pump, having a capacity of 33,000 gals. per minute, against a maximum total pumping head of 16 ft., complete with suction and discharge piping, priming apparatus, and water level maintaining system. Three tandem compound condensing Corliss engines, or other acceptable type of steam engine, furnishing equivalent steam economy, direct connected to the centrifugal pumps; three Schutte-Koerting Eductor Condensers, or other acceptable type, complete with circulating pumps, etc.; three water tube steam boilers for 160 lbs. working pressure, completely equipped; three Foster Superheaters, or other acceptable type, for 475° F., working temperature; two sets of induced draft fan apparatus; two boiler feed pumps, with duplicate piping; one open feed water heater equipment; one auxiliary boiler· one indirect steam heating system for engine room; one hand operated traveling crane; coal storage platform, tramway, tracks and boiler room scales, together with all auxiliary equipment, instruments, appurtenances, piping, etc., necessary for complete working plant.

The principal machinery required for electric pumping was: One 3-phase, 33,000-volt transmission line, about 20 miles in length; one 33,000/2,200 volt step-down outdoor substation; two low head, double suction centrifugal drainage pumps, each having a capacity of 85,000 G. P. M., against a maximum total pumping head of 16 ft., complete with suction and discharge piping, priming apparatus, and water level maintaining system; one low head, double suction centrifugal drainage pump, having a capacity of 33,000 G. P. M., against a maximum total pumping head of 16 ft., complete with suction and discharge piping, priming apparatus and water level maintaining system; three 2,200-volt multi-speed motors, connected by herringbone gears or silent chain drives to the centrifugal pumps; two motor driven No. 6 General Condenser Co.'s rotary vacuum pumps, capacity 300 cu. ft. per minute each, or other acceptable make of vaccum pumps of equal capacity; one water level maintaining system; one warm air heating system; one main switchboard; one hand operated traveling crane; all auxiliary equipment, appurtenances, piping, wiring, etc., necessary for the complete working plant.

The requirements for data, guarantees and tests of pumping machinery were substantially as follows for both plants:

The machinery contractor shall furnish head-capacity and efficiency-capacity curves for the centrifugal pumps for various pumping heads and speeds throughout the working range of the pumps; curves showing the I. H. P. of engines at most economical cut-off at various R. P. M., and with steam pressures of 100 lbs., 125 lbs. and 150 lbs.; curves showing steam consumption of engines at different percentages of full load for steam pressures of 100 lbs., 125 lbs. and 150 lbs. with engine running at the mean speed required by the pumps; tabulation of the computations on which guarantees are based, together with all data necessary to enable the engineer to check the results.

The machinery contractor shall guarantee the specified capacity of the plant when operating against the maximum head of 14 ft. (difference in water level). He shall also state the guaranteed maximum coal consumption in pounds, per acre foot of water pumped when operating at a 6 ft. head (difference in water level). The fuel used will be equivalent to Illinois, Fulton or Peoria County, 1¼ ins. screened lump coal having a proximate analysis about as follows: Moisture 12.5 per cent, ash 14 per cent, volatile matter 33.5 per cent, fixed carbon 40 per cent heat value, per pound of coal as received, 10500 B. T. U. In case the coal used in acceptance test shows a different analysis the results will be reduced to coal of above quality.

Tests for capacity and efficiency shall be made by the engineer as early as practicable after the completion of the plant. These tests will be made at a time when the pumping head (difference in water level) ranges between 4 ft. and 7 ft. The results will then be reduced to a head of 6 ft. by means of the capacity and efficiency curves furnished by the contractor, at which head contractor's guarantees are made. The contractor shall furnish all of the labor, piping, fittings and apparatus required by the engineer as necessary for the tests and may have an acceptable expert representative present to assist. The instruments shall be calibrated under the direction of the engineer.

BUILDINGS.

The important engineering features of the building work are the machinery foundations and the suction pump structures. Figures 4 and 5 are the foundation plans for steam and electric plant, respectively. The requirements for foundation work are condensed as follows:

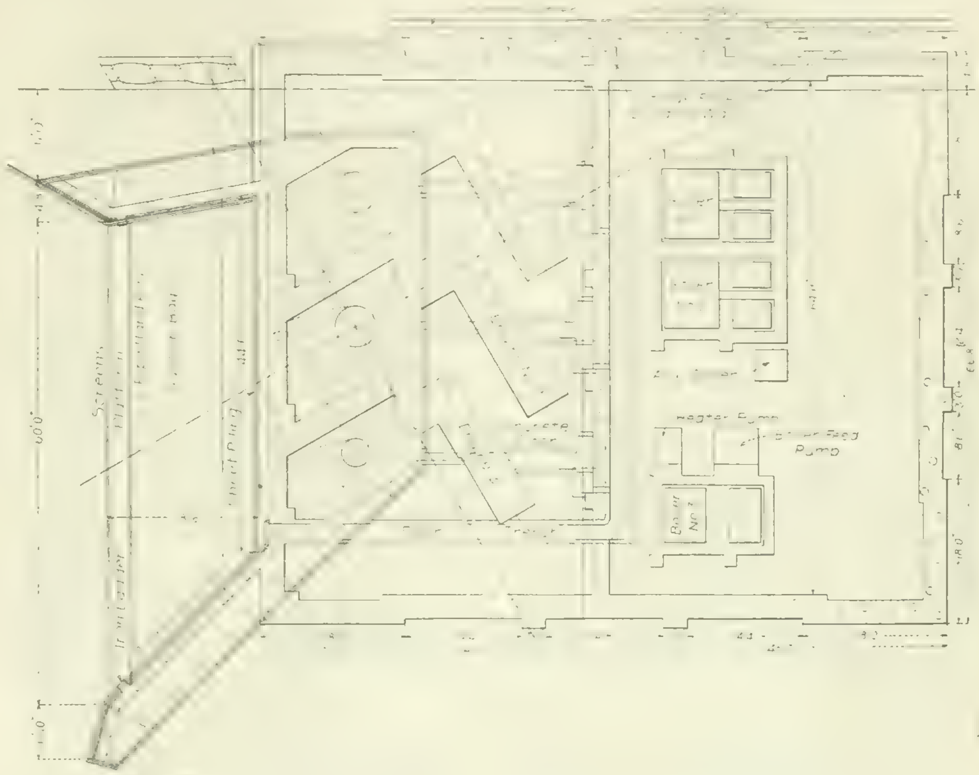


Fig. 4. Foundation Plan Steam Operated Drainage Pumping Plant.

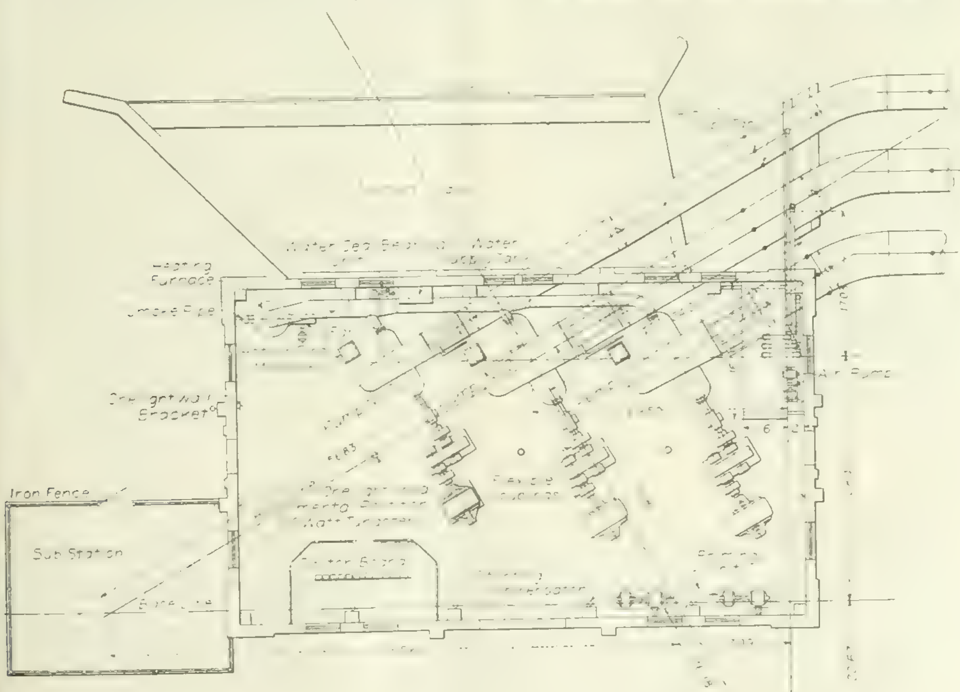


Fig. 3. Machinery Plan Electrically Operated Drainage Pumping Plant.

Excavating and Refilling.—All excavated earth is to remain on the premises and must be graded off inside the building up to the proper height to receive the concrete floor, and this filling must be wetted down and tamped in solid so that it will not settle.

Piling.—All round piles shall be of sound cedar, at least 8 ins. in diameter at the point, reasonably straight, and trimmed close. They must be spaced as required, driven vertically and, when necessary, bound with iron hoops, and after being driven to the proper depth, sawed off square and horizontal at the proper grade to project into footings 3 ins. All timber for sheet piling is to be of long leaf southern pine; it must be sawn square and of uniform thickness and must be free from large or loose knots, wind shakes or other imperfections impairing its durability or strength. All sheet piling is to be sawed off true and straight at the proper height.

Concrete Walls and Footings.—All footings and all walls of the building are to be built up of concrete to consist of "Atlas," "Chicago AA," "Universal" or some other brand of Portland cement equally satisfactory to the engineer, clean sharp sand and clean gravel or broken stone. The cement shall be subject to test by the American Society of Civil Engineers standard method and if not up to standard may be rejected or a richer mixture required. The gravel or broken stone, to be known as the coarse aggregate, shall be clean and of such sizes that all will stop on a screen of ¼ in. mesh, about 50 per cent will stop on a screen of ½ in. mesh, and all will pass a screen of 2-in. mesh. The concrete shall be made in the following proportions of the specified materials by measure, viz.: 1 of cement, 2½ of sand and 5 of coarse aggregate. If the mixing is done by hand the cement and sand shall first be thoroughly mixed dry and after adding the coarse aggregate shall be thoroughly mixed with a sufficient amount of water to produce a wet mixture before being deposited. If the mixing is done by machine it shall be with a batch mixer of type approved by the engineer. Immediately after mixing the concrete shall be deposited and thoroughly compacted in place. In cases where forms are necessary, they shall be built of smoothly dressed planks, securely fastened so that they cannot spring out of place when the concrete is tamped in. These forms must remain in place until the concrete is firmly set. All exposed surfaces of the foundations are to be finished smooth without resorting to plastering.

Concrete Floors.—The entire building, except under the pumps, boilers and engines, is to have a concrete floor constructed as follows: Level off the earth to a smooth, even surface and put down a 3-in. layer of cinders or sand well tamped down, and over this place a 6-in. layer of concrete composed of 1 part of Portland cement to 2 parts of sand and 4 parts of coarse gravel. Tamp this layer off true and even and immediately put on a ¾-in. wearing layer composed of 1 part of Portland cement to 3 parts of clean, fine sand. Trowel the surface of this

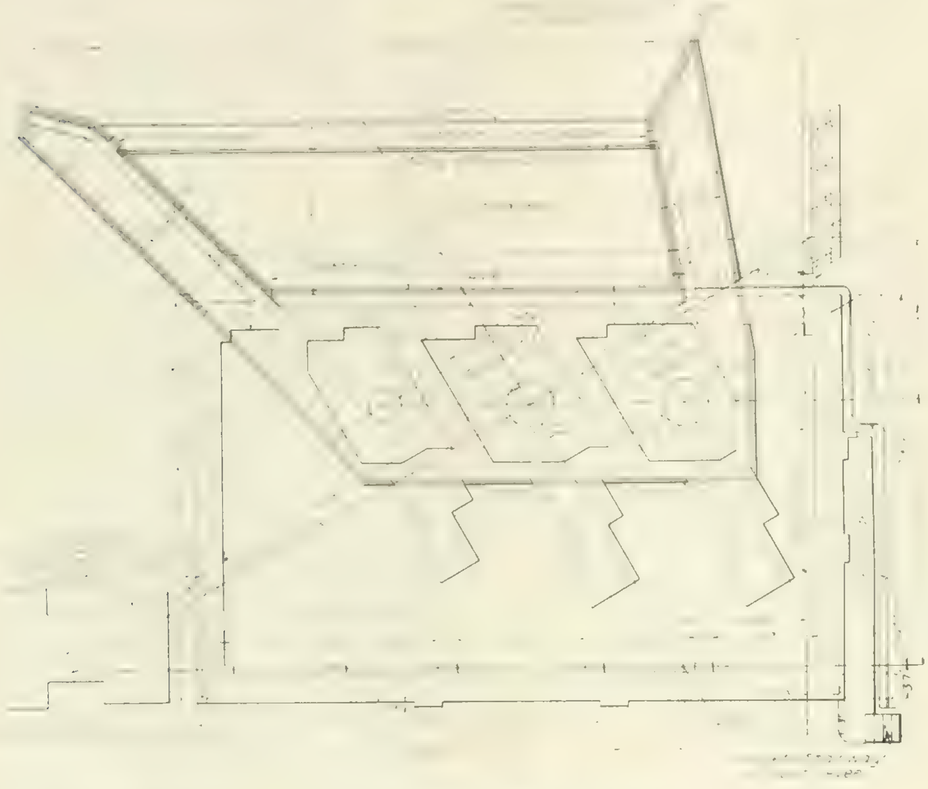


Fig. 5. Foundation Plan, Electrically Operated Drainage Pumping Plant.

wearing layer off smooth and divide into squares of convenient size to prevent cracking.

Further general details of structural features are given by the illustrations. The plants illustrated were designed by the Harman Engineering Co., Peoria, Ill., engineers, and contract for construction of a steam pumping plant has been awarded to the MacDonald Engineering Co., Chicago, Ill.

The Diaphragm Method of Measuring the Flow of Water in Open Channels of Uniform Cross Section.

The chemical method of gaging stream flow which has within a few years been developed in Europe was described in our issue of Sept. 16, 1914, and comparison of its accuracy was made with current meter and floating curtain, or diaphragm, methods. Since the diaphragm method, like the chemical method is somewhat unfamiliar in American practice, we give here some descriptions of it

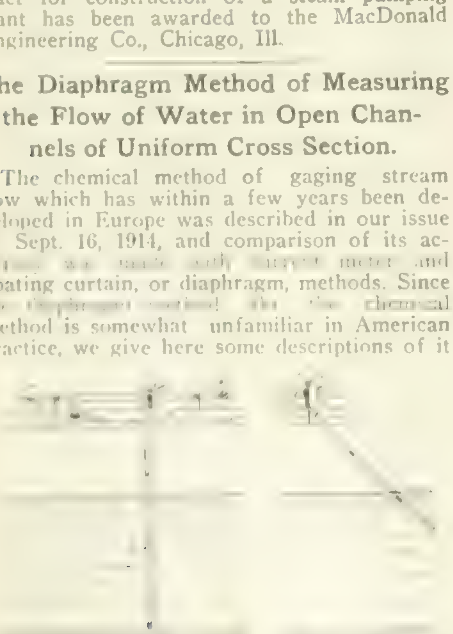


Fig. 1. Diaphragm and Car at Turbine Testing Station at Heidenheim, Germany.

from a recent bulletin of the University of Wisconsin Engineering Experiment Station.

GENERAL FEATURES.

The apparatus consists of a diaphragm suspended from a car, which runs on a carefully lined and leveled track laid on the walls of the canal. The car and the frame of the diaphragm are built of steel tubing or light steel angles and the wheels of the car are usually made of aluminum and run on ball bearings. The diaphragm frame is covered with oiled canvas or some light metal and swings, about a horizontal axis, in the direction of the current so that it can be readily immersed or withdrawn without causing an appreciable rise in the elevation of the water surface. When a measurement of the velocity is being made a clutch holds the diaphragm in a vertical position.

The method of procedure in obtaining a gaging is quite simple. A measured distance is first laid off along the canal walls and the diaphragm is then dropped into the stream at a point sufficiently far upstream, so that it will have obtained uniform motion by the time it reaches the beginning of the measured distance. The time of transit over this distance is then observed, from which the velocity of the diaphragm can be computed. The mean velocity of the water is then usually assumed to be the same as the velocity of the diaphragm. A correction should, however, be made for the frictional resistance of the car, and for the velocities in the clearance between the diaphragm and the periphery of the canal, which are not integrated by the diaphragm. In well designed apparatus this frictional resistance and the clearance—usually about half an

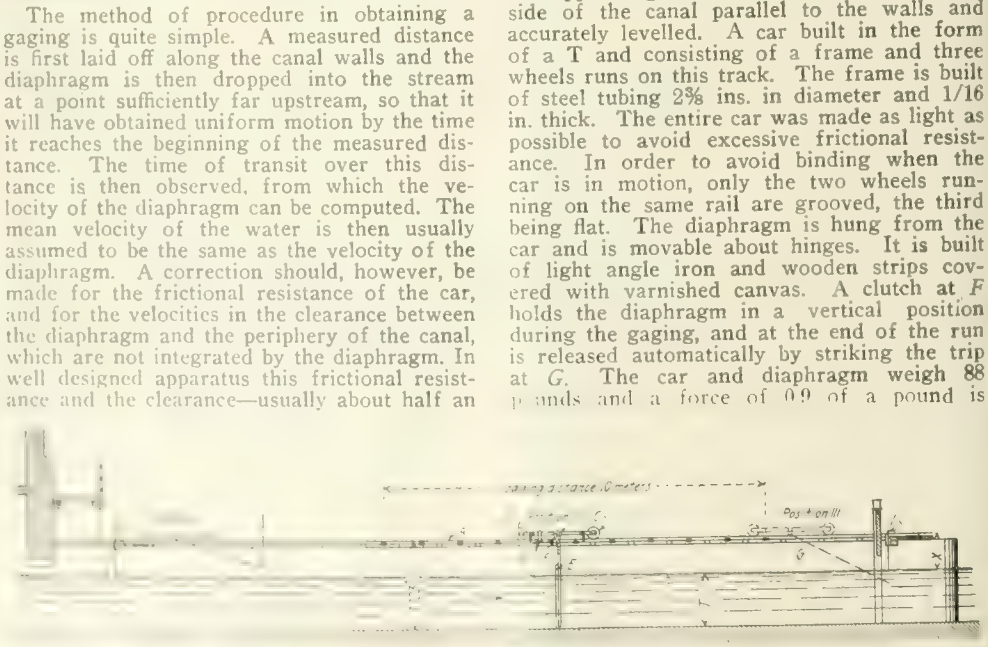


Fig. 2. Diagram Showing Maneuver of Diaphragm at Heidenheim, Germany.

inch—are so small that no serious error is caused by neglecting their effects, especially since they have a tendency to counteract each other on account of the smaller velocities occurring near the periphery. To obtain the discharge the cross-sectional area of the water must be known; the depth of water in the channel is, therefore, constantly observed during a gaging.

The chief advantage in this method is the rapidity with which the measurement can be made. This is of importance especially in the testing of turbines, where it is rather difficult to keep operating conditions constant for a time sufficient to obtain a good current meter measurement. In testing turbines at power stations, weir measurements are usually impracticable on account of sacrificing part of the available head, and current meter gagings, although the least expensive, require skilled observers, an accurate rating of the instrument, and considerable time in computing the discharge. A complete diaphragm gaging can be made in but a few minutes and the result immediately determined.

The disadvantages of the diaphragm method are that a channel of sufficient length and uniform section must be available, and that the cost of installing the necessary apparatus is rather high. For these reasons the method is limited to the measurement of moderately large quantities of water, and its application will probably be restricted to hydraulic laboratories, turbine testing stations, and places where the apparatus can be used often and first cost is, therefore, not such an important factor. For high-head power plants or small low-head plants, the installation of the diaphragm apparatus would be of considerable commercial importance, inasmuch as the operating efficiency of the turbines could be obtained at all times with a few simple measurements. The new method could also be applied to discharge measurements in the main laterals of irrigation systems, where considerable trouble has been experienced with weir measurements on account of the silt deposits. With the diaphragm the velocities can be made high enough so that this drawback will be overcome.

DESCRIPTIONS OF SPECIFIC INSTALLATIONS.

There have been abroad, a number of diaphragm gaging plants, installed and operated, but a description of two of these exemplifies practice.

Heidenheim, Württemburg, Germany.—This installation is at the turbine testing station of J. M. Voith. The testing canal is 9.84 x 4.59 x 72 ft. Angle iron rails *A*, Fig. 1, with the upper edge machined were laid on each side of the canal parallel to the walls and accurately levelled. A car built in the form of a T and consisting of a frame and three wheels runs on this track. The frame is built of steel tubing 2⅜ ins. in diameter and 1/16 in. thick. The entire car was made as light as possible to avoid excessive frictional resistance. In order to avoid binding when the car is in motion, only the two wheels running on the same rail are grooved, the third being flat. The diaphragm is hung from the car and is movable about hinges. It is built of light angle iron and wooden strips covered with varnished canvas. A clutch at *F* holds the diaphragm in a vertical position during the gaging, and at the end of the run is released automatically by striking the trip at *G*. The car and diaphragm weigh 88 pounds and a force of 0.9 of a pound is capable of moving it along the track when the canal is empty.

In making a gaging two men first hold the diaphragm out of the water with ropes, as shown in position *I*, Fig. 2. At a given signal it is dropped quietly into the water and the current carries the car and diaphragm along. The diaphragm dips into the water quietly without causing any commotion or formation of wave action, so that the regimen of the stream is not disturbed. After a short distance the diaphragm hangs in a vertical position, as shown in position II, and is held there by the clutch *F*. The actual length of the gaging distance is only 32.8 ft., as the diaphragm must travel over a considerable distance before the velocity becomes uniform. The time of transit is taken with a stopwatch and the velocity can then be computed immediately. As soon as the clutch strikes the trip at *G*, the diaphragm is released and takes the position shown as III.

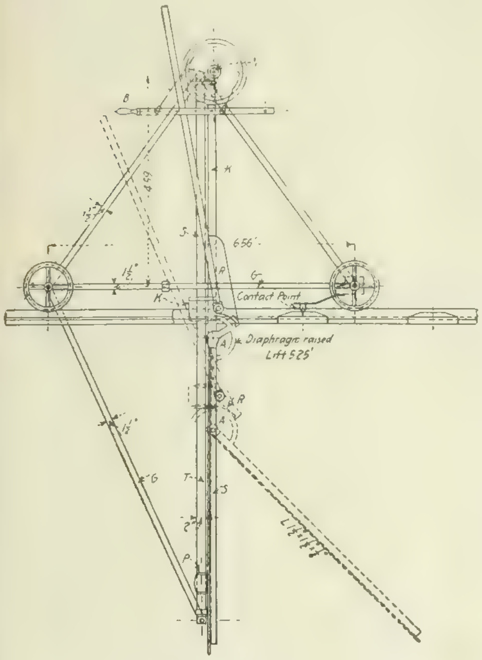


Fig. 2. Diaphragm and Car at Berlin Technische Hochschule.

During the run observers read the depth *X* of the water surface below the top of the rails at points *H* and *I*, Fig. 2. The readings at these points as a rule do not vary more than 0.005 ft.; the depth of water in the channel is, therefore, computed from the mean of these readings. The average depth of the bottom of the canal from the top of the rail is 7.723 ft. and the average width is 9.816 ft. The discharge is then computed from

$$Q = (7.723 - X) \times 9.816 \times v,$$

in which v is the mean velocity of the diaphragm.

The clearance between the diaphragm and periphery of the canal is about 0.4 in. As the smaller velocities occur in this part of the cross-section, it would seem that the velocity as indicated by the diaphragm would be larger than the actual mean velocity of the water. It seems, however, that this error is offset by the frictional resistance of the car and no correction is made for it. A few experiments made at this station with calibrated orifices to check the diaphragm method showed a close agreement, and that the diaphragm gave accurate measurements with velocities as low as 0.02 ft. per second.

Berlin Technische Hochschule.—The framework of the car, Fig. 3, is made of very thin steel tubing, brazed together and although weighing but 88 lbs. is very rigid. The diaphragm is built of light angle iron covered with oiled canvas and is hung from the horizontal axis *A*, Fig. 3, about which it is free to swing in a forward direction. It can also be raised or lowered by two small cables attached to the two hand wheels *N*, the guides *K* sliding along the two vertical tubes *T*. A brake *B* is attached to one of the wheels so that the speed with which the diaphragm is lowered can be easily regulated. Its descent is limited by the two rubber buffers *P*. When in a vertical position the diaphragm is held rigid with the vertical frame by means of the clutch *R*, and when the clutch is released the current swings it around the axis *A* to the dotted position.

The bottom and sides of the canal were carefully plastered with cement mortar, so that the clearance between the diaphragm and walls is only about 0.2 to 0.3 in. A distance of about 10 ft. is necessary for immersing the diaphragm, so that the actual length of the gaging distance is only 23 ft. Experience has shown, however, that this distance is sufficient with the rapid vertical immersion of the diaphragm.

With the normal depth of water in the tail-race the velocity is about 2.3 ft. per second and the discharge can be obtained with great accuracy. When the velocity of the water becomes as small as 0.3 ft. per second the measurement becomes unreliable, as each little wave, due to disturbances outside of the tail-race, then affects the motion of the diaphragm. With such small velocities, however, current meter measurements are also unreliable and in the opinion of the director of the laboratory diaphragm measurements are preferable in all cases. A comparison between the results obtained with the current meter and with the diaphragm at this laboratory, showed a satisfactory agreement. In one of the tests the mean velocity obtained with the current meter held at 21 points in the cross-section was 2.729 ft. per second, while with the diaphragm the velocity recorded was 2.709 ft. per second, or a difference of about 0.7 per cent.

Comparative Tests.—Comparisons of meter, diaphragm and chemical gagings obtained by the Swiss Bureau of Hydrography were given in our issue of Sept. 16 previously mentioned. A further comparison is given by the diagrams of Fig. 4. The different methods of gaging have been distinguished by using different characters for the plotted points. The solid curve represents the average of the plotted meter gagings, while the dashed curve is the computed weir-discharge curve, using Frese's coefficients. The latter curve at first glance appears abnormal because it crosses the meter discharge curve. This anomaly may, perhaps, be partly due to the contractions caused by the two stems for raising the weir, the effect of which seems to vary with the head. From the curves it can be seen without further discussion, that the diaphragm gagings check the meter gagings very closely. The differences range between such small limits, that for most practical purposes they are of no significance. The greatest difference obtained with a direct comparison of meter and diaphragm gagings was 0.7 per cent. Theoretically the results of the diaphragm gagings should be too small rather than too large, as the velocity of the diaphragm is retarded by the frictional resistance of the car. It seems, however, that the effect of this resistance is offset by the smaller velocities, existing in the clearance between the diaphragm and the periphery of the canal, which are not measured by the diaphragm. The smallest quantity gaged in the rating canal at Ackersand was 1.24 cu. ft. per second, at which rate of discharge the diaphragm still operated faultlessly.

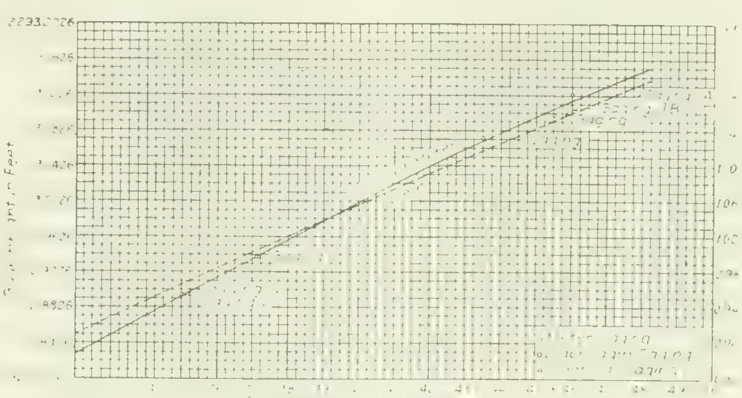


Fig. 4. Comparison of Meter, Chemical and Diaphragm Gagings by Swiss Bureau of Hydrography.

Some Methods and Costs of Cleaning Drainage Ditches.

Methods employed in cleaning drainage ditches are described by Mr. Seth Dean, civil engineer, Glenwood, Ia., in the Proceedings of the Iowa State Drainage Association:

In the spring of 1910 the writer cleaned a bed of silt ranging from 6 ins. to 3 ft. in thickness and three-fourths of a mile in length from a channel originally cut 16 ft. wide on the bottom, but at the time in question the stream of water flowing over the silt was about 10 ft. wide and 1 ft. deep, the rate of fall being about 2 ft. per mile. There was considerable sand and some drift in the silt but no growth of weeds or brush. The plant used consisted of a flat-bottomed boat or scow, 7 × 18 ft. in size and 16 ins. deep, made of 1-in. plank. In the bottom of the scow a platform of 2-in. plank was laid to support the machinery, which consisted of a 4-hp. gasoline engine belted to a Myers pump with 3-in. suction and 2½-in. discharge. The pump was equipped with 10 ft. of 3-in. suction hose with strainer on the inlet end, and for discharge had about 15 ft. of 2½-in. fire hose with 1-in. nozzle. The scow when loaded required about 6 ins. depth of water to float. Commencing at the lower end of the silt bed the boat was poled forward or held in place, as required, and a jet of water turned through the nozzle into the silt that readily broke and stirred it up, permitting the water to float it away. The work was done in March and April, when the flowing water was clear and capable of carrying silt in suspension, the distance from the center of the silt bed to the outlet of the ditch was about 10,000 ft., and the current sufficiently strong that little settling of silt occurred. Three highway and one railroad bridge spanned the ditch in the distance cleaned, but the boat readily passed under them. Two men operated the machine and the total amount of silt removed was 2,346 cu. yds. in 33 working days. The cost of the equipment was as follows, viz.:

Cost of scow	$ 45.00
Engine and pump	200.00
15-ft. condemned hose and nozzle	3.00
Belting and fixings	8.60
Freight hauling and setting up	32.00
Two men 33 days at $4	132.00
Gasoline and oil	26.40
Repairs on machinery	1.05
Total	$448.05

After the work was completed the plant was dismantled and the engine and pump shipped to other work which was charged with their cost, thus making the net cost of the plant $248.05 and the cost of cleaning 10.53 cts. per cubic yard.

On one occasion a bed of silt interspersed with logs, brush, cornstalks, etc., was removed, using drags made from the beams and shovels of wornout corn cultivators by bolting the parts together in such manner that they presented the appearance of two anchors placed at right angles. The point of the beam was fitted with a swivel so the implement could revolve. By attaching ropes to the drag, placing a team on each bank and dragging the plow in the channel, the mass was [illegible] up. After [illegible] and wire (dynamite being used sometimes to dislodge them) the water floated out the silt. A close measurement of the silt and drift removed from the channel was not made, as the work was done under the day system, but approximately 2,800 cu. yds. were taken out, the cost being the following items:

[illegible] for 21 days	$[illegible]
Two [illegible] with [illegible]	[illegible]
[illegible]	[illegible]
Foreman, 21 days at [illegible]	[illegible]
Total	$411.00

Or about 15 cts. per cubic yard.

In the fall of 1912 we cleaned and deepened what is known as Seaton's ditch, near Missouri Valley. This is a drainage ditch 7,600 ft. long with 6 ft. bottom width, and side slopes 1 to 1. During the rainy season and for a time afterward the ditch carries water but is usually dry during the fall months. The work of cleaning was done by contract at 19 cts. per cu. yd. The contractor bid to do the work with teams, but the ground proved too soft for this method, and a small drag line dredge was purchased and the work successfully carried out with this, which proved to be an excellent machine for the work. The machine was made at Cherokee, Ia., of light timber construction. The framework, 16 ft. wide, is mounted on rollers and designed to work astride the ditch in clean-out work. The power is generated by an 8-hp. gasoline engine, which also serves to move the machine forward or transport it from one job to another along the country roads if the distance is not great. It uses a one-third yard scoop; two men operate it, using about 10 gals. of gasoline per day. About 250 cu. yds. of earth in ten hours was the capacity of the machine on the job in question. The machine is of wood construction and is not very durable, but as most of it is of sizes kept in all lumber yards, defective parts can be easily replaced.

WATER WORKS

The Causes of Breaks in Large Water Mains in Chicago.

At intervals for many years breaks have occurred in large water mains in Chicago. While such breaks were made a matter of record their causes were not investigated. During the spring and summer of 1912 a large number of leaks were reported and, since no cause was clearly apparent, the Commissioner of Public Works ordered a thorough examination to be made to discover the causes. The investigation was made by Claude E. Fitch, assistant mechanical engineer, and his report, containing his suggestions for remedial and preventive measures, is published in the annual report of the Department of Public Works for 1913. The matter here given is based upon that report.

Mr. Fitch lists 13 causes for breaks in mains, as follows: Improper design; poor material; improper handling and installation; subsidence or rise of soil; jarring from traffic; frost; temperature changes; electrolysis; corrosion; excessive static pressure; excessive momentary pressure; periodic pressure waves; and, extraordinary causes. Explanations of these causes follow.

[illegible]

For the design of pipes there are numerous formulas giving very nearly the same results. Pipes have been manufactured in such quantities that these formulas are as much the result of experience as of theory, and have generally been considered satisfactory. For the design of valves and special fittings, theory and [illegible] more likelihood of bad design in these appurtenances than in the pipe. New makes, untried by long practical application, are, of course, much more likely to be deficient in strength than the well-tried valves and other fittings.

[illegible]

All reputable builders and nearly all large users employ competent inspectors. Measurement, superficial inspection, pressure test, ham[illegible] the methods generally employed. Chemical tests are not so common, but have their value, as the excess or deficiency of certain elements is known to be detrimental.

[illegible] therefore indicates only tensile strength. Quietly applied pressure gives no indication of brittleness, one well known characteristic of cast iron. The hammer test might discover it, if excessive, but the blows are mostly as local in their indications as in their effects.

Superficial defects are sometimes covered up by the paint [illegible] painting would not discover them. Measurements taken after [illegible] same criticism. The metal is sometimes improperly cooled, causing internal strains. It is sometimes locally cooled, or the metal is not hot enough to solidify into a homogeneous mass, causing cold shuts. It sometimes encloses gas, air or foreign solids, thus reducing the apparent [illegible] also separate chemically [illegible] different compositions, therefore having different strengths and coefficients of expansion.

The breaking of a test bar can be taken so as to give indications as to the character of the metal in the heat, but can be manipulated so as to give false indications.

IMPROPER HANDLING AND INSTALLATION

A pipe otherwise good may be badly damaged by mishandling in transit or at laying, and thus weakened so as to be unserviceable.

It may be improperly caulked. If the caulking is skimped, the joint may pass inspection, but leak shortly afterward. If too tightly caulked, the bell may be strained so that settlement of the pipe, temperature changes or jarring such as may be ordinarily expected in service, can easily furnish the added stress for breaking or leaking.

When back filling the trench, careless work will cause the pipe to settle and get out of line. It may easily be conceived that serious strains can be set up in the pipe, if local settlement causes a section to act as a beam supported at the end or middle, and loaded by the weight of earth and pavement above, street traffic, etc.; also that strains may be caused by rigidity of attachment to adjacent sections.

Bad alignment has another possibility, which may become serious if motion is not prevented. Each deflection of the moving column of water by the pipe is accompanied by a pressure of the water tending to straighten out the bend. The pressure is not serious at low velocities, nor at constant velocity, and a firmly anchored pipe would suffer nothing worse than a scouring action within. A pipe not firmly anchored has a tendency to move, and might move appreciably. This would be serious if the water pressure fluctuated rapidly. The worst case under this condition would be at a right angle bend. The use of a bend would be by design, yet the pressure here would cause a tendency to movement in the direction of pressure, and if not prevented by proper installation can prove a source of trouble.

[illegible]

Assuming that the pipe has been properly laid, then the soil might be disturbed in one of several ways. A parallel trench for other underground pipes or conduits would, while open, relieve the side support of the earth between, and if properly back filled, be a probable cause of lateral movement of the soil. Construction of the enormously heavy modern buildings has caused movement of considerable bodies of soil, not only laterally, but also upward and downward. Construction of tunnels and subways seems to have affected structures above and adjacent, possibly not only from removal of material, but by drainage of water; the volume of water drained being replaced, at least in part, by subsiding soil. Sufficient soil displacement will cause leaks and maybe breaks.

[illegible]

Where pipes pass through embankments under railroads, under street railways and in streets subject to heavy traffic, there is not only displacement of soil from the weights above, but also a jar which is transmitted to the [illegible] considerably, the danger to the pipes is the same as described in the previous paragraph. The jarring may break a brittle pipe, but the more probable trouble is at the caulking, where the lead is continuously deformed by the blows, until no longer tight. Then the leakage causes a softening of the soil, making further displacement probable.

FROST.

Under ordinary conditions this is little to be feared in large mains, as the depth to which the ground freezes in severe winters has been determined by observations extending over many years. Further, the water is scarcely ever below 39° F. in temperature, and if flowing will ordinarily not freeze solid. Troubles from frost are mostly confined to smaller mains and to service pipes.

TEMPERATURE CHANGES.

The range of water temperature is not very great in Chicago, probably 25° F. This is enough to cause a measurable amount of expansion. Expansion or contraction is accompanied by great force; almost invariably great enough to cause endwise motion of the pipe against the friction of surrounding soil. The motion undoubtedly is taken up in the more loosely caulked joints, causing them to become more loose and to leak a little, but this ability to move slightly at the joints is the reason for the use of bell and spigot joints, rather than bolted joints.

A more serious phase of this condition may occur at a tightly caulked joint, where the cast iron of the bell is first severely strained by the excessive caulking. Should this caulking be done in cool weather, the expansion of the lead, being about five times as great as that of cast iron, may cause sufficient additional strain to rupture the bell.

ELECTROLYSIS.

The evidences of damage by electrolysis are generally plain and are likely to be confused only with chemical action due to seepage of corrosive liquids through the soil. Chemical tests of the surrounding soil and electrical tests for difference of potential between the mains and adjacent conductors will differentiate between these causes.

CORROSION.

The pipes are made thicker than necessary for resisting the physical strains imposed in order to allow for rusting and are coated within and without to protect the metal. So far as simple rusting is concerned, a pipe otherwise strong would probably outlive its usefulness several times in a community of rapid changes. Chemical action may occur from acids in underground water. The presence of deleterious free acids, other than carbonic acid, is generally due to neighboring manufacturing plants employing chemicals, and their source should be an easy matter to trace.

EXCESSIVE STATIC PRESSURE.

In a small system, relatively new and tight, this might be a source of danger. Derangement of the pressure governors of the pumps and simultaneous carelessness of station employes, can then cause excessive sustained pressure. The danger to a large system with numerous pumps, numerous engines, and mains much cross connected, some old and leaky, some new, cannot be considered great. Should one pump run away, other pumps would automatically slow down. Should one station go seriously wrong, the attention of other engineers would be drawn to the trouble.

The pipes, as a general rule, are tested to three times the working pressure at the foundry and 1½ times the working pressure when completely laid, so that unless some other trouble occurs in conjunction, this is the least to be feared.

EXCESSIVE MOMENTARY PRESSURE.

Shock, momentary pressure in the nature of a blow, covers in water service a number of troubles, which are frequently separately described, but their action and their effect is the same—that of a blow given to the pipe and fittings by the water. The various terms are as follows: Shock, hydraulic shock, water hammer, water ram, hydraulic inertia strains, pulsations from pumps, pressure waves, sudden changes in velocity of water, sudden closing of valve, and air in pipes.

This is one of the most serious of the troubles to which water mains are subject. It is serious for two reasons; one, that its intensity cannot be calculated with any degree of certainty and therefore it cannot be provided for in the design except approximately; the other, that present methods of inspection, even if perfectly carried out, will not reasonably insure the mains against its effects. The blow struck by the water is of the most searching character, as every part of the water, even that in joints and corners, delivers it to the iron. If it does not cause rupture or get other relief through a safety valve or an air chamber, it is reflected back into the water as a wave and then travels through the system until its force is spent.

Pulsation from Pumps.—Centrifugal pumps, unless they are drawing air through the suction, give little trouble in this way, and when air is so drawn in, they make considerable noise but do little harm. Owing to the absence of valves in centrifugal pumps it is necessary to have check or gate valves in the pipes to prevent water from running backward from the mains, when they are not running, and these valves, if closed suddenly (as check valves do when the pumps are suddenly stopped), cause water hammer.

Reciprocating pumps have plungers or pistons in combination with valves permitting flow in one direction only, though the plungers at the end of each stroke reverse their motion. Therefore, one-half the time they are drawing in water, and the remaining half they are delivering water. Each time they commence their delivery strokes they must first build up a pressure in the chambers sufficient to close the suction valves (assisted by their springs or other mechanism), then sufficient to start the water in motion towards the delivery valves and, lastly, build up a pressure sufficient to open the delivery valves against the pressure holding them to their seats.

In crank and fly wheel pumps the change of the speed of the plunger should be gradual by reason of the crank action, but the fly wheel and inertia of moving parts may be the cause of shock, thus: Assume that the suction valves are slow acting or leaky, then a portion of the water will pass back through them while the plunger is passing the point of reversal of motion and is gaining speed on its next stroke. When the suction valves have closed as tightly as possible, the friction of the water in passing through the leaks increases because of the increasing quantity of water being moved by the plunger, and consequently the pressure in the pump chamber increases until the delivery pressure is reached. This action takes a measurable amount of time and the plunger has gained velocity but no water has been delivered. The instant the pressure is able to open the delivery valves, water-flow through them commences suddenly. The water above the valves has been quiet for the time of the suction stroke and then is almost instantly in motion. Its inertia tends to delay this, but the force stored in the flywheel and moving parts of the engine is enough to overcome it suddenly. Thus a blow is struck the water in the mains.

Most pumps have air chambers to protect the mains, but if the air chambers are not working properly, a most serious blow can be transmitted to the water in the mains. If the crank or crosshead pin bearings are loose, there is lost motion, and the reversal of motion is not accomplished gradually but suddenly. A fourth trouble in pump operation, not alone of pumps with crank and flywheel, but also of those without, is the presence of considerable air with the water pumped. This air must be compressed before any pressure is obtained in the pump chamber. An exaggerated case of the last trouble came under the writer's observation, where fully ten per cent of the stroke was spent in compressing the entrained air.

The reciprocating pumps without crank and flywheel, to assist the reversal of the plungers, are forced to admit an excess of steam, which starts the plungers with a jerk and brings about the same sudden increase of pressure in the pump chamber, as in the other type.

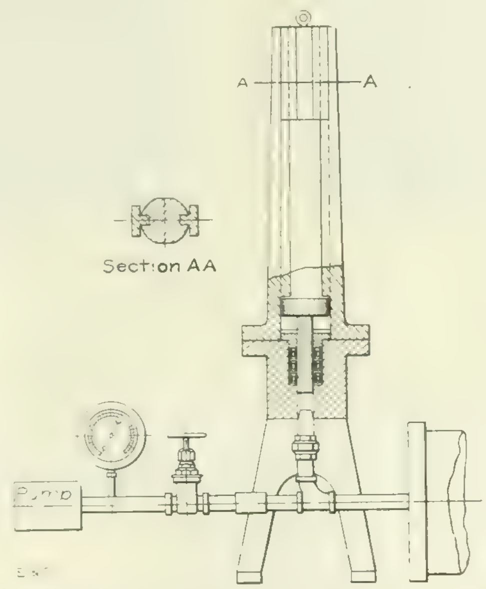


Fig. 1. Diagram of Apparatus for Testing Water Pipes and Fittings by Water Hammer.

All of these causes are bad, but when several act in conjunction, and especially when the air chambers are insufficient or insufficiently filled, the shocks endanger the entire hydraulic system, pumps as well as mains.

Water Hammer.—Water hammer is due to a sudden check to the velocity of the flowing water. Its intensity depends on the quantity flowing and the amount and suddenness of change in velocity. It is almost invariably due to the action of the consumer or his machinery in suddenly shutting off the flow.

The condition for the formation of water hammer having been produced, say by the sudden closing of a valve in a pipe of moving water, then the action is as follows: The energy of the flowing water is used up in compressing the water and simultaneously by expanding or distending the cast iron. This continues until the water has stopped. Then the elasticity of the iron and water impels them to return to their former condition without the compression and distention, which is accomplished by sending a wave of compressions back into the water traveling at a speed approximately 4,670 ft. per second. The particles of iron and water concerned swing beyond a condition of no strain, back and forth several times, being rapidly checked in their motion, before coming to rest.

The action in the iron is not easily made apparent, but in the water is seen as a throw of the index of the water pressure gage, or better, by the more sensitive engine indicator. The pressure waves diminish so rapidly that only the first and second are of much account, so far as injurious results are concerned. These waves have somewhat the characteristics of a wave on the surface of water when approaching a shore, that is, they are retarded and have a steep front. They also climb over each other (in a sense) as surface waves, by adding the pressure of one to the pressure of another. Therefore it is possible to get a worse condition away from the source than obtains there. The waves will travel either with or against the flow indifferently. If the cross section of the water pipe increases, the waves are diminished in intensity; if it diminishes in cross section, especially with a funnel shape, the waves increase correspondingly.

Air in Pipes.—Air in pipes when free to move with the water causes trouble when the water is flowing out, as the water and air are ejected alternately, and a slug of water strikes a faucet or bend after each flow of air. This is also water hammer, and is easily relieved by automatic valves. Air in pipes, where there are great changes in elevation, can prevent flow of water completely, but that effect is not within the scope of the present report.

PERIODIC PRESSURE WAVES.

This cause of injury to mains is of less importance than most of the preceding ones. The damage which it may cause can be as serious as any of the others, and it works in a way which might not be easily discovered.

Assume a pipe which is so constituted or laid that it has a natural period of vibration, either laterally, up and down, or by expansion and contraction, just as a pendulum has its time of swing. Then let it receive a set of pressure waves in tune with its natural period of vibration. It will vibrate with increasing force until friction prevents further increase or the impressing vibrations cease or get out of phase. Brittle material like glass is shown as an example of this phenomenon, which is called resonance, and it can be broken to pieces when the impressing force is nothing greater than a musical sound, if the conditions are right. Cast iron is more or less brittle, and also somewhat elastic, so that it may vibrate. Such pressure waves, if on the right period, can cause a rupture.

EXTRAORDINARY CAUSES.

These require little explanation beyond naming explosions and blasting, earthquakes, lightning, intentional injury, and possibly the breakdown of a heavy truck above.

SIMULTANEOUS CAUSES.

While it is entirely possible for any one of the causes previously listed and explained to rupture a water main, yet, as two or more may occur at one time, it is more probable that this is the true reason for most of the breaks, each contributing its share of the excess strain. This adds to the difficulty of determination in any one instance.

Excessive caulking, strain from temperature changes, lateral displacement, jarring from traffic, maximum static pressure, local weakness, not discovered at test, and water hammer can all occur simultaneously; forces the total of which can not be calculated, nor provided against in the design by reasonable thickness of material.

ANALYSIS OF 47 BREAKS IN 1912

In his report Mr. Fitch schedules 47 locations of "breaks" in large water mains which occurred in Chicago from May 4 to Oct. 20, 1912. These "breaks" consist of 42 joint leaks, 3 broken valves and 2 burst pipes.

The broken valves were of a type no longer installed and considered poor in the matter of design. Without question the trouble came from improper design. One of them burst when closed, while the street was opened up for other repair work.

The joint leaks predominate. One was under a railroad embankment, and being isolated, may properly be laid to jar from passing traffic and subsidence of soil. Four others were all under street car tracks and in one locality. In the last few years tracks have been relaid and heavier cars used. It is probable that these have at least in part the same causes as the preceding, jar from traffic and subsidence of soil. Two others were also close to street car tracks, but also come within the territory where there are numerous other similar leaks, therefore the jarring is not the probable cause.

While jar from traffic may be responsible in part for this formidable list of joint leaks, there are two other reasons which may be inferred. A large number were discovered in the spring and early summer after a very severe winter and temperature changes may be in part responsible. Reference to a spot map, however, shows that the overwhelming majority of them were within a mile or so of Fourteenth Street Pumping Station. Temperature changes could not have produced such a local result. Harrison Street Pumping Station is the next in order of nearness to the locality of the numerous breaks, but nearly all of those occurring about that station were under street car tracks. An investigation into the conditions at the two stations should show with more certainty whether one or the other were the more likely cause.

Harrison Street Pumping Station has two E. P. Allis' pumps, 27 in., 46 in. and 70 in. by 60 in., triple expansion steam cylinders with triplex 32¾ in. single acting plungers. Fourteenth Street Pumping Station has three exactly similar pumps by the same makers and one Lake Erie Engine Co. pump. The latter has cylinders 34 ins., 60 ins. and 94 ins. by 60 ins. with plungers 42 ins. by 60 ins. and has twice the rated capacity of each of the others. It has further flywheels of the same diameter as the others (20 ft.), but weighing 32 tons as against 25 tons. Either they have too heavy, or it has too light a flywheel; undoubtedly the latter, as weight contributes to regularity of motion. Lastly, the E. P. Allis type has a very large air chamber for each plunger, the Lake Erie pump has only one and that on the pipe line. Thus the latter has only one air chamber as against three for the others to prevent the pulsations of pumping from reaching the mains. This pump has the largest rated capacity of all, the variations in flow during each revolution are consequently the greatest, and it has been the poorest protection for the mains. It is, therefore, the most likely engine to cause trouble by pulsations and shock to the water in the mains. Another element in the design of this pump is the check valve. With reference to the air chamber it is on the side next the water main. Pulsations of flow, then, cause the valve to open more or less, and if sufficient, cause it to close entirely, when water hammer results without benefit of the air chamber.

The greater part of the joint leaks were found on the main supplied by this large pump, and their cause was undoubtedly some derangement of the action of the pump which transmitted pulsations to this main. This action may have been assisted by the temperature changes from the extreme winter to moderate weather and by subsidence or rise in the soil, as the main leads into the district of recent large building construction. This will account for the greater number of breaks in this district, and then only the weaker joints would be affected at greater distances.

The broken pipes were two in number. The first could not be well studied, as no particular investigation had then been commenced. It had the greater part of the bottom blown out nearly from end to end.

[illegible] about 2½ sq. ft. area blown out clean. The pipe was examined for electrolytic injury, [illegible] of a power plant, but none was discovered. The edge of the broken part showed no flaws. It was two weeks before the piece blown [illegible] churned the ground so much, and then the edge was already rusted, so that it was impossible to discover if there were an earlier crack which had suddenly increased. There was no pitting which could be attributed to electrolytic or chemical corrosion. The metal was slightly thicker than called for by the specifications (0.9375 in.) as against [illegible] in. [illegible] Two test bars [illegible] were cut out and showed 19,550 and 19,400 lbs. per square inch. A transverse piece 1 in. by 2 ins. by 24 ins. broke at 2,080 lbs. with 0.45 in. deflection. The metal therefore shows excellent qualities under test.

Both these pieces burst about 1:00 or 1:30 a. m., the hour of maximum pressure. It must be presumed that this maximum pressure was in part responsible, though it was never higher than 41 lbs. at the nearest station. The mains are tested to 100 lbs. when laid, therefore some other cause acted in conjunction with the maximum pressure. The general rupture of the first pipe would indicate poor metal. The local rupture of the second, would indicate a cold shut or irregular cooling strain over a limited area. The excess pressure, causing rupture, was without doubt due to the crest of superposed pressure waves. This acted suddenly and locally and found weak metal at the point of rupture.

NEW TEST METHOD.

To reproduce the stress caused by water hammer definitely and positively, a device can be easily provided. A schematic drawing of such a device is given in Fig. 1. Here is shown a plunger of small size, free to move inward a short distance towards the water in the pipe while on the testing floor. The motion outwardly is prevented by stops. A weight in guides is shown above the plunger ready to fall upon it. The water under the plunger is connected to the water in the pipe, which, when the weight is dropped, should have a static pressure equal to the maximum working pressure, say 70 lbs. for Chicago. The action of the weight on the plunger will be to convey a blow to the water and from the water to the pipe, giving the exact reproduction of water hammer so far as straining the pipe or fitting is concerned. The size of the weight and the distance it falls can be varied for different test pressures, and, when once fixed by experiment, will be tolerably constant in results. This tests the cast iron for brittleness, one of its properties, and the one least tested for in practice.

It is possible to pursue the method further and create vibratory strains by intermittent light blows upon the plunger, but unless further proof of its necessity is adduced this would not seem necessary.

AIR CHAMBERS ON PUMPS.

Air chambers are in nearly universal use on pumping engines, and their value in equalizing the flow and the pressure of the water is so well recognized that little argument as to their usefulness is needed. An air chamber is usually half filled with compressed air equal in pressure to the average water pressure. On a large air chamber a water gage is generally placed so that the dividing line between air and water can be readily seen. The air chamber is connected to the water by a passage of such size and shape as to permit easy flow of the water in and out, but no flow of the air, unless the latter is excessive in quantity.

The size of an air chamber is proportioned so that, when the air is compressed further by inrushing water of higher pressure, enough work is absorbed by this compression to overcome the force in this water without increasing the air pressure unduly.

When the water under higher pressure has been forced in until the pressures are equalized, the mains have been relieved from taking the rush of the excess water suddenly and have been subjected to a pressure higher than the average, by the amount the air pressure has been increased. As soon as the excess pressure impressed on the water diminishes, then the air forces out water until equilibrium is established.

[illegible]

Air chambers, being so useful as adjuncts to pumps, can be applied to mains to protect them in like manner. When pulsations of flow on pressure do occur in the mains, some get back into the air chambers connected at the pumps, but in so doing act over long sections of the mains. The city has many build- [illegible] stations, where some corner of the basements could be spared for air chambers. By running in a connection to the main in the adjacent street wherever possible, the most perfect protection could be afforded over the whole system against water hammer. Possibly business blocks could be used in localities where public buildings are unavailable.

In connection with such an air chamber some method of charging would be necessary, and periodic inspection to keep the proper amount of air in. The persons in buildings so utilized would be asked to do no more than to report an accident, such as breaking of a gage glass, or noisy action. The latter would indicate insufficient air. The cost of this would be almost altogether first cost of installation. One inspector can look after many in one day and need not visit each oftener than once a week. It is worthy of note that air chambers will also protect water meters from shock and breakage.

DESIGN OF AIR CHAMBERS.

The size of the air chamber, to be installed under the system suggested, would depend upon the size of the main to which it is connected, and upon the available room. Its diameter should be, if possible, two or three times that of the main, and its height 2½ to 4 times its diameter. The size of the neck and the connecting pipe to the main should be equal, or nearly so, to that of the main. At the main the connection should not be by a plain tee, but by a double branch elbow of easy curves. The reason for this is that the pulsations of flow and pressure would not then find their easiest path straight by, but would be deflected by the elbow, at least in part, into the branch. The return or equalizing flow from the air chamber would not come squarely against the back of a tee, but divided or deflected by the easy curve of the elbow.

The shorter the pipe from the main to the air chamber, the better is the action of the chambers. It would be best if mounted directly upon the main, but this would generally be impossible, except at pumping stations.

As air chambers are designed to resist shock, an elastic material is to be preferred in their construction; therefore steel plate should be used as much as possible.

RELIEF VALVES.

A relief valve is in fact a safety valve applied to a water pipe. Its action is to allow escape of water at times of excessive pressure. Its use is necessary to protect water mains from excessive pressure long maintained, where the air chamber can give relief only from temporary excessive pressure. It will also afford a measure of relief from water hammer, but on such an occasion it acts more slowly than an efficient air chamber and the initial pressure wave is higher. The excessive pressure of water hammer lasting but a brief period, the valve will open and close suddenly. If placed where frequently required to relieve water hammer it will in time pound itself or its seat to pieces. The operation of a relief valve necessarily implies an escape (waste) of water, and therefore its use for protection against water hammer is not to be generally recommended.

OPERATION OF CONSUMERS' POWER PLANTS.

Hydraulic elevators and various types of pumps in power plants of consumers draw intermittently from the water mains. These have quick-acting valves. Other plants have large check valves in their supply pipes or other quick acting valves. Sudden closing of these valves produces water hammer if the velocity of flow is considerable. The city should demand that every plant, no matter how small, which has any sort of quick acting valve, should provide an air chamber. This should apply even to house plumbing, where, in all good installations, it is now done.

The operation of reciprocating pumps of the "direct-acting" type is a prolific cause of water hammer because of the complete cessation of flow at the end of each stroke. Check valves are the most sudden in their closing of all valves. Especial care should be taken in both of these cases.

FOURTEENTH STREET PUMPING STATION.

It would appear from the discussion of the breaks that the large pump at Fourteenth Street Pumping Station is responsible for large fluctuations of water flow and water pressure and also of water hammer in the mains. Having but one air chamber and that too small it affords too little protection to the mains. The check valve, if closed suddenly, creates water hammer in the mains, which, although causing noise, might not be heard in the engine room. The variations in delivery give pulsations of pressure as well as flow and these not being properly reduced, because of the insufficient air chamber, were transmitted to a 48 in. main.

Referring to the topic water hammer the measure of the force of water hammer is stated to be dependent on the weight of the water whose motion is suddenly checked or accelerated. The main is 48 ins. in diameter and the variation in velocity even with perfect valve action, is considerable. Hence large fluctuations can be caused. The pulsations traveled through the entire system connected therewith and brought to light weak spots elsewhere.

The pump in question was shut down from Oct. 29 to Nov. 1, 1912, and some overhauling done on the valves, since which time no further leaks of consequence have been reported.

It is possible that this same trouble may recur in the future unless an air chamber of proper size is placed on the main at the station and an independent air compressor installed in the station for charging it and the present one.

REMEDIAL AND PREVENTIVE MEASURES

The following remedies are recommended: 1. In addition to present methods of inspection, test all pipes, fittings and valves where made for strength under water hammer as explained herein. 2. Protect the mains by installing air chambers in basements of public buildings. 3. Require use of air chambers in all plants wherever there is any quick closing valve of size equal to say one-quarter the area of the supply pipe leading into the premises. Require the air chamber to be installed anyhow when the supply pipe exceeds 4 ins. diameter. 4. Lay all new mains to line and grade and keep accurate records. Get levels of all portions of old mains whenever uncovered. Check them at every opportunity for displacement by soil movements. 5. Put a new air chamber on the main leading from the large pump at Fourteenth Street Pumping Station. This should be taller than the present one, and located on the side of the check valve next the main. 6. Install at every pumping station an independent air compressor for filling the air chambers. Provide every air chamber with a gage glass, if not already there, and with a light to make it possible to see, and lastly, require the engineer on watch to make periodic inspection of the air chambers in use.

Method of Lowering Mains Under Pressure and Cost of Laying Water Mains at San Diego, Calif.

(Staff Article.)

Occasion frequently arises to lower water mains because of regrading the streets in which the mains are laid. This work must usually be carried on without shutting off the water pressure. The present article describes the method of doing this work employed by the Department of Water of San Diego, Calif. The diagram of cost data included herein does not pertain to the lowering of mains, but gives the cost of cast iron water mains as laid in San Diego. The itemized unit costs cover the various sizes of cast iron mains in place.

At San Diego the adjustment of water mains to a new grade is not allowed to interfere with the supply to consumers. When brought to grade the mains are thoroughly tested for leaks while under full pressure. When the work is done by contract the Department of Water has 24 hours' notice before any work commences on the adjustment of any section of mains. The opening and closing of all gates and service cocks is done by Department employes. Service connections are handled entirely by the Department.

Figure 1 shows the connection between the high and low service mains by means of a reducing valve. These mains parallel each other for about 400 ft. They were lowered 5 ft. below the old grade in order to fit the new grade. Figure 2 illustrates the trench in which the pipe was lowered and also shows the pilasters back of each joint which supported the pipe.

After excavating the trenches as shown, a workman was placed at each pilaster and carefully picked it away, allowing the pipe to drop about 1 in. at a time. The various pilasters were, of course, reduced simultaneously so that the lowering was made as uniformly as possible. As a joint started to leak the calkers would calk it tight. In some cases the pilasters were moist to such an extent that they crumbled, and in that event jacks were placed under the main, and it was lowered by that means. This difficulty was encountered, however, on only a small portion of the work. The pressure on the high service main was 140 lbs. per square inch, and was reduced through reducing valves, as shown in Fig. 1, to 40 lbs. in the low service mains.

No difficulty has been experienced at San Diego in lowering a water main extending over a hilltop by the method of adjustment here described. While the regrading shortens the distance, it has been found that in making the pipe conform to the new contour it "takes up" sufficiently in each joint to accommodate itself to the new conditions. The mains of an entire block, including the services from the curb line, have been lowered without separating the lead connections. It was necessary only to lower the main, and the services dropped in the service pipe trenches without causing any trouble whatsoever.

Figure 3 gives the curves of cost of cast iron mains laid complete in the ditch. The reader should note the condensed form of

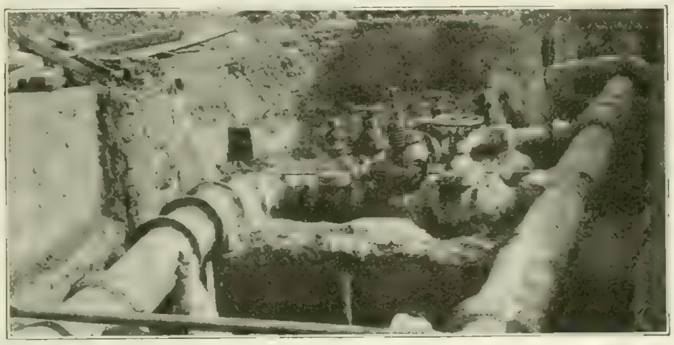


Fig. 1. View of Connection of High and Low Service Mains at San Diego, Calif, as Excavated for Lowering Under Pressure.

Fig. 2. View of Parallel High and Low Service Mains Ready for Lowering Under Pressure. Note Supporting Sections of Earth Back of Each Bell.

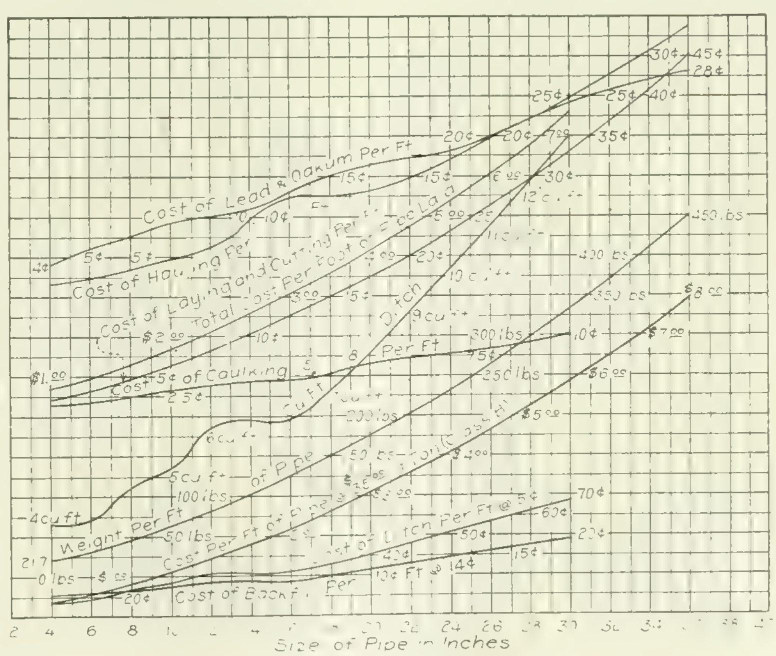


Fig. 3. Diagram Showing Cost and Other Data Pertaining to Cast Iron Mains as Laid Complete at San Diego.

[illegible] using one scale [illegible] the abscissae of all curves, but varying the ordinates of the several curves according to the nature and magnitude of the quantities [illegible].

ACKNOWLEDGMENT.

For the information here given we are indebted to Mr. H. A. Whitney, Hydraulic Engineer of the San Diego Department of Water.

Permanent Cotton Disc Sediment Records for Water and Sewage.

In spite of all that has been said and written on the subject of water analysis, and in spite of the fact that hundreds of samples of water and sewage are being analyzed every day, it still remains true that the results of the analyses are unintelligible to a large proportion of the people who are interested in the general subject and, it might be added, to a great many of those who make the analyses. The interpretation of a water analysis we have been told over and over again is a matter for the expert.

Yet there are some characteristics of water that are evident to layman and expert alike—the conspicuous physical characteristics of color and turbidity, taste and smell. Within recent years tests for these properties have been simplified and standardized, but simple as they are the results cannot be understood by one who does not know the scales of measurement. There is often needed some means of bringing to people's attention the character of a public water supply or the condition of water in a stream, or that of a sewage effluent, and this [illegible] appealing to something within their everyday experience, by presenting something that they can see or smell or taste. Such a something is dirt.

Every citizen desires a supply of clean water. If the water supply is dirty it fails to give satisfaction, and any method which will exhibit its dirtiness will appeal to a universal instinct and experience. We all desire clean waterways. We believe that the final effluents from sewage treatment works should be reasonably clean—otherwise why treat the sewage?—and any method which will exhibit the dirtiness of the water in a stream or in a sewer will prove a useful weapon in the fight for cleanliness which is being continually waged by sanitarians.

[illegible] forms of dirt found in [illegible] substances which pollute [illegible]. Some forms of dirt are in solution, or are present as colloidal substances, or as exceedingly fine particles [illegible] special methods of comparison and exhibition of results. But there is one form of evident dirt for which we have thus far had no satisfactory [illegible] form of suspended matter [illegible] measuring turbidity [illegible] the presence of the larger particles of obvious dirt. They are largely optical and the larger particles of suspended matter do not greatly interfere with the passage of light through water unless they are quite numerous, hence the [illegible] recorded. A simpler method with more [illegible] the needed. Such a [illegible] matter given being taken from a paper on this subject presented before the recent annual meeting of the American Society of Municipal Improvements. The paper was prepared by Mr. [illegible], Professor of Sanitary Engineering in Harvard University, and Mr. John W. M. Bunker, Instructor in Sanitary Analysis in Harvard University.

The method described is not original; it is an adaption of a method already in wide use for exhibiting and recording the amount of sediment in milk.

The method consists merely in straining a given volume of water, say one gallon, through a disc of [illegible] in. in diameter, and [illegible] the purpose. This is most conveniently done by means of a simple app[illegible] the "Wizard Sediment Tester" which is made by the Creamery [illegible] Manufacturing Company of Albany, New York. The cotton disc, which is the filter [illegible], is held between two supports of wire cloth in a cap attached to an ordinary glass milk bottle. The water to be filtered is placed in the bottle and allowed to flow out through the cotton disc. Filtration is hastened by increasing the pressure of the air within the bottle by the use of a simple air compressor operated by a hand bulb. In order to filter a gallon of water the bottle has to be filled four times.

This particular form of apparatus is not necessary. Other similar sediment testers are on the market. Nor is it necessary to use compressed air in filtering. A gravity flow will give the same result, or a flow hastened by centrifugal force or by the water pressure of the city mains. All of these methods have been used in the laboratory of sanitary engineering at Harvard University.

The coarse sediment in the water is retained in the pores of the cotton disc and remains as a permanent record which readily appeals to the eye and which by its appearance gives at once a good idea of the quantity and character of the substances present. For permanent record it is well to provide sheets or cards to which the cotton discs may be easily attached after drying by using a drop of mucilage or a little paste. The suspended matter does not change in appearance even after a long time, and does not shake off unless too much sediment is allowed to collect. For the sake of uniformity the writers suggest that the following table be accepted as a tentative standard for quantities of water to be filtered:

Standard Quantities for Filtration Through Cotton Discs.

Filtered water	One gallon.
Ground water	One gallon.
"Clean" surface water	One quart.
"Dirty" surface water	One pint.
Sewage and sewage effluents	One-half pint.

While the metric units of measurement are more scientific than the commonly used units of liquid measure, the latter are better comprehended by the public, and are more appropriate for this crude test.

The quantity to be filtered varies with different waters. It is advisable to filter the smallest quantity that will give a definite record, inasmuch as increased quantities tend to pile up the sediment on top of the first layer and therefore do not show small variations. A water rich in algæ will produce a definite stain on a cotton disc after one quart has passed through. A clean water does not leave any noticeable discoloration when this amount is filtered but may color the disc slightly when a larger sample is taken for the test. A typical crude sewage, on the other hand, will clog the disc when from 150 to 200 cubic centimeters have been strained, and more can be forced through only with great difficulty. The best [illegible] sewage effluent will give some discoloration when this quantity has been filtered, so that a ready comparison is possible.

The following are a few illustrations of the use of the cotton disc method for collecting sediment:

Samples of the sediment found in the tap water of Cambridge, Mass., have been taken [illegible] for more than a year. The cotton discs record the changes in the growths of algæ with striking picturesqueness. In summer when the chlorophyceæ are abundant, the stain on the cotton disc has a bright green color. During the ascendency of the cyanophyceæ, in the [illegible], the color is a slaty bluish green. The quantity of sediment fluctuates somewhat from day to day as the wind blows the algæ from one side of the reservoir to the other and as the algæ rise or fall in the water. In the spring the sediment has a brownish color, due to the presence of diatoms. Samples of water collected at different depths show very distinctly the distribution of the algæ in a vertical direction from [illegible] are most [illegible] with greater depths. The samples at the bottom [illegible] colored sediment, due to the influence of the organic matter and iron found in the stagnant layers.

Any departure from the normal distribution is quickly noted by this method of examination, as is shown by the set of discs made at Lake Cochituate on Oct. 2, 1914. This series shows a steady decrease in depth of color down to the transition zone, at which point there is a darkening of the discs, due no doubt to a concentration of organisms at this point. Immediately below this the organisms fall off and again increase gradually until the forty-five foot level is reached. The last fifteen feet, from the forty-five foot depth to the bottom, shows the presence of a different kind of sediment, darker in color and more abundant in quantity, in which iron is present in large amounts.

On some days the amount of sediment in the Cambridge water has been found to suddenly increase, owing to the disturbance of deposits in the pipes. The character of this sediment is quite different from that normally found. Again, samples taken from the hot and cold water taps often show marked differences, the hot water sediments showing the presence of iron rust derived from the action of the water on the heater or the piping in the houses.

The method is particularly well adapted to show the effect of filtration on dirty water. During the past year an experimental mechanical filter has been in operation at the Laboratory of Sanitary Engineering, Harvard University, and daily records have been kept of the sediment in the water before and after filtration. The results are often more striking than the figures which show the removal of bacteria. On certain days when the filter was not operating at its best the cotton discs through which the filtered water passed were slightly discolored.

Mr. Clifton L. Rice, who has been recently assisting Mr. Frank A. Barbour in a series of valuable experiments on the purification of the water at Lowell, Mass., has used the cotton disc method with marked success. Mr. Barbour's report shows photographs of the sediment collected on the cotton discs. The problem there is the removal of iron and manganese from the water. The efficiency of the purification processes which it is proposed to employ is well indicated by the stains made by the raw water and the absence of stain when treated water is passed through the cotton.

ADVANTAGES OF COTTON DISC SEDIMENT RECORDS.

The cotton disc sediment record has the following advantages:

1. It can be made very easily and quickly.
2. It can be made by the laborer and the office boy, as well as by the engineer or chemist.
3. It is inexpensive.
4. The records are picturesque and easily understood by everyone.
5. Within limits the records are quantitative as qualitative. They serve well for comparing samples from the same place on different dates.
6. The records are permanent.
7. The records may be conveniently mounted for preservation.
8. The records may be photographed.
9. A relatively large volume of water is tested.
10. It is a valuable supplement to the regular water analysis.

$3,000,000 for Sewerage at Baltimore, Md.

A $3,000,000 loan for sewerage purposes will be voted on by the citizens of Baltimore, Md., at an election next month. The funds will be used to complete the east and west low level interceptors, the Jones Falls interceptor, the high level interceptor and to extend the sanitary lateral sewers and storm-water drains in the northeastern, southeastern, southwestern and northwestern sections of the city and the suburbs of Arlington, Woodberry and Govanston, thereby completing the sewering of the entire built-up portion of the city with sanitary sewers and storm-water drainage.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design and to Methods and Cost of Construction

Volume XLII. CHICAGO, ILL., NOVEMBER 4, 1914. Number 19.

An Important Series of Articles on Wooden Stave Pipe.

In this issue we publish the first of a series of five articles under the general caption of: The Use, Design, Construction, Cost and Durability of Wooden Stave Pipe. The first article pertains, chiefly, to the uses and misuses of this kind of water pipe. The evolution of the pipe is traced in short space and the modern types are illustrated and described. The principle uses of machine-banded and continuous stave pipe are enumerated, and the proper application of the pipe to these uses is critically discussed. An especially valuable portion of the article relates to some of the abuses and misuses of wooden pipe. Several examples of this character are given, including faulty design, poor location, failure to provide for or to prevent water hammer, the use of wood pipe for lines where the flow is intermittent, failure to provide proper protection, and failure to provide air valves and standpipes.

The second article will relate to the durability of wooden pipe. In this connection the staves are discussed with respect to kinds of wood, design, decay and earth covering, and the bands with respect to material, coating and design. The decay of staves is further discussed with special reference to the location of the pipe line, kinds of earth, care in construction and the saturation of the staves. This article also discusses the misuses of this class of pipe.

The third article will cover the design and selection of material; the fourth the organization of construction forces and construction methods; and the fifth and final article will give numerous detailed data on the cost of pipe lines constructed of this material.

The articles, which are the result of a correspondence extending over a period of three years, are contributed by Mr. Andrew Swickard, hydraulic engineer, Palo Alto, Calif. Mr. Swickard has had more or less to do with wooden pipe for the past ten years. Four and one-half years of this time were given exclusively to the design, manufacture and installation of wooden pipe. During this period he had charge of the installation of pipe in California, Montana, Idaho, Utah, Nevada and Mexico. He built continuous stave pipe up to 10 ft. in diameter, and is also experienced in laying machine-banded pipe. He is also the designer of machinery for the manufacture of the latter class of pipe. This varied and extensive experience qualifies Mr. Swickard to write authoritatively upon this subject. A reading of his first article will show that he writes with the impartiality of the engineer who, while appreciating their merits when used properly, also recognizes the limitations of materials of construction when they are subjected to misuse.

Cost Data on Bridge Foundation Work.

To be most effective the keeping of force and cost accounts and the analysis of these data require careful planning in advance of construction and a certain amount of educational work by those in charge. The mistake is often made of permitting accounts to be kept by inexperienced and poorly paid men who at best have little interest in the work Unless those who analyze the accounts are thoroughly familiar with the manner in which they have been kept, as well as with the construction methods used and the conditions under which the work is done, the unit costs can hardly be expected to have much value. Many cost data are of limited application because items involving both labor and materials have been combined instead of being kept as separate accounts. To be of definite value to engineers and contractors contemplating similar work it is necessary that the conditions under which the work was done be fully stated. Too often such items as weather and labor conditions, delays, and equipment used are not fully recorded. In the past engineers and contractors have not often permitted cost data on all parts of an important job to be published, even though they may have given out data on certain items of the work. There is a growing tendency, however, to make public detailed information, instead of restricting its use to one organization, as it is being realized that the personnel and efficiency of any organization are the real factors making for success.

In our issue of Oct. 21, 1914, we published an article on the design of the substructure of the double-leaf trunnion bascule bridge over the Chicago River at Chicago Ave., Chicago. In the "Bridges" section of this issue there appears an exceptionally complete article on the labor costs of constructing each part of the substructure of this bridge. The foundation work was especially complicated and the cofferdam construction difficult due to unusual conditions. The article is contributed by the resident engineer in charge of construction, a man who has had considerable experience in keeping and analyzing cost data. The data given represent actual labor costs—no account being made of the cost of materials. Each item of the work is treated separately, and the conditions under which the work was done are carefully stated. We believe the article contains the most complete and carefully analyzed cost data ever published on work of this character. An interesting comparison is made between the price bid for each item of the work and its actual cost to the contractor. The data are presented in such a manner as to expedite their use in estimating the cost of similar work.

The Sewerage System of the Panama-Pacific International Exposition.

The sewerage system for the Panama-Pacific International Exposition, like all other engineering features of the exposition, was designed to serve its purpose for a short and well-defined period of time at a minimum cost. It is said that exposition engineering is becoming a specialty. This is brought about by the fact that expositions are of short duration and yet their constructional features must be sound and serviceable, although produced at the lowest possible cost. Consideration of these conditions has led the editor to conclude that the designer of Dr. Holmes' "wonderful one-hoss shay" would have made, with the proper special training, a highly successful designer of engineering works for expositions. Thus the ideal sewerage system for the Panama-Pacific Exposition would be one which would be serviceable while the exposition is being held and which would fail completely at the close of the exposition. This is literally true, since this particular sewerage system will be torn up as soon as it has served its purpose.

Thus, as our contributor states, the problem to be solved in designing this sewerage system was entirely different, as regards selection of the materials of construction, from the design of a system to serve a municipality. It is worthy of note, however, that with regard to the selection of materials the exposition sewerage problem is quite similar to construction-village sewerage. It occasionally happens that a power dam, or other extensive piece of work, must be constructed at a point remote from cities. In such cases, where the work is likely to last a year or two, small villages of unpretentious houses have been constructed along streets laid out in the temporary townsite. Such streets sometimes contain sewer and drain pipes. It would seem that in such cases pipe lines and appurtenances of the materials employed at the exposition under consideration would adequately serve at a minimum of expense. Aside from the lack of permanency in this design, however, the solution of its hydraulic problems is of interest to all classes of sewerage engineeers.

The system is more extensive and more expensive than one would at first thought suppose. We publish a map of the exposition buildings and grounds which shows the sewer layout and at the same time gives the reader an idea of the extent and arrangement of the various exposition palaces. The system will cost about $141,000. The area sewered is about 485 acres. Thus the cost is about $290 an acre, though this figure does not include the cost of catch basins and the pipes connecting them to the system. The total length of the system is approximately 28.4 miles. Many of the special constructions illustrated and described are very interesting examples of versatility in design.

County and Municipal Engineering: A Developing Field for Civil Engineers.

The field of county and municipal engineering offers no inducements to the seeker of wealth. The engineer of brilliant intellect that seeks to conquer mountains and bridge untamed rivers and expects to reap rewards commensurate with his ability had best seek some other field for the utilization of his gifts. But to the man of sound judgment and moderate but well balanced attainments in the details of the work of a civil engineer the field of county and municipal engineering has always proved attractive and will become more so in the future.

The welfare of the modern city and the modern countryside is yearly becoming more dependent upon those structural works which it is the province of civil engineers to construct and maintain. Moreover the proper administration of civic affairs requires such a detailed knowledge of these mechanical features that a public official is no better fitted for their oversight than by the training received by a civil engineer. That the public is realizing this fact is well illustrated in the appointment of many civil engineers to positions as city managers, county road superintendents and city and county engineers.

The salaries attached to such positions have always been low and will probably always remain low. But the number of openings is steadily increasing and the certainty of continuous employment is perhaps on a firmer basis than in any other branch of civil engineering. A field for service as operating engineers for structural works is developing. Examples of the positions in this field are those of county engineers and road superintendents, waterworks superintendents and sewage plant superintendents.

Civil service and the commission form of government are factors that have contributed toward eliminating much of the uncertainty of employment that heretofore has been the chief objection to such work. And yet with its uncertainty the engineering work of cities and counties must be performed regardless of wars and financial conditions and the engineer well versed in the details of such work has no real excuse for being idle.

The field is, however, for the general practitioner rather than the specialist. More than mere technical proficiency is required. Tact, executive ability and a disposition to remain contented in one position are also necessary. The duties of men employed in this field while broad in their scope and demanding ingenuity and the ability to use funds—frequently insufficient funds—to the best advantage, are comparatively simple and routine from an engineering standpoint. For this reason, perhaps, salaries are low.

For the engineer that takes pleasure in designing engineering structures and working out his designs on the ground in person, for the engineer that delights in the organization of construction work and the trying out of his individual ideas, for the engineer that is content to work for a moderate salary, county and municipal engineering offers attractions. But for the man that commercializes his ability and demands adequate financial return the field offers few inducements.

The Hydraulics of Drainage Irrigation and Other Channels.

To the Editors: The following errata are found in the article, "The Hydraulics of Drainage, Irrigation and Other Channels," published in the Sept. 23 issue of Engineering and Contracting.

Page 285, column 3, line 6 from above, substitute $\frac{2g}{Z_1}$ for $\sqrt{\frac{2g}{Z_1}}$.

Page 286, Table III, under the heading D^5 substitute 133.85 for 143.85 and 182.59 for 184.59.

Page 287, column 3, in the formula for I the fall in feet per 1,000 ft., substitute the exponent 2 for 2/5.

Page 288, column 1, line 29 from above, insert "in earth" after the words "for channels."

Page 288, column 1, line 30 from below, substitute "planed" for "plaster."

Page 289, column 2, line 23 from above, substitute $48 + 12 \times 2 = 72$ for $8 + 2 \times 12 = 72$.

Page 289, column 2, between lines 9 and 10 from below, after the words, "inserting these values" insert the equation for the loss of head H found in line 14 from above and give the first term between brackets the exponent 2.

Page 290, column 3, line 29, from above omit the words rivers, etc.

Page 290, column 3, under the heading (a) substitute $V_1 - \frac{1}{10}$ for $V_1{}^{\frac{1}{10}}$, and $V_1 - \frac{1}{20}$ for $V_1{}^{\frac{1}{20}}$.

Respectfully,

Louis Schmeer

San Fernando, Cal., Oct. 7, 1914.

WATER WORKS

The Use, Design, Construction, Cost and Durability of Wooden Stave Pipe.

I.

The Uses and Misuses of Wooden Stave Pipe.

Contributed by Andrew Swickard, Hydraulic Engineer, Palo Alto, Calif.

Wooden stave pipe has, after a demonstration covering a period of about 35 years, won a permanent place among the utilities of mankind. Its use is limited by the comparatively low pressures that it is practical to construct it to resist. It, therefore, is not supplanting iron pipe, but has taken a place alongside of iron to be used when conditions and circumstances make it preferable. Its comparative low cost, including interest and depreciation, ease of transportation and construction and repair, durability when not misused, and constant carrying capacity, makes it a very desirable form of construction.

The modern types are the result of a process of evolution, beginning possibly with the bored-log pipes used before the advent of cast-iron pipe, and coming down through the various forms of stave pipe used in the penstocks of some of the early New England and Canadian mills. The staves in these penstocks were formed by hand, and bound into a pipe by means of metal bands of various shapes. In some cases the staves were tapered and formed into sections of pipe by driving on solid metal rings; the tapered sections thus formed being joined by driving the smaller end of one section into the larger end of another, thus forming a telescopic joint. In others the ends of the staves were butted together, and these butted joints staggered around the pipe, so as to form a continuous tube of uniform diameter. This form, necessitating the use of bands that could be tightened at will, is the immediate progenitor of our present type, known as "continuous" wooden stave pipe.

The Denver Union Water Co., of Denver, Colo., with Chas. P. Allen as chief engineer, made the first extensive use of the continuous stave pipe. Their experience resulted in details that were patented in March, 1887. Later other details were developed, principally for the purpose of circumventing Allen's patents. Since the expiration of the patents the Allen details have been adopted generally.

Following close after the successful development of the "continuous" pipe, another type made its advent. Because of the nature of the method of banding this latter type is known as "machine banded" pipe. This type has been successfully perfected and is now being extensively used. Both the continuous and machine banded types are illustrated in Fig. 1.

The staves of the two types are formed in the same general way. The faces are concentric and the edges radial. For the "continuous" type the edges are milled smooth, but for the "machine banded" the edges are tongued and grooved. The purpose of the tongues and grooves is to hold the staves in their proper position while the banding is being done. For the "continuous" type the tongues and grooves are not practical, because of the difference in the method of construction, and also because the tongues would be broken and slivered during shipping, hauling, and distributing along the pipe line, and thus become more of a hindrance and nuisance than a help. Saw kerfs of uniform position, width and depth are cut across the ends of each piece of stave of the "continuous" type. During construction metallic or wooden tongues are inserted in the kerfs, being of a length and width such that, when the staves are in final position, these tongues jam slightly into the bottoms of the saw kerfs and into the edges of the adjoining staves. This means that the tongues must be

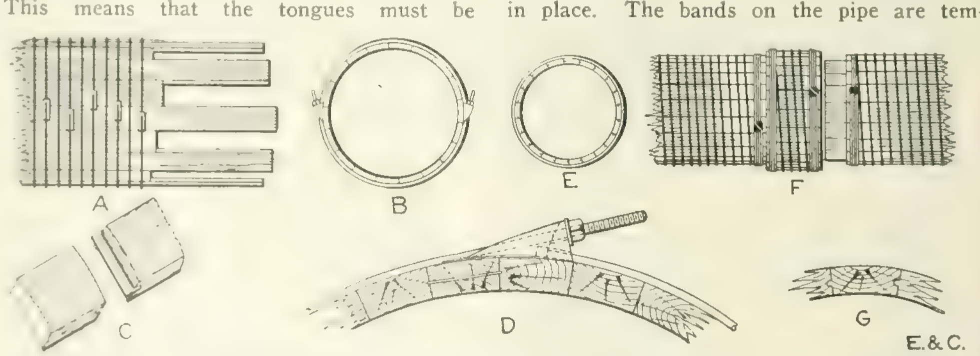


Fig. 1. Sketches Illustrating the Two Types of Wooden Stave Pipe: Continuous Stave and Machine Banded.

A, side view of continuous stave; B, cross section; C, detail of stave joint; D, showing shoe in position; E, cross section of machine banded pipe; F, side view showing collar in position; G, showing tongues and grooves.

slightly wider than twice the depth and slightly longer than the length of the saw kerf. Butting the ends together with the tongues inserted, and alternating the butted-joints thus made, has the effect of making, after banding, a continuous rigid tube with each stave continuous throughout the entire length of the pipe line. The staves are bound together by individual bands made from round rods and are, in effect, long bolts bent around the pipe; the two ends being joined by a "shoe" of a form that permits a wrench being worked on the nut and thus tightening the band. In the case of the "machine banded" the tube is finished in sections, from 10 to 20 ft. long, that are joined together by means of short wooden collars, and sometimes iron collars, into which the abutting ends of the various sections are driven. The staves of each section are of uniform length and are bound together by round steel wire or flat steel, wound on spirally from one end to the other, leaving unbound a few inches at each end, or a length equal to one-half the length of the connecting collar. These ends are turned down, forming tenons that fit snugly into the collars.

The material for the "continuous" type is shipped in the "knock-down," distributed along the pipe line location and is constructed in place, as illustrated in Fig. 2. Figure 2 is a photograph of a 66-in. double line of pipe built for the Sierra and San Francisco Power Co. The horizontal crest in the background is the top of an earthen dam about 60 ft. in height. The water is carried through the dam in two steel pipes. Joined to these are wooden pipes that extend, for about 2,400 ft., to the steel header at the upper end of the steel pipes leading down to the power house, about 1,400 ft., in elevation, below. The pipe as shown has crossed a sag and one line is going down into another sag, while the other is on the crest, but ready to start down. The lower pipe shows a section of staves partly in place. The bands on the pipe are temporarily placed, they being closer spaced over the short length in which the joints occur. A crew follows, putting on more bands and spacing them properly. The tools used in constructing continuous stave pipe are few in number, and very simple.

The machine banded type is necessarily factory made. The band is wound on under high tension and, especially for the larger sizes, heavy machinery is required to do this. The cutting of the tenons on the ends of the sections of pipe, so that they will fit accurately the connecting collars, requires quite an extensive piece of machinery.

The "continuous" type is made in sizes from 12 ins. in diameter up. The largest size that has been constructed to date for practical purposes is 13½ ft. The larger sizes must be supported by cradles, and with cradles properly built there is no reason, from the contractor's point of view, why larger pipe cannot be constructed. The "machine banded" type is made in sizes from 2 ins. in diameter to 48 ins. and possibly 60 ins. But few manufacturers make pipe as large as 48 ins.; 24 to 32 ins. marks the present limit for most of them.

The staves for most of the pipe being built is manufactured from California redwood (*Sequoia sempervirens*) and Douglas spruce (*Pseudotsuga douglasii*), generally known as Douglas fir, and locally as yellow fir, Washington fir, Oregon pine and Canadian pine. Long-leaf yellow pine (*Pinus palustris*) has been used but to a limited extent.

Redwood trees furnish long, straight, close-grained timber, and each tree furnishes quite a large percentage of clear lumber, free from sap wood and other objectionable features. The wood is highly durable and is almost ideal material for pipe staves. The market price of redwood is somewhat higher than that of the other kinds of lumber used. The redwood tree is found only on the Coast Range Mountains of California.

Douglas spruce trees furnish long, straight, rather coarse-grained timber, containing some pitch seams, but making excellent pipe staves. In comparison with redwood it is not so durable, and clear material is not so easily obtainable. Each tree furnishes a comparatively small quantity of clear lumber, but most of the knots in fir less than ¾ in. in diameter, are tight, while in redwood they are loose. The sap wood of the fir has considerable strength, and is not objectionable in pipe staves provided it is exposed only on the inside of the pipe. The sap of the redwood is of little value and should not be permitted in pipe staves at all. The Douglas spruce forests are extensive. The trees grow very large in the northwestern Pacific States and in British Columbia.

Long-leaf yellow pine furnishes long staves, but knots must be acceptable. The wood is heavily laden with resin, which flushes out under water pressure, thus permitting considerable water to be lost by oozing through the resin ducts. These trees grow in extensive forests in the coast regions from North Carolina to Texas.

The bands of the "continuous" type are made from round rods, of either Bessemer or open-hearth steel. The latter will probably last longer because of its greater purity. For sizes of pipe up to 54 ins. in diameter each band is usually made in one piece, and for larger sizes, in two pieces. When made in two pieces, one is threaded at both ends and the other is headed at both ends. The best thread is one that is cold-rolled. Threads can be cold rolled on good material, only. The cold rolling process strains the material to such an extent that the strength of the band in the threaded part is increased over that of the body of the rod. Thus we secure the advantages of threads cut on upset ends, and none of the defects due to the process of upsetting.

The shoes, or band couplings, are malleable iron castings. Each size of pipe requires a specially designed shoe. The shoe should be shaped so as to fit accurately the surface of the pipe. Where bands made in two parts are used two shoes are required for bands.

The "machine banded" type is bound by galvanized soft steel wire, either flat or round; the round is used much more extensively than the flat, for reasons that will be given in a later article.

The outside of the "continuous" type is rarely given a coat of paint or other protective coating. Anything forming a layer or coat over the surface will not adhere, because of the percolating of the water through the staves. Any of the penetrating preservatives would be leaked out. Most of the manufacturers of the "machine banded type, coat the outside of the pipe with a hot asphaltum preparation, and then let the pipe roll from the bath down an inclined bank of sawdust. The coat of sawdust prevents the asphaltum from running and sticking to anything it touches. This coating gives the pipe an exceedingly durable appearance, but aside from protecting the wood from the deposit of decay spores, for a comparatively short time, is not of much value as a wood preservative. The coating is primarily for protecting the banding, and being put on in this manner resists abrasion during transportation and handling much better than when the wire only is coated with asphaltum. If the galvanizing on the wire is heavy and of good quality, and is not broken or injured in any way during manufacture, transportation, or installation, then the asphaltum coating is of little value. But the galvanizing is almost sure to be broken in places and the coating will afford protection. One manufacturer of redwood pipe does not coat even the bands, claiming that, by requiring the galvanizing to stand a severe copper test, galvanizing is secured that amply protects the wire.

The continuous type is used mostly for conveying water to hydro-electric power plants, to cities and towns, and to lands for the purpose of irrigation; and occasionally is used for outfall sewers. Nearly all the largest pipe that has been built is in connection with power plants using large quantities of water under low pressure. Nearly all these large installations are found in regions where there

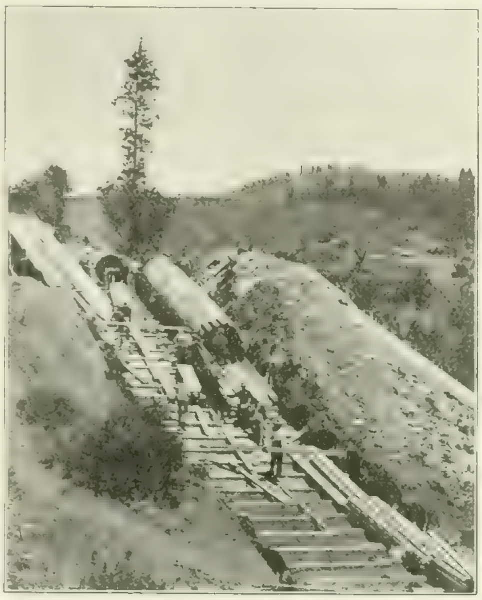


Fig. 2. View of Double Line of 66 in. Continuous Stave Wooden Pipe Under Construction for Sierra and San Francisco Power Co.

is an abundance of water. For plants operated under high heads, only the upper end of the pipe line can be made of wood. A 200 or perhaps 250-ft. head would be the maximum for which it would be practicable to build this type and there might be conditions that would make it advisable to fix the limit at even less than 200 ft. Usually economic considerations fix the maximum head allowable. For city and town water supplies, the pipe lines are generally smaller in diameter and much longer in length. The maximum head would probably again be determined by economic considerations, and if not, the maximum head that it is practical to build pipe to stand would determine the limit. If the topography will permit, the location should be such that the pressure be kept moderately low, for the cost increases rapidly with an increase in pressure. The pressure, however, should not be lower than that necessary to secure a thorough saturation of the staves. When a pipe line is located over undulating country, those portions of the line that dip below the allowable maximum head can very readily be filled in with steel pipe.

Wooden pipe should be used advisedly in connection with irrigation projects. The pipe should be full of water always, and as irrigation systems are not, as a general thing, operated continuously, there are likely to be periods when the pipe would be empty for a considerable time. Inverted siphons on the main supply line would be about the only portions of the conduit that it would be advisable to build of wood.

Out-fall sewers are often through marshy ground where foundations are poor. Here the continuous type can be used to advantage. The flow being more or less intermittent and the staves not being thoroughly saturated from within, the water in the ground will saturate and prevent the staves from decaying, and the rigidity of the pipe will bridge over soft places, and its elasticity will permit of considerable settlement without injury. If the sewer is through salt marsh, the bands will corrode rapidly. About six or seven years is the maximum life of bands in salt marshes, hence frequent renewals of bands would have to be figured upon.

In addition to the uses of the continuous type the machine banded type has one other that is quite important. Because of the character of its construction it can easily be connected to gate valves, tees, crosses, elbows and hydrants, and is therefore largely used in the water distribution systems of cities and towns. In order to take care of water hammer it should be banded extra heavy. Sections under electric railway can be banded with copper wire to prevent electrolysis of the banding. Service connections are readily made by screwing brass nipples into a hole of the proper size cut in the staves, or better by an outlet flange, put on when the pipe is being banded. The flange can be secured by bolts and by the bands also. When the nipple is used the wood will soften around one made of iron, but a brass one will remain tight.

It is practicable to make machine banded pipe that will stand a higher pressure than will the continuous stave pipe. The reason for this is that the banding can be closer spaced on the first than on the latter. The width of the shoe used to couple the individual bands determines the minimum spacing on the continuous type. The width of the shoe being about 3½ times the diameter of the rod, and with the adjacent bands against the shoe, the minimum spacing center to center would be 2¼ times the diameter of the rod, while with the machine banded type there is no such cause limiting the spacing. When, on any particular piece of work, the cost of wooden pipe reaches the cost of steel pipe, then it is probable that the wooden pipe will end and the steel pipe begin, even if the minimum spacing bands on the wooden pipe has not been reached. However, considerations other than the cost alone would probably influence the change from wood to steel, or vice versa.

SOME EXAMPLES OF THE MISUSE AND DEFECTS IN DESIGN OF WOODEN STAVE PIPE.

In the past a number of pipe lines have been built of wood under conditions and circumstances that should have precluded its use. This fact, in connection with improper design, poor location, neglect properly to protect the pipe, and neglect to furnish safety devices in the way of air-valves, and standpipes, has in isolated cases had the effect of bringing wooden pipe to some extent into disrepute. Fortunately, for the reputation of a good form of construction, the misuses have not been frequent, for the opponents of wooden pipe have not been slow in using the results of even the most aggravated cases of misuse as an argument against wood pipe in general. The want of a proper appreciation of the limitations that hedge about its adaptibility to general use and a want of knowledge of the effect of various conditions of soil, and other natural phenomena encountered along the location, and of the principles entering into the design, has been responsible for nearly all misuses. The fact that the demand is on the increase is, in itself, a wholesome sign of its usefulness.

Faults in design may be several, but for the most part they would result in small leaks, inconvenience and extra expense during construction, but there are three points that are important and should be looked into carefully; namely, the thickness of the staves, the strain in the bands, and the bearing strength of the part of the surface of the staves in contact with the bands.

The thickness of staves must be such that when the bands are properly tightened the result will be a tube of sufficient rigidity to withstand any outside pressure that will come upon it, such as the weight of the backfilling when the pipe is laid in a trench, pressure due to a partial vacuum, and the reaction of the supporting bed due to the weight of the pipe and the weight of the water within the pipe. Hence, staves too thin may result in a collapsed pipe, while, on the other hand, if they are thicker than necessary, the degree of saturation of the outer surface is less than with thinner staves, and the resulting conditions are better for the starting and growth of decay. The saturation of the staves with water is the only effective preservative. Its effectiveness as a preservative depends on the approach of the degree of saturation to that resulting from complete immersion in water. Any feature of design or location that unnecessarily interferes with the saturation of the staves should be avoided. Decay once started makes rapid inroads; prevention of the beginning should have every attention, because at the best decay will make some progress.

Water pressure, and the compression of the staves produce the strain on the bands. Determining the numbers of bands per foot is an easy matter, when the amount of the permanent compression in the staves has been settled upon, and the water pressure is known.

In a pipe under pressure the water is constantly pushing the staves against the bands, and unless the supporting strength of the narrow strip indented by the band is ample, there will be a movement outward and leakage will occur unless the widening of the indented strip provides sufficient bearing before leakage occurs. But if the increase in depth and width of the indentation is sufficient to break the fiber of the wood, then irreparable damage has been done. After a comparatively short time the wood will begin to soften where the fiber has been broken, and the seams will open and the pipe become worthless. If the pipe is of the "continuous" type extra bands can be put on after the defect is discovered and the pipe saved, but this cannot be done with the "machine banded" type. Too much emphasis cannot be put upon the importance of providing enough bearing surface under the bands. It is the most important element entering into the design, and the one most often overlooked. The bearing surface controls the band spacing on all the smaller sizes of pipe. For ½ in. bands it controls for a pipe 30 ins. in diameter.

As an example: A few years ago an important city on the Pacific Coast advertised for bids for furnishing the material and constructing a pipe line of considerable length; the pipe to be 18 ins. in diameter. The spacing of the bands was specified among other details, and while the number of bands per foot were sufficient to withstand safely the strain produced by the water pressure specified for use in the test, the bearing surface under the bands was far from being sufficient to prevent the staves from moving outward, and the seams opening. The contractor did not understand such matters and went ahead in good faith with the construction. When, finally, the test was made the seams opened and the water came out in streams. The bands were tightened further but made no appreciable decrease in the leakage. The newspapers of the city published the conclusion of the City Council that wooden pipe was of no value. The fault was entirely with the City Engineer.

Other defects in design might occur as follows:

(a) Staves not of proper dimensions to provide for the reduction in width due to compression produced by tightening the bands. When the band spacing is close the resulting diameter of pipe might be considerably less than specified.

(b) If the bands are not of proper length either there will be much trouble in getting them on the pipe, or the thread will be used up before the pipe is properly cinched.

(c) The shoe might be made so short that the maximum pressure under it will exceed the supporting strength of the wood, or the shape might be such that the bands will slip off during the process of tightening.

If a pipe line is laid on a uniform grade under little or no pressure, the top staves will decay in an exceedingly short time. If laid to a uniform grade it should be arranged so that there will be a pressure of not much less than that due to a 40-ft. head. This will insure a degree of saturation that will prevent decay. Some years ago the city of Cheyenne, Wyo., built about 9,000 ft. of pipe, using Douglas fir staves. The upper 4,000 ft. of this line was never full of water. As a result the decay of the staves was rapid, and within a comparatively short time was replaced with sewer pipe.

If the location is over undulating ground care should be taken that the pipe does not come close to the hydraulic grade line at the high points. If it does rapid deterioration at these points is certain. The city of Astoria, Ore., installed about 7½ miles of 18-in. wooden pipe, located over undulating ground. At the high points the pipe came practically to the hydraulic grade line, and at certain stages of the water supply did not run full at these peaks. The result was extensive renewals within ten years. An undulating location puts most of the pipe under sufficient pressure to insure a long life to the staves.

Trees of rapid growth, requiring much water, will entwine their roots in masses about a wooden pipe. They sap up all the moisture, reducing the degree of saturation of the staves to a point favorable to decay, and a rapid deterioration takes place. A few years ago about ½ mile of redwood pipe was laid at Long Beach, Calif.; a portion of it being laid in ground overgrown by willows. Within 3½ years the portion through the willows was nearly gone, while the other portion was in perfect condition. Avoid locating in such ground. If necessary to cross this sort of a place keep the pipe above ground.

One of the important elements of a good location is the keeping of the pipe away from conditions that will hasten its destruction.

To subject a pipe to a pressure greater than that for which it was banded is inviting disaster. Care should be taken to keep the discharge end of a wooden pipe always open unless the pipe has been banded for the full static head or more. Where a pipe is liable to sudden increases in pressure such as that due to sudden changes in velocity, liberal allowance should be made for such probable increase in pressure and the pipe should be banded accordingly. Under similar conditions, pressure in excess of the normal due to sudden changes in velocity are not as great in wooden pipe as in iron, because of the elasticity of the wood, but if not taken into account in the design trouble and perhaps failure is the result. The city of Porterville, Calif., included some wooden pipe in their water works distribution system. The static pressure was nearly as great as that for which the pipe was banded. In a very short time the pipe was worthless, and was replaced by iron.

One of the first things to determine in connection with a proposal to use wooden pipe is the constancy of the flow. If the flow is to be intermittent wood should not be used. However, if the location is over undulating ground, the portions of the pipe across the lower parts, which will remain full of water during the period of no flow, can be made of wood, and the remainder of iron. But whatever is done, wood should not be used in any portion of the pipe that is going to be empty part of the time.

Whether or not a wooden pipe should be buried or built above the ground surface is not categorically answerable. Liability to damage from fire, falling objects, or wantonness might make it necessary to put the pipe under ground. The character of the soil in which it is to be buried might also be a determining factor. If built above ground the pipe would give best service if protected by a timber covering. If to be buried the top of the pipe should be about 4 ft. below the surface. A pipe line in northern California that had been in disuse for 2 or 3 years was uncovered for repairs. The pipe was laid over uneven ground, and at places was entirely above ground and at other places buried to considerable depth. Where the pipe passed from above to underneath the ground surface, the deterioration was the greatest immediately under the surface and decreased as the depth of covering increased. The same phenomena is observable in a fence post; greatest amount of decay, just below the surface, but decreasing downward. The staves of a pipe buried in a heavy soil will probably last longer than if above ground, but the bands on the pipe above ground can be cared for and made to last much longer than they will under ground.

A wooden pipe line *must* be provided with means of letting air in and out of the pipe. If for any reason there is a discharge of water along the lower portion of a pipe line that suddenly increases the velocity of the water, partial vacuums will form somewhere in the pipe, causing pressure from the outside that might be sufficient to collapse the pipe. It sometimes requires an accident of this kind to impress the necessity of air-inlets.

At the high points of an undulating line entrapped air must be allowed to escape or decay will attack the upper staves at these points. Standpipes will best let air in or out, if circumstances will permit of their installation; otherwise valves that will serve the purpose must be provided.

A power company operating in the Sierra Nevada Mountains (California) installed a pipe line made up of 48-in. wooden pipe at the upper end, intermediate part of steel, and the lower end of cast iron. A section of the cast iron pipe burst, resulting in a collapse of the wooden pipe; there was no way for air to get in. The Astoria pipe, referred to above, was furnished with check valves at the high points. They would let air into the pipe but not out of it. When the pipe was running partly full the top staves at these points would tend toward a condition of moisture under which decay would thrive; when the pipe was full the entrapped air would prevent the thorough saturation of the staves. Without doubt the two circumstances combined caused the renewals that were made after 10 years' service.

To install a pipe under conditions and circumstances for which it is not suitable and to neglect details that tend toward a longer life are misuses. Some of the instances of misuse mentioned in the foregoing that particularly affect the durability of the pipe will be taken up more in detail in the article to follow, entitled "Durability of Wooden Pipe."

Method Employed In Connecting Suction Tunnel to Pumping Station Wet Well Without Draining the Latter.

The construction of a suction shaft and tunnel and the connection of the latter to a wet well in service were special features involved in the improvements made in 1913 to the Springfield Avenue Pumping Station of the Chicago water works. The present article describes the method of handling this work.

The work was made necessary by the installation at this station of a new Allis-Chalmers 25,000,000-gal. turbine-driven centrifugal pump. The city provided the foundations for the pump and also the suction shaft and tunnel to a connection with the wet well which supplies the four other pumping engines in operation in this station.

The method employed for the connection of the new suction tunnel to the station wet well consisted in placing a curved cast iron shield in the wet well. The edge of this shield was grooved and fitted with a 2½-in. rubber fire hose, acting as a gasket to permit a water-tight connection between the shield and the masonry wall of the well. On the upper end of the shield were two flanged connections, one for the 8-in. pipe extending to the level of the gallery on top of the wet well at an elevation plus 8, and the other for a 24-in. pipe extending to a point just below a 24-in. horizontal suction pipe, which supplemented the main suction of old pumping engine No. 4. It was expected that the head of water caused by pumping the water out of the shield through either one of these pipes would give a pressure sufficient to hold the shield in place. The empty space between the shield and the wall would then give opportunity for making a connection between the old brick masonry of the well and the new concrete lining of the tunnel.

The material underlying the foundations and pit floor of the pumping station is solid rock. It was therefore necessary to blast out the shaft and tunnel. The foundations of engines previously erected, between which the shaft and tunnel were to be built, are 20 ft. apart and rest directly on the rock. Both of these pumping engines were to be kept in continuous operation, and extreme precautions were necessary.

Work was started at the station May 12. The excavation for concrete footings for the pump and discharge pipe pier was carried to rock at an elevation of about minus 20.5. The rock was cleared with picks to a level surface to minus 21.75. The foundations were erected of 1:2:4 concrete. Forms were made of 2-in. lumber and bolted together so that they could be used for similarly designed foundations elsewhere.

Drilling for the shaft blasting was started by hand May 20, and by machine May 31. The work of drilling, blasting and mucking was carried on from May 22 to July 8. Two hundred and ninety 1x8-in. sticks of 60 per cent dynamite, and 568 electrical exploders, were used on this part of the work. The blasting was stopped at a point 5 ft. from the interior surface of the well, leaving 2 ft. of rock and 3 ft. of masonry to be taken out. About 80 holes 12 ins. deep were drilled in the face preparatory to the work of removing the last 5 ft., which was to be done after the placing of the tunnel shield.

The tunnel shield was made of cast iron, approximately rectangular in shape, about 8 ft. 7 ins. high, 6 ft. 9 ins. wide and 3 ft. through. The casting was made in the foundries of the American Brake Shoe and Foundry Co. The hose gasket was a piece of condemned fire hose with the usual canvas lining, and was filled with a special preparation, called "chemical rubber," which is being used as a filler for automobile tires. The gasket was held in place with T-headed bolts, the heads of which were inserted in the lining, the shanks extending through to the outer surface of the groove, where they were held in place with nuts.

On Aug. 27 the shield was lowered into the well, with the 24-in. spiral pipe and the two lengths of 8-in. cast iron pipe attached, using the station crane. It was suspended from two 1¼-in. cast steel locomotive cables, which in turn were suspended from two 3-ton chain falls. These were suspended from 6x8-in. timbers supported on the gallery and upper edge of masonry of the well.

A No. 4 drive well jet pump, with 1-in. steam supply, was placed inside of the shield, with the bottom at elevation minus 32, the discharge passing up over the top and down the side of the well, and discharging at about elevation minus 13. The suction of a No. 3 Blakeslee bilge pump was also lowered into the shield through the 24-in. pipe.

The two pumps, with a combined capacity of 83 gals. per minute, did not lower the water inside the shield sufficiently, proving that a larger pumping capacity was necessary to force the shield and gasket tight against the masonry wall.

A No. 2 double-cylinder Emerson pump was added Sept. 3, placed in the wet well and connected to the steam line. The 3-in. suction of this pipe was let into the 24-in. pipe. The operation of this pump produced the desired effect of tightening the shield, emptying the same to the limit of the suction in about five minutes. The shield proved to be tight Sept. 8, and the blind flange was placed on the 24-in. pipe leading up from the shield. This was necessary, as the water level in the well was above normal, pumping engine No. 4 having been shut down. Consequently the deep well jet pump, which was operated intermittently, took care of all the leakage. The leakage was steadily reduced, requiring about 2 minutes of pumping every 15 minutes at first and 20 seconds of pumping every 20 minutes toward the completion of the work.

The plans for the installation of the centrifugal pump provided for a 36-in. hydraulically operated gate valve, to be placed in valve pit just below the floor level of the main engine pit. The shaft and tunnel were lined to an average thickness of 6 ins. with 1:2:4 concrete and the valve placed and connected, giving an additional safeguard in the event of a possible accident.

The first foot of rock of the two remaining was mined out by blasting, using 19 sticks of dynamite in 62 of the holes previously drilled. The rest of the rock and the brick work varying from 3 ft. to 4 ft. in thickness, was mined by drilling holes and breaking out material with bull points and sledges. All mining was completed Sept. 17.

The remaining portion of the tunnel was then lined with concrete of minimum thickness of 4 ins., the average thickness being 8 ins. The lining was completed Sept. 18. Forms were removed and water admitted into the tunnel Sept. 22. The emergency pump, tunnel shield and concrete chute were then removed from the station. The wet well cover and railing were replaced and the interior of the station cleaned up.

ACKNOWLEDGMENT.

The foregoing description of the work is taken from the latest annual report of Mr. Henry W. Clausen, engineer of water works construction. Mr. F. Carl Martini, assistant engineer, was in local charge of the work.

On Making Water Bills a Lien on Real Property.

It is the desire of all municipalities operating public utilities to operate them at a minimum of expense. Two of the elements of expense are the collection of bills and the loss of delinquencies. All users of water, gas and electric energy will not voluntarily go to the city hall to pay their monthly bills, and some of our utility officers are either slack, or indulgent, so that in all communities the cost of collecting and the delinquencies are greater than they should be.

If the rates for furnishing such utilities were a lien on the real property of the owner of the premises to which the utility was so furnished, superior to any other lien, or mortgage (except perhaps regular taxes) and subject to lease, or sale in practically the same manner as real property is now sold for delinquent taxes, the owners of the real property in the municipality would be responsible for the payment of the rates, and municipalities would thereby reduce the collection thereof to a minimum, and loss from delinquencies would be eliminated.

The question here discussed is: Can bills for services rendered by water departments be collected from the owners of the real property in the municipality by assessment and the owners forced to pay by a sale of their properties without a judgment being obtained in a court of law? The discussion is from a paper by Mr. Wallace Rutherford, city attorney of Napa, Calif., before the recent annual meeting of the League of California Municipalities. In dealing with this subject Mr. Rutherford has not gone into the question as to whether the powers of our municipalities, chartered, or otherwise, or the provisions of the constitution of the state, are broad enough to enable municipalities, or the legislature of the state to legislate upon the subject, but gives the result of his examination into the subject with the idea that all necessary amendments can be submitted in the future if his suggestions are adopted.

In considering this subject the author was fortunate in finding a number of charters in the state of New Jersey dealing therewith. The provisions of the charter of the city of Hoboken and Jersey City have been sustained by the Supreme Court of that state and also by the United States Supreme Court.

Among the enumerated powers found in the charter of the city of Hoboken are the following:

> To make all necessary arrangements for a full and copious supply of good and wholesome water for public and private use; to make ordinances for the distribution of the same, and to impose assessments for the use thereof, which assessments shall be binding upon improved and unimproved lots so assessed, and may be collected in the same manner as prescribed in this act for the collection of assessments for grading, paving, curbing, flagging, and laying out of public streets, etc. * * *
>
> That all taxes and assessments, which shall hereafter be assessed or made upon any lands, tenements, or real estate situate in said city, shall be and remain a lien until paid, notwithstanding any device, descent, alienation, mortgage, or other encumbrance thereof; and that if the full amount of any such tax or assessment shall not be paid and satisfied within the time limited for the payment thereof, it shall and may be lawful for the council to cause such lands, tenements, or real estate to be sold at public auction, for the shortest terms for which any person will agree to take the same, and pay such tax or assessment, or the balance thereof remaining unpaid, with the interest thereon, and all costs, charges, and expenses and to execute, under the common seal of the said city, a declaration of such sale, to be signed by the mayor and city clerk, and to deliver the same to the purchaser; and such purchaser, his executors, administrators, or assigns, shall, by virtue thereof, lawfully hold and enjoy the said lands, tenements, or real estate, for his and their own proper use, against the owner or owners thereof, and all persons claiming under him or them, until his said term shall be completed and ended, that the lands, tenements, or real estate, so sold, may be redeemed by the owner, mortgagee, occupant, or person interested therein, or by any other person, for and on behalf of the owner, mortgagee, or claimant of such lands, tenements, or real estate, at any time within two years after the sale, for either taxes or assessments, or for both, by paying, etc. * * * No mortgagee, whose mortgage shall have been duly recorded before sale for any tax or assessment, shall be affected by such sale, unless six months' notice in writing shall have been given to him by the purchaser, or those claiming under him, either personally or if not to be found in the city, then such notice shall be deposited in the post office of said city, directed to him at his last known place of residence (or at the post office nearest thereto), but nothing contained herein shall be so construed so as to impair the lien created by such tax, assessment, or sale.

The Supreme Court of New Jersey held, however, that the foregoing provisions were not broad enough to cover an assessment on real property for water furnished through a meter—where the charges were not regular, but depended upon the quanity used. It did not decide, however, that an assessment for such meter rates could not be levied, where the law was enacted to cover such circumstances.

Dillon in his work on the law of Municipal Corporations, says:

> The water rate or rent for water actually consumed on the premises may by statute be made a lien upon the property prior to all encumbrances in the same manner as taxes and assessments, and as such may take priority over a mortgage made after the passage of the statute creating the lien, whether the water was brought into the property and used before or after the making of the mortgage.

The foregoing quotation from Dillon is based upon the New Jersey decisions—particularly that of the U. S. Supreme Court in affirming the judgment of the New Jersey court.

The case of Provident Institution for Savings in Jersey City vs. Mayor and Aldermen of Jersey City, found in the 113 U. S. at page 506 contains the following that will be of interest:

The ground on which the decision below was based was, that the laws having made the water rents a charge on the land, with a lien prior to all other encumbrances, in the same manner as taxes and assessments, the complainant took its mortgages subject to this condition, whether the water was introduced on to the lot mortgaged before or after the giving of the mortgage; and hence the complainant had no ground of complaint that its property was taken without due process of law.

As the case comes before us, it is not necessary to enter into the discussions that have occupied the State Courts. We are to assume that the rents, penalties and interest claimed by the City have been imposed and incurred in conformity with the laws and Constitution of the State; and that, by virtue of said laws and Constitution, they are a lien on the property mortgaged to the complainant prior to that of its mortgages; and, this being so, we are only concerned to inquire whether those laws thus interpreted are, or are not, repugnant to the Constitution of the United States. The only clause of the Constitution supposed to be violated is that portion of the 14th Amendment which declares that no State shall deprive any person of life, liberty or property without due process of law.

Even if the water rents in question cannot be regarded as taxes, nor as special assessments for benefits arising from a public improvement, it is still by no means clear that the giving to them a priority of lien over all other encumbrances upon the property served with the water would be repugnant to the Constitution of the United States. The law which gives to the last maritime liens priority over earlier liens in point of time, is based on principles of acknowledged justice. That which is given for the preservation or betterment of the common pledge is in natural equity fairly entitled to the first rank in the tableau of claims. Mechanics' lien laws stand on the same basis of natural justice. We are not prepared to say that a legislative act giving preference to such liens over those already created by mortgage, judgment or attachment, would be repugnant to the Constitution of the United States. Nor are we prepared to say that an act giving preference to municipal water rents over such liens would be obnoxious to that charge. The providing of a sufficient water supply for the inhabitants of a great and growing city, is one of the highest functions of municipal government, and tends greatly to enhance the value of all real estate in its limits; and the charge for the use of the water may well be entitled to take high rank among outstanding claims against the property so benefited.

The plan of leasing the premises to the highest bidder for a given term, in order to collect the charges due, seemed to Mr. Rutherford to be a novel and practical method. The non-payment is brought home to the owner in a forcible manner, and he is not deprived of the title to his property. Giving him an opportunity to redeem by paying the charges plus interest and penalties would amply protect his interests.

The conclusion drawn by Mr. Rutherford from his examination of the authorities found is that a lien can be created on real property for the collection of charges from public utilities, such lien to be created by the legislature for municipalities created under the general laws and by ordinance where the charters are sufficiently comprehensive.

BRIDGES

Design, Construction and Detailed Labor Costs of the Substructure of the Double-Leaf Trunnion Bascule Bridge at Chicago Ave., Chicago, Ill.

Contributed by Carl O. Johnson, Resident Engineer in Charge of Construction.

[In our Oct. 21, 1914 issue we described and illustrated the design features of the substructure of the double-leaf trunnion bascule bridge which spans the Chicago River at Chicago Ave., Chicago. In this issue we shall give the detailed labor costs of the various parts of the substructure work, together with data on the equipment, methods used, and conditions under which the work was done. The cost data have been carefully analyzed, and we believe they are the most complete ever published for work of this character. The labor costs given are actual costs; no charge has been made for tug service, plant rental or depreciation, interest, or central office overhead charges. The general cost data are given first, these data being followed by a discussion, for each division of the work, of the construction methods used, the unit costs, and the conditions influencing the costs.—Editors.]

II. Cost Data and Construction Methods.

ESTIMATED QUANTITIES, UNIT BIDDING PRICES AND ACTUAL QUANTITIES PLACED.

Table I (p. 427) gives the estimated quantities of materials, the unit bidding prices, the actual quantities of materials placed, and the estimated and total costs of each item of the substructure work.

In addition to the successful contractor's bid of $105,346.20, three other bids were received, the total amounts of these bids being $106,137.50, $107,100.00 and $116,950.00.

CONTRACT AND CONTRACTOR'S EQUIPMENT.

The contract between the city of Chicago and Byrne Bros. Dredging and Engineering Co. was signed Dec. 2, 1912, and notification to begin work was given by the city Dec. 13, 1912. The time limit for this work was nine months. Construction work was actually begun March 17, 1913, and the work was finished March 23, 1914.

The contractor was required to furnish all labor, material and plant necessary for the construction work, and was made responsible for all damages due to the construction of the substructure. The construction plant consisted of the following equipment:

One 6-cu. yd. "Marion" dipper dredge.
Two dump scows, [illegible] cu. yds. capacity each.
Two dump scows, 250 cu. yds. capacity each.
One derrick scow equipped with a 12-in. sand pump.
Two deck scows.
One floating pile driver.
One shore pile driver.
One stiff-leg derrick with 40-ft. wood mast and 80 ft. boom.
One stiff-leg derrick with 40-ft. steel mast and 90-ft. boom.
One 22-HP. hoisting engine.
One 30-HP. hoisting engine.

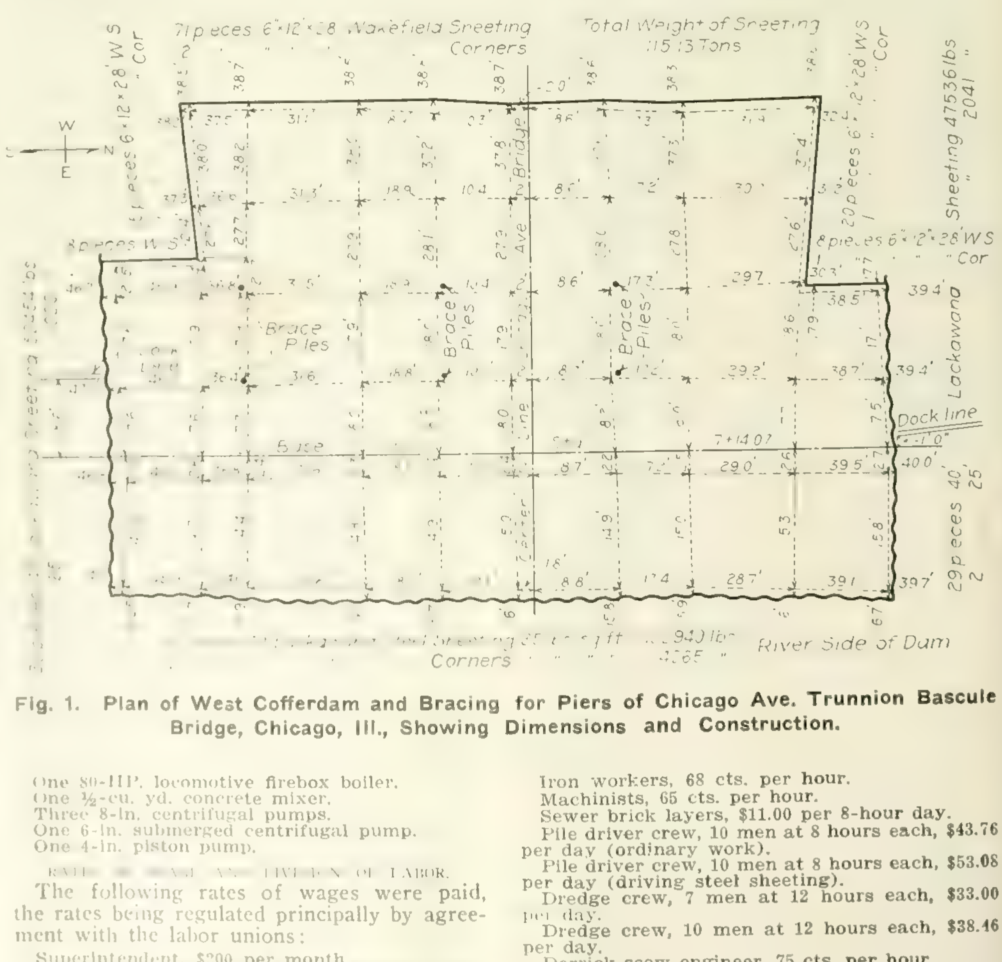


Fig. 1. Plan of West Cofferdam and Bracing for Piers of Chicago Ave. Trunnion Bascule Bridge, Chicago, Ill., Showing Dimensions and Construction.

One 80-HP. locomotive firebox boiler.
One ½-cu. yd. concrete mixer.
Three 8-in. centrifugal pumps.
One 6-in. submerged centrifugal pump.
One 4-in. piston pump.

RATES OF WAGES AND DIVISION OF LABOR.

The following rates of wages were paid, the rates being regulated principally by agreement with the labor unions:

Superintendent, $200 per month.
T[illegible] $1.50 to $[illegible] per day; average rate, 35.9 cts. per hour.
W[illegible] per day.
Hoisting engineers, 75 to 80 cts. per hour; average rate, 76.2 cts. per hour.
Firemen, 46 cts. per hour.
Winchmen, 52½ cts. per hour.
Signalmen, from 40 to 50 cts. per hour; average rate, 45½ cts. per hour.
Carpenter foremen, 75 cts. per hour.
Carpenters, 65 cts. per hour.
Carpenters' helpers, 48 cts. per hour.
Labor foremen, from $4.50 to $7.00 per day; average rate, 58.2 cts. per hour.
Laborers, from 25 to 60 cts. per hour; average rate, 44.2 cts. per hour.
Iron worker foreman, $1.14 per hour.
Iron worker straw boss, 93¾ cts. per hour.
Iron workers, 68 cts. per hour.
Machinists, 65 cts. per hour.
Sewer brick layers, $11.00 per 8-hour day.
Pile driver crew, 10 men at 8 hours each, $43.76 per day (ordinary work).
Pile driver crew, 10 men at 8 hours each, $53.08 per day (driving steel sheeting).
Dredge crew, 7 men at 12 hours each, $33.00 per day.
Dredge crew, 10 men at 12 hours each, $38.46 per day.
Derrick scow engineer, 75 cts. per hour.
Derrick scow fireman, 30 cts. per hour.

A day, or shift, was 8 hours. The superintendent, timekeepers, and labor foremen worked 8 to 12-hour shifts. The average rate of wage for all classes of labor for the entire job was 53 cts. per hour.

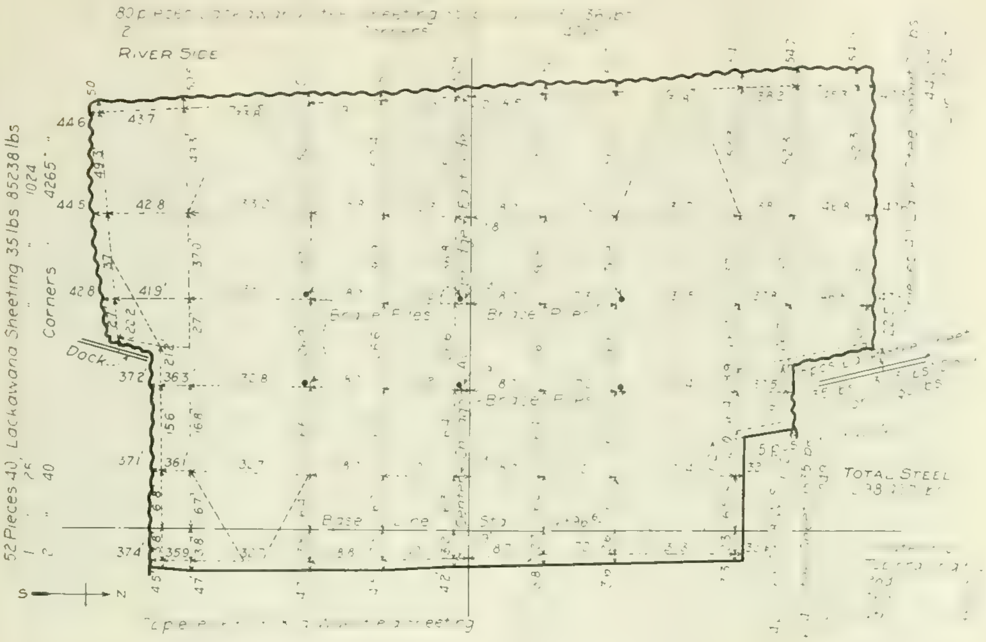


Fig. 2. Plan of East Cofferdam and Bracing for Piers of Chicago Ave. Bridge, Showing Dimensions and Construction.

Table II gives the division of labor on the work, classified both as to time and cost.

TIME AND COST DATA FOR VARIOUS LABOR ITEMS.

In all the cost data given in this article, only *actual job labor costs* have been considered. No charge has been made for tug service, plant rental or depreciation, interest, or central office overhead charges, etc.

TABLE I.—ESTIMATED QUANTITIES, UNIT BIDDING PRICES AND ACTUAL QUANTITIES PLACED.

Item of contract.	Ref. letter.	Estimated quantities.	Unit price.	Total bid.	Actual quantities placed.	Actual amount of contract.
Remove obstructions	A	......	.....	$ 650.00	......	$ 650.00
Build, protect, maintain and remove two cofferdams	B	......	.....	22,300.00	......	22,300.00
Miscellaneous work	C	......	.....	910.00	.....	910.00
Excavation	D	8,500 cu. yds.	$ 1.29	10,965.00	6,774 cu. yds.	8,738.46
Oak timber, in place	E	18,000 ft. B. M.	62.40	1,123.20	5,564 ft. B. M.	347.19
Pine timber, in place	F	6,000 ft. B. M.	39.00	234.00	5,128 ft. B. M.	199.39
Test piles, delivered and driven	G	4 piles.	30.00	120.00	4 piles.	120.00
Oak piles, delivered	H	6,500 lin. ft.	0.18	1,170.00	7,182 lin. ft.	1,292.76
Norway pine or cypress piles, delivered	I	11,000 lin. ft.	0.16	1,760.00	5,100 lin. ft.	816.00
Driving piles for foundation	J	15,000 lin. ft.	0.115	1,725.00	9,902 lin. ft.	1,138.73
Portland cement concrete	K	3,900 cu. yds.	7.25	28,275.00	3,604 cu. yds.	26,129.00
Portland cement mortar	L	350 cu. yds.	11.00	3,850.00	357 cu. yds.	3,927.00
Concrete shaft foundation, from El. —20 to El. —45	M	20,000 cu. ft.	0.39½	7,850.00	18,648 cu. ft.	7,319.34
Removal of boulders from shafts	N	100 cu. ft.	0.12	12.00	......	....
Steel sheeting, furnished and driven in shafts	O	275,000 lbs.	0.02½	6,875.00	279,470 lbs.	6,986.75
Steel reinforcing bars, furnished and placed	P	150,000 lbs.	0.02	3,000.00	145,710 lbs.	2,914.20
Setting substructure steel, furnished by superstructure contractor	Q	210,000 lbs.	0.03	6,300.00	204,550 lbs.	6,136.50
Structural steel, furnished and set	R	16,000 lbs.	0.02½	400.00	9,136 lbs.	228.40
Divert and extend 5-ft. brick sewer	S	25 lin. ft.	9.60	240.00	69.6 lin. ft.	668.16
Divert and extend 4-ft. brick sewer	T	55 lin. ft.	8.40	462.00	......	.....
Concrete cylinder foundation, from El. —45 to rock, about El. —81	U	15,000 cu. ft.	0.47½	7,125.00	14,601 cu. ft.	6,935.48
Total	..	......	....	$105,346.20	......	$97,757.96

TABLE II.—DIVISION OF LABOR ON JOB.

Kind of labor.	Percentage of total hours worked.	Percentage of total cost of work.
Superintendent, timekeeper, watchman, etc.	8.5	8.2
Engineers and firemen	12.4	15.7
Carpenters	10.6	13.2
Laborers	50.5	43.4
Iron workers	0.8	1.2
Machinists	0.1	0.1
Bricklayers	0.1	0.2
Pile driver crew	14.4	16.2
Dredge crew	2.4	1.7
Derrick scow crew	0.2	0.1
Total labor	100.0	100.0

Tables III to IX, inclusive, contain accounts which were prorated among the contract items as shown. These tabular data give essential information on the cost and the time required to complete various parts of the work.

The average rate for the work indicated in Table III was 54 cts. per hour. The items given in Table III were prorated among all items where the pile driver crew was used according to the number of hours worked, as follows:

Item.	Total hours.	Total cost.
Cofferdams	875	$ 474.90
Steel sheeting for concrete caissons	454	245.00
Miscellaneous	41	22.30
Pine timber	41	22.30
Oak timber	11	4.66
Test piles	11	4.55
Pile driving for foundation	289	156.00
Removal	247	137.00
To Table VIII	82	44.60
To Table IX	11	4.55
Total	2,062	$1,114.86

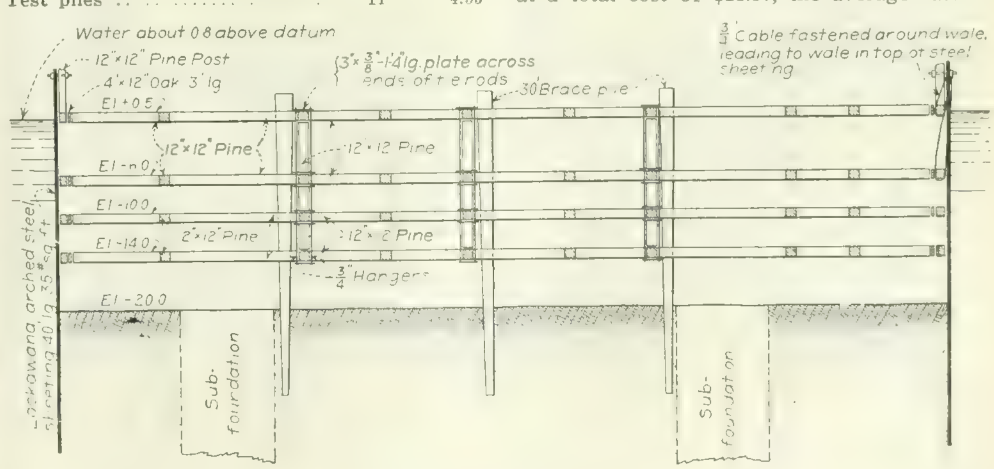


Fig. 3. Typical Cross Section of East Cofferdam for Chicago Ave. Bridge, Showing Bracing and Central Brace Piles.

TABLE III.—PILE DRIVING EQUIPMENT.

Item.	Total hours.	Total cost.
Rigging floating pile driver No. 1	60	$ 32.86
Unloading and erecting shore pile driver No. 2	193	109.37
Rigging and erecting shore pile driver No. 3	730	360.89
Removing wrecked pile driver	164	90.98
Rebuilding shore pile driver	535.5	313.15
Removing steam hammer for another job	11.5	7.51
Wrecking old pile driver	112	56.86
Repairing pile driver scow	216	121.36
Storing pile driver	40	21.88
Total	2,062	$1,114.86

Unloading the steel cofferdam sheeting, the steel caisson sheeting and the reinforcing bars for the concrete work, hauling this material from the cars to the scow, towing the same about 1,500 ft., and unloading the material on the docks required 344 hours, at a cost of $171.50, the average rate of wage for this work being 50 cts. per hour. These items were prorated according to the amount of steel used for the various parts of the work as given in Table IV.

TABLE IV.—HANDLING STEEL.

Item.	Total hours.	Total cost.
Cofferdams	189	$94.41
Steel caisson sheeting	100	49.70
Reinforcing bars	55	27.40
Total	344	$171.51

Unloading coal for the plant on the west side of the Chicago River required 64½ hours, at a total cost of $28.57, the average rate of wage being 45 cts. per hour. This item was prorated as given in Table V.

TABLE V.—HANDLING COAL.

Item.	Total hours.	Total cost.
Pumping water from cofferdam..	58	$22.71
Excavation ...	6½	5.86
Total ...	64½	$28.57

The cost and the time required to sort the old lumber used in the cofferdams are given in Table VI. The average rate of wage for this work was 47 cts. per hour, and the work was prorated 50 per cent to each cofferdam.

TABLE VI.—SORTING LUMBER.

Item.	Total hours.	Total cost.
Sorting lumber	281½	$131.42

The time and cost data on the labor which may be classified as superintendence is given in Table VII. This item includes the work of the superintendent, timekeepers, watchman and the unclassified time of the carpenter and labor foremen. It amounts to 12.1 per cent of the net pay roll, the average rate of wage being 53 cts. per hour.

TABLE VII.—SUPERINTENDENCE.

Item.	Total hours	Total cost.
Superintendence	9,612	$5,010.01

Table VIII gives the cost and the number of hours worked on items pertaining to the work of the derricks.

TABLE VIII.—WORK PERTAINING TO DERRICKS.

Item.	Total hours.	Total cost.
Clearing space for west plant..	47	$ 18.80
Driving five 28-ft. piles for derrick foundation	85	46.50
Framing west derrick, 40-ft. mast, 80-ft. boom	40	26.00
Rigging and erecting west derrick	195½	106.91
Housing boilers	371½	189.84
Wrecking west derrick and plant	114	57.82
Cleaning up site of plant......	30	12.00
Prorated amount from Table III	55	29.74
Total (average rate of wage, 52 cts. per hour)	938	$ 487.61
Driving three 45-ft. and two 30-ft. piles for foundation of east derrick	40	$ 21.88
Cutting and framing bent to above piles	12	7.80
Building crib for foundation of derrick sill	36	14.10
Rigging and erecting east derrick, 40-ft. mast, 90-ft. boom	291	153.91
General work on east plant	[illegible]	628.49
Prorated amount from Table III	27	[illegible]
Total (average rate of wage, [illegible] cts. per hour)	[illegible]	$ [illegible]
Unloading east and west plants	74	$ 33.35
General work on east and west plants	[illegible]	92.92
Total (average rate of wage, 55 cts. per hour)...........	226	$ [illegible]
Grand total	[illegible]	$[illegible]

As the items given in Table VIII were charged principally to the derrick plant, they were prorated among the contract items according to the "derrick engineer" hours charged against these items, as follows:

Item.	Total hours.	Total cost.
Cofferdams	320	$ 174.57
Miscellaneous	[illegible]	87.31
Excavation	480	260.32
Concrete	[illegible]	320.04
Mortar	134	73.78
Caissons above El. [illegible]	[illegible]	174.57
Caissons below El. [illegible]	[illegible]	291.02
Setting substructure steel.	133	73.78
Total	[illegible]	[illegible]

The cost and the time charged to the concrete plant are given in Table IX.

TABLE IX.—CONCRETE PLANT.

Item.	Total hours.	Total cost.
Loading and unloading sand scow	357½	$ 151.00
Placing protection over east pit floor	8	3.60
Cleaning rubbish and concrete from east pit floor...........	144	73.40
Clearing space for east concrete mixing plant...........	20	8.15
Erecting east concrete mixer and 85½-ft. tower for same, using 4,454 ft. B. M. lumber.	965	549.89
Handling concrete chutes.....	270	139.53
Removing east concrete mixing plant.	100	50.00
Removing east concrete tower	62	32.55
Prorated amount from Table III	11	4.55
Unloading stone scows	332	144.98
Clearing storage space for sand and stone on west side of river	37	14.80
Setting west concrete mixer and running water pipe line.	72½	43.24
Resetting concrete mixer in second position	138	70.08
Resetting concrete mixer in third position	141	74.22
Changing concrete chute to west side	93	45.20
Removing west concrete mixing plant	25	10.00
Building and removing platform for stone storage on west side	332	144.98
Unloading stone from canal boats on west side...........	373	191.50
Clearing rubbish and concrete from west pit, etc.	438½	207.33
General repairs to concrete plant	270½	122.72
Miscellaneous, moving of cement	153	69.19
Total (average rate of wage, 50 cts. per hour)	4,182	$2,117.03

Fig. 4. View from South of West Cofferdam for Chicago Ave. Bridge—Note Two Types of Sheeting and Brace Piles.

The items given in Table IX were charged against "concrete" items, as follows:

Item.	Total hours.	Total cost.
Concrete	3,027.35	$1,532.44
Mortar	243.33	123.10
Concrete caisson foundations between El. —20 and El.—45	501.97	254.25
Concrete caisson foundations below El. —45...............	409.35	207.24

Discussion of Construction and Cost Data and Unit Costs.

A description of the work done under each subdivision of the contract, together with a discussion of the cost data will now be given. Each item is referred to by the reference letter given in the specifications and shown in Table I.

[illegible] REMOVAL OF OBSTRUCTIONS.

This item included all work done in removing obstructions which interferred with the construction of the substructure. The work consisted principally of removing a rubble masonry pier containing 103 cu. yds., timber and pile approaches, parts of brick sewers, and concrete walks. The lump sum bid for this work was $650. The construction plant used consisted of a pile driver, a derrick scow and a deck scow. The actual labor costs for this work were divided as follows:

Item.	Total hours.	Total cost.
Removal of timbers, piles, etc..	1,733	$ 831.38
Removal of stone, brick, concrete	748	282.05
Prorated charge	247	137.00
Total	2,728	$1,250.43
Superintendence, etc., 12.1 per cent	330.1	151.30
Grand total	3,058.1	$1,401.73

Average labor cost, 46 cts. per hour.

"G"—TEST PILES.

The price bid for furnishing and driving four 60-ft. test piles was $120. These piles were located so as to be used later as protection and foundation piles. The plant used was one pile driver. The actual labor cost for this work was divided as follows:

Item.	Total hours	Total cost.
For driving four 60-ft. test piles (cost per foot, 11½ cts.; rate per 8-hour day, 6.4 piles)...........	50	$27.35
Prorated charge........	11	4.55
Total	61	$31.90
Superintendence, etc., 12.1 per cent	7.38	3.86
Grand total	68.38	$35.76

Average labor cost, 52 cts. per hour; rate per 8-hour day, 4.7 piles.

"H" AND "I"—FURNISHING OAK AND NORWAY PINE PILES.

The piles under the abutments were only about 25 ft. long, this short length being used so as not to interfere with the future subway construction at this location. The remaining foundation and protection piles were about 45 ft. long. The cost of these piles delivered was as follows:

Oak piles, 7,182 lin. ft. at 18 cts.........	$1,292.76
Norway pine piles, 5,100 lin. ft. at 16 cts.	816.00
Total	$2,108.76

"J"—DRIVING PILES.

The bidding price below cut-off for driving piles in foundations, piers, pier protections, abutments, outside walls and dock lines was 11½ cts. per linear foot. There were 320 piles, 9,902 lin. ft., actually used, the bidding price for this item being $1,138.73.

The actual labor cost of driving these piles was divided as follows:

Item.	Total hours.	Total cost.
Driving piles (320 piles, 9,902 lin. ft., below cut-off)......	1,466	$ 799.90
Prorated charge	266	144.00
Total	1,732	$ 943.90
Superintendence, etc., 12.1 per cent	209.45	114.21
Grand total	1,940.45	$1,058.11

Cost (exclusive of prorated charge and superintendence), 8 cts. per linear foot below cut-off; rate, 17.4 piles per 8-hour day.

Cost (including prorated charge and superintendence), 11 cts. per linear foot below cut-off; rate, 13 piles per 8-hour day.

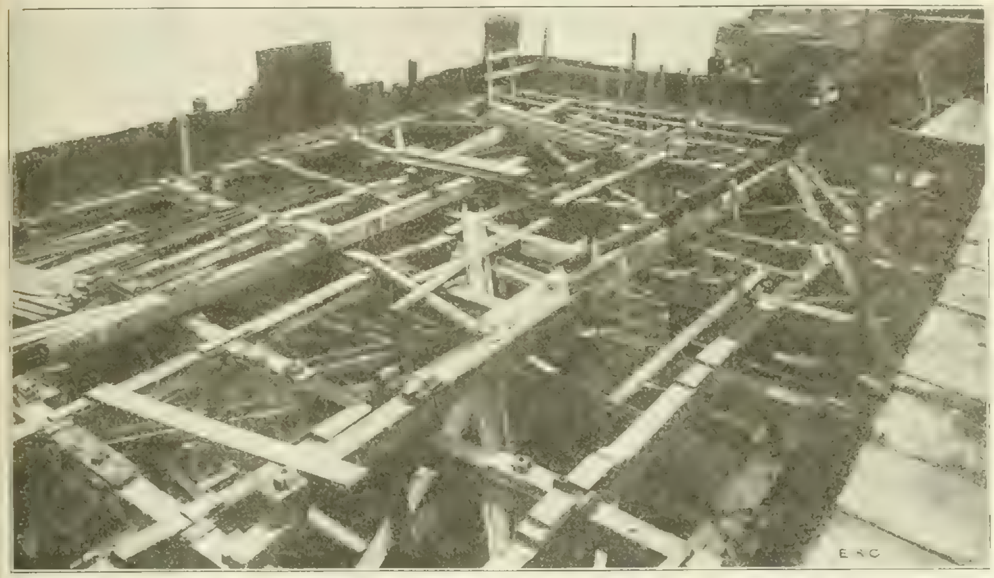


Fig. 5. View from Northwest of West Cofferdam for Chicago Ave. Bridge, Showing Bracing and Concrete Chutes—Substructure Trusses Shown in Place.

"C"—MISCELLANEOUS WORK.

The miscellaneous work included the following items: Making preliminary borings; furnishing scows and other supports for steel tapes during the taking of measurements across the river; making temporary sewer diversions; building and removing shed for storage of cement; protecting, supporting, maintaining, and restoring adjacent buildings affected by the construction of the cofferdams and substructure (no work required as buildings were not damaged); providing office space and temporary telephone service during construction. The lump sum bid for this work was $910. The borings were made by the city. One of the offices given below was 12x20x10 ft. high and the other, 10x20x9 ft. high.

The actual labor cost for this work was divided as follows:

Item.	Total hours.	Total cost.
Twice diverting 5 ft. east sewer into 140-ft. wooden flume	1,522.8	$ 759.[illegible]
Diverting 4-ft. west sewer into 146-ft. wooden flume...	427.1	186.59
Constructing two offices, 3,100 ft. B. M. lumber used.......	170.39	102.91
Miscellaneous work	973.59	439.[illegible]
Total (average wage rate, 48 cts. per hour)............	3,093.88	$1,488.21

"E"—COFFERDAMS

The cofferdams, which were of the single-wall type, were built of steel and wooden sheeting, and entirely enclosed the main piers and small walls. The lump sum for building, maintaining, protecting and removing the two cofferdams was $22,300.

The west cofferdam had maximum dimensions of 86.7 ft. by 55.3 ft., enclosing an area of 4,424 sq. ft. Figure 1 shows a plan of the west cofferdam and gives essential data concerning its construction. The maximum depth of water outside of this dam was 19.9 ft.

The east cofferdam had maximum dimensions of 91.9 ft. by 56.3 ft., inclosing an area of 4,655 sq. ft. The maximum depth of water outside of this dam was 20.9 ft. Figure 2 shows a plan of the east cofferdam and gives data on its construction features, and Fig. 3 shows a typical north-and-south cross-section of this cofferdam and indicates the method used to brace the cofferdam.

The excavation (all soft clay) was carried down to a general elevation of —20.0 (the river being at elevation about +0.8), from which depth four caissons were sunk to bed rock, which lies at an average elevation of —81.1. The sites of the cofferdams were first cleared with dipper dredges, the west cofferdam being dredged from an original *average* depth of 2.3 ft. to an *average* depth of 10.7 ft., and the east cofferdam, from an *average* depth of 4.3 ft. to an *average* depth of 11.7 ft. The excavated material was dumped into scows and towed to dumping grounds in Lake Michigan. The only dredging paid for consisted of that enclosed by the cofferdams, although considerable dredging was done outside of the cofferdam walls. After the site was cleared the foundation piles and the cofferdam sheeting were driven. "Lackawanna" arched web sheeting, weighing 35 lbs. per square foot and having a length of 40 ft., was used in the river and also up to a point about 10 ft. inland where it connected with 6x12-in.x28-ft. "Wakefield" sheet piling. At the east side of the river the steel sheeting was extended along a nine-story reinforced concrete building and a one-story freight house where their foundations appeared to be in danger. Figure 4 shows a view of the west cofferdam looking from the south side. The view also shows the brace piles near the center of the cofferdam.

Six brace piles were also driven in each cofferdam to support temporarily the system of bracing. The waling timbers were of 12x12-in. pine and were suspended by cables which passed through holes in the top of the steel sheeting. A 12x12-in. post was set on top of the upper tier of waling and bolted to the steel sheeting, the cables and posts preventing any movement of the waling due to the changing elevation of the water level within the cofferdam. (See Fig. 5.) The remaining timbers were either 12x12-in. pine or waste pieces of piles. Where timbers butted against waling pieces a 4x12-in.x3-ft. oak block was used. Corner braces were used in the cofferdam pockets over the caissons. Four sets of bracing were placed in each dam—at elevations +0.5, —6.0, —10.0 and —14.0 (see Figs. 3 and 5), the bottom of the main excavation being at elevation —20.0. The tiers of bracing were separated by 12x12-in. posts and were tied together with double ¾-in. tie rods. They were also bolted to the brace piles where convenient.

One 12-in. and three 8-in. centrifugal pumps were required to pump the first 6 ft. of water from the west cofferdam, this pumping requiring about two hours. From this level a 6-in. or an 8-in. pump, operated from time to time, was sufficient to remove the water from the cofferdam. When the pumping began, fine ashes were distributed along the outside of the steel sheeting, which proved very effective in stopping leaks. No other means were employed to make the sheeting watertight. Figure 6 shows a view of the east cofferdam, the top tier of bracing, and the centrifugal pumps.

At the completion of the work all the material composing the cofferdams and bracing was recovered except the "Wakefield" sheeting and 43 pieces of the "Lackawanna" steel sheeting.

The high cost of pumping given in Table X was due to the fact that it was necessary to pump part of the time both day and night, which necessitated the employment of hoisting engineers as pumping engineers for 24 hours each day.

Tables X and XI give cost data, for the west and east cofferdams, on the driving of wood and steel sheet piling, the pumping of water from the cofferdams, the bracing of the cofferdams, and removing them.

Table XII gives a summary of the labor costs of constructing and removing the two cofferdams.

Fig. 6. View from Northeast of East Cofferdam for Chicago Ave. Bridge, Showing Top Tier of Bracing and Centrifugal Pumps in Place—Raised Section of Sheeting Is Over Water Pipe Tunnel.

TABLE X.—FORCE ACCOUNT AND LABOR COST DATA FOR WEST COFFERDAM

DRIVING WOOD PILES AND WOOD SHEETING

Item	Total hours	Rate, cts. per hr.	Total cost	Remarks.
Sorting [illegible]	140.8		[illegible]	
Driving [illegible] piles at [illegible]	[illegible]		[illegible]	Rate, [illegible] per [illegible] day.
Driving [illegible]	[illegible]		[illegible]	Rate [illegible] day.
Driving [illegible] piles, and 2 protection [illegible]	[illegible]		[illegible]	Rate, 26 0 piles per 8-hr day.
Placing [illegible] ft. Wakefield sheeting [illegible]	[illegible]		[illegible]	
[illegible] ft. Wakefield sheeting [illegible]	[illegible]		244.35	Rate 22½ sheets per 8-hr. day.
Prorated charge	[illegible]		73.00	
Total	1,080.8	[illegible]	[illegible]	
Superintendence, etc., 12.1 per cent	130.7		69.99	
Grand total	1,211 [illegible]	[illegible]	[illegible]	
DRIVING STEEL SHEETING.				
Preliminary handling of [illegible]	[illegible]		$ 27.67	
Loading steel sheeting from [illegible] to scow	[illegible]		53.08	
Driving steel sheeting, [illegible] =115.13 tons	617		404.72	Rate, 18 [illegible] per 8-hr. day.
Prorated charge	[illegible]		71.00	
Total	[illegible]	[illegible]	[illegible]	Cost, 7.2 hrs. per ton = $4.83, or $3.94 per sheet; av. sheets per day=13.
Superintendence, etc., 12.1 per cent	107		67.34	
Grand total	991	63	[illegible]	Cost, 8.6 hrs. per ton= $5.40, or $4.44 each; av. sheets per 8-hr. day 11½.

PUMPING WATER FROM COFFERDAM

Item	Total hours	Rate, cts. per hr.	Total cost	Remarks.
Installing pumps and pumping water	[illegible]		$3,222.30	
Handling coal for pumps	[illegible]		25.71	
Placing ashes to water-proof sheeting	1,0[illegible]		447.41	
Plugging extraordinary leaks	[illegible]		111.33	
Tending boiler plant for [illegible]	[illegible]		284.58	
Total	[illegible]	[illegible]	$4,091.33	
Superintendence, etc., 12.1 per cent	874.8		495.05	
Grand total	[illegible]	[illegible]	[illegible]	

BRACING COFFERDAM

Item	Total hours	Rate, cts. per hr.	Total cost	Remarks.
Unloading timber from [illegible]	[illegible]		$ 32.33	
Placing first set of bracing	[illegible]		149.24	
Placing second set of bracing	[illegible]		216.08	
Placing third set of bracing	[illegible]		195.81	
Placing fourth set of bracing	[illegible]		195.28	
Prorated charge	[illegible]		7.60	
Total timber used, 58,526 ft. B. M., of which 44,776 ft. B. M. was 12x12-in. waling and bracing [illegible]	[illegible]	[illegible]	$826.34	58,526 ft. B. M. at 23.2 hrs. per M = $14.20, or 44,776 ft. B. M. at 28½ hrs. per M = $17.40. 100 sq. ft. area in 31 hrs. = $18.70.
Superintendence, etc., 12.1 per cent	161½		[illegible]	
Grand total	[illegible]		[illegible]	[illegible] per M = $15.90, or 44,776 ft. B. M. at 32 hrs. per M = $19.40. 100 sq. ft. area in 35 hrs. = $21.00.

REMOVING COFFERDAM

Item	Total hours	Rate, cts. per hr.	Total cost	Remarks.
Removing [illegible]	[illegible]		$108.15	
Removing pit timbers	301		[illegible]	
Pulling 141 pieces steel sheeting 115.13 tons	[illegible]		521.48	Pulled, 11.1 pieces per 8-hr. day; $3.70 each, or $4.53 per ton.
Prorated charge	116		79.30	
Total	1,456		[illegible]	
Superintendence, etc., 12.1 per cent	176.2		[illegible]	
Grand total	[illegible]	[illegible]	[illegible]	

TABLE X.—FORCE ACCOUNT AND LABOR COST DATA FOR EAST COFFERDAM

DRIVING WOOD PILES AND WOOD SHEETING.

Item.	Total hours.	Rate, cts. per hr.	Total cost	Remarks
[illegible] Wakefield [illegible]	12		$ 4.80	
Driving 101 pieces Wakefield sheeting, 6x12-in. [illegible]	[illegible]		204.80	Rate, 21½ pieces per 8-hr. day.
Driving 10 20-ft. piles at [illegible]	[illegible]		32.82	Rate, 13 piles per 8-hr. day.
Driving and chaining 2 [illegible] piles and 4 protection poles	130	..	71.11	Rate, 17 piles per 8-hr. day.
Driving 9 brace piles....	30	..	16.41	Rate, 21 piles per 8-hr day.
Sorting old piles and timber	110.75	..	65.71	
Prorated charge	108	..	59.00	
Total	855.75	53	$454.65	
Superintendence, etc., 12.1 per cent..........	103.55	..	55.01	
Grand total	959.3	53	$509.66	
DRIVING STEEL SHEETING.				
Loading steel sheeting from dock to scow....	258	..	$171.08	
Driving 1[illegible] pieces of steel sheeting 40-ft. long and 3 pieces 25-ft. long=149.73 tons...	695		461.13	Rate, 21 pieces per 8-hr. day.
Prorated charge	172	..	93.50	
Total	1,125	63	$725.70	Rate 13 pieces per 8-hr. day=$4.00 each. 7½ hrs. per ton=$4.86 per ton.
Superintendence, etc., 12.1 per cent..........	136 1		87.81	
Grand total	1,261.1	63	$813.51	Rate, 11½ pieces per 8-hr. day=$4.48 each. 8½ hrs. per ton=$5.40 per ton.

PUMPING WATER FROM COFFERDAM.

Item.	Total hours.	Rate, cts. per hr.	Total cost	Remarks
Installing pumps and pumping water	4,366	..	$2,623.56	
Handling coal for pumps	34	..	18.15	
Placing ashes for water-proofing sheeting	467		191.47	
Plugging extraordinary leaks in dam	48	..	27.86	
Puddling parts of cofferdam	182.5	..	83.13	
Prorated charge	52	..	28.00	
Total	5,149.5	57	$2,972.17	
Superintendence, etc., 12.1 per cent..........	623.1	..	306.03	
Grand total	5,772.6	57	$3,278.20	

BRACING COFFERDAM.

Item.	Total hours.	Rate, cts. per hr.	Total cost	Remarks
Unloading timber from scow	105	..	$ 55.57	
Waling and mud sills for back wall	45		24.16	
Placing first set of bracing	322		209.30	
Placing second set of bracing	467.5		290.23	
Placing third set of bracing	270		170.65	
Placing fourth set of bracing	339.5		213.26	
Prorated charge	19		10.50	
Total (timber used, 62,845 ft. B. M., of which 50,646 ft. B. M. was 12x12-in. waling and bracing. Area of cofferdam = 4,655 sq. ft)...	1,568		$ 973.67	62,845 ft. B. M. at 24½ hrs. per M. = $15.50. 50,646 ft. B. M. at 31 hrs. per M. = $19.60. 100 sq. ft. area in 33.2 hrs. = $21.00.
Superintendence, etc., 12.1 cent	189.7		117.81	
Grand total	1,757.7	62	$1,091.48	62,845 ft. B. M. at 28 hrs. per M. = $17.70. 50,646 ft. B. M. at 34.7 hrs. per M. = $21.50. 100 sq. ft. area in 37.7 hrs. = $23.40.

REMOVING COFFERDAM.

Item.	Total hours.	Rate, cts. per hr.	Total cost	Remarks
Removing bracing of cofferdam	401	..	$ 233.66	
Removing timbers from [illegible]	139		67.85	
Sawing off back line of Wakefield sheeting	38		15.20	
Burning off 12 pieces [illegible] sheeting at [illegible]	32	..	16.66	Burn about 7 per hr. The 32 hrs. includes helpers' time.
Pulling 138 pieces steel sheeting = 113½ tons..	558		357.23	Rate 20 pieces per 8-hr. day $2.59 each $3.14 per ton.
Prorated charge	96		52 10	
Total	1,264	59	$ 742.70	
Superintendence, etc., 12.1 per cent..........	152.9		89.87	
Grand total	1,416.9	59	$ 832.57	

"D"—EXCAVATION.

The price bid for excavating the site of the piers, tail pits and abutments, including the necessary back-fill, was $1.29 per cubic yard.

The excavation naturally divides itself into three classes: that removed by dredging; the excavation outside of the cofferdams; and the excavation within the cofferdams. Before excavation was commenced soundings were taken so that the river bottom at the bridge site could be accurately plotted. The sites of the cofferdams were then dredged and the dams constructed. Later, another set of soundings was taken, and the actual quantity of material removed from the cofferdams by dredging was computed to be 2,651.5 cu. yds., which was about 39 per cent of the total excavation. The material within the cofferdams was soft, blue clay and this was excavated by hand, the material being handled in buckets by derricks into dump scows and towed to the dumping grounds in Lake Michigan. The excavation outside of the cofferdams, principally at the site of the abutments, was also clay. The labor of cutting off the foundation piles was included in that classified under excavation. The total and unit labor costs of excavation are given in Table XIII.

TABLE XII.—SUMMARY OF LABOR COSTS OF EAST AND WEST COFFERDAMS.

Item.	West Dam. Total hours.	West Dam. Rate cts. per hr.	West Dam. Total cost.	West Dam. Per cent of total.	East Dam. Total hours.	East Dam. Rate cts. per hr.	East Dam. Total cost.	East Dam. Per cent of total.	Both dams. Total hours.	Both dams. Rate cts. per hr.	Both dams. Total cost.	Both dams. Per cent of total.
Driving piles and sheeting for cofferdam	2,202.5	..	$1,272.25	16.4	2,220.4	..	$1,323.17	20.3	4,422.9	..	$2,595.42	18
Pumping water from cofferdam	8,104.3	.	4,586.38	59.2	5,772.6	..	3,278.20	50.2	13,876.9	..	7,864.58	55.4
Bracing cofferdam	1,525	..	926.33	11.95	1,757.7	..	1,091.48	16.7	3,282.7	..	2,017.81	14
Removing cofferdam	1,632.2	.	959.32	12.45	1,416.9	..	832.57	12.8	3,049.1	.	1,791.89	12.6
Total	13,464.0	58	$7,744.28	100	11,167.6	59	$6,525.42	100	24,631.6	58	$14,269.70	100

"O"—STEEL SHEETING FOR CONCRETE SHAFTS.

The price bid for the steel sheeting used for the shaft foundations, including furnishing of same, driving and leaving in place was 2.5 cts. per pound. It was specified that the sheeting must weigh at least 35 lbs. per square foot. The length of the steel sheeting was 25 ft., its top being at elevation —20.5 and its bottom at elevation —45.5. "Lackawanna" arched web sheeting, weighing 35 lbs. per square foot was used. The size and shape of the top portions of the sub-foundation piers were changed to 10.5 ft. square for the river caissons and 8 ft. 2 ins. by 9 ft. 4 ins. for the anchor pier shafts, to accommodate the sheeting and at the same time give the same area as called for in the original plans (see Part I in our Oct. 21, 1914, issue). The sheeting was all driven with a floating pile driver, before the cofferdams were closed, so as not to blockade the river. The method of procedure was as follows:

A corner sheet was first placed. This sheet was then plumbed very carefully in one direction with a transit and in the other by a hand level. It was driven into the mud far enough so that its top was just above the water. The remaining sheets on this side of the shaft were then placed, particular attention being given to the other corner to insure its being vertical. The two adjacent sides of the caissons were set simultaneously, care being taken to keep the correct distance between them. The opposite wall was then set. (It should be noted that the sheets were merely set in place in the mud, all their tops being above water.) The sheeting was then driven by using another 25-ft. steel sheet as a follower. Two steel lugs were bolted to the sides of the follower at the bottom, to keep it in place on the first sheet. The "Worthington" steam hammer used for this work was an additional help in keeping the follower in place. A total of 264 pieces, weighing 139.74 tons, was used. Table XIV gives the labor cost for this work.

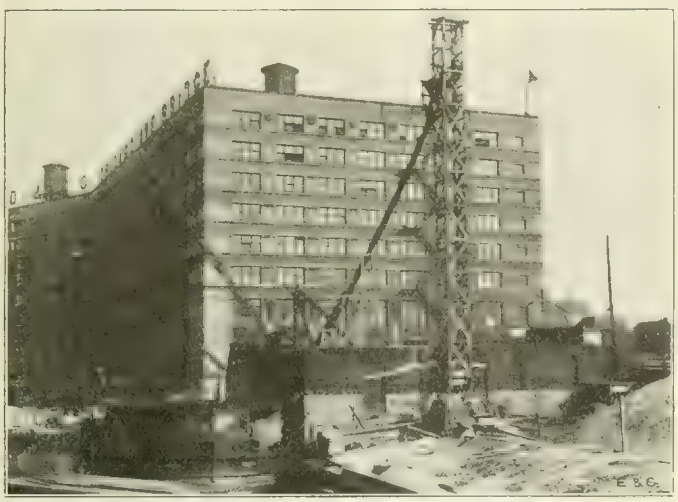


Fig. 7. View of 85-ft. Tower and Chutes for East Piers of Chicago Ave. Bridge—Concrete Chuted Over Adjacent Building—Sand Delivered by Boats.

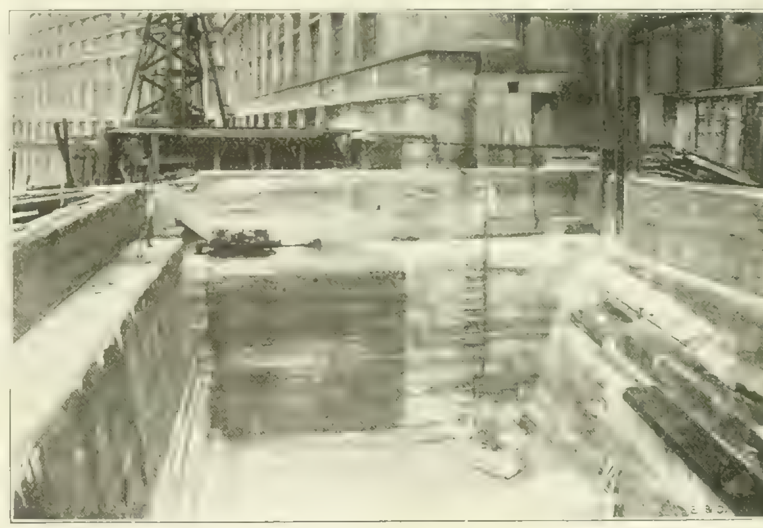


Fig. 8. View from South of East Piers and Pit of Chicago Ave. Bridge—Note Anchor Column and Floorbeam.

TABLE XIII.—LABOR COST OF EXCAVATION.

Item.	Total hours.	Rate, cts. per hr.	Total cost	Remarks
DREDGING WITHIN COFFERDAMS.				
Dredging, 2,651.5 cu. yds.	2,168	..	$712.43	Cost per cu. yd. (0.815 hrs.) = $0.268
Prorated charge	95	.	37.31	
Total	2,263	33	$749.74	
Superintendence, etc., 12.1 per cent	273.8	..	90.72	
Grand total	2,536.8	33	$840.46	Cost per cu. yd. (0.96 hrs.) = $0.32
EXCAVATION OUTSIDE OF COFFERDAMS.				
Excavation for east and west abutments, 1,389.7 cu. yds.	1,660	48	$800.03	Cost per cu. yd. (1.2 hrs.) = $0.58
Prorated charge	75	..	42.00	
Total	1,735	..	$842.03	
Superintendence, etc., 12.1 per cent	209.9	..	101.89	
Grand total	1,944.9	48	$943.92	Cost per cu. yd. (1.4 hrs.) = $0.68
EXCAVATION OUTSIDE OF COFFERDAMS.				
Excavation, 2,949 cu. yds.	6,258	..	$3,136.53	
Prorated charge	280	..	166.31	
Total	6,538	51	$3,302.84	Cost per cu. yd. (2.2) hrs. = $1.065
Superintendence, etc., 12.1 per cent	791.1	..	399.64	
Grand total	7,329.1	51	$3,702.48	Cost per cu. yd. (2.5 hrs.) = $1.25
BACK-FILL.				
Back-fill	667	43	$289.55	
Prorated charge	30	..	14.50	
Total	697	..	$304.05	
Superintendence, etc., 12.1 per cent	84.3	..	36.67	
Grand total	781.3	43	$340.72	
Grand total, all items, 6,990.2 cu. yds.	12,592.1	46	$5,826.58	Cost per cu. yd. (1.8 hrs.) = $0.83

"M"—CONCRETE SHAFT FOUNDATIONS FROM ELEVATION —20 TO ELEVATION —45.

The price bid for completed concrete shaft foundations below elevation —20 and above elevation —45 was 39¼ cts. per cubic foot (net volume). This price included cost of excavation, removal of water, removal of boulders of less than 20 cu. ft. each, and furnishing of all labor, materials, tools, machinery, etc., necessary to do the work (except steel sheeting and reinforcing bars). The quantity of concrete placed was 18,648 cu. ft., making the price bid for this item $7,319.34.

The concrete caisson foundations were divided into two parts, the rectangular portion from elevation —20 to elevation —45 and the circular portion from elevation —45 to bed rock. The upper was already lined with steel sheeting; therefore it was only necessary to place the bracing, which consisted of 6x12-in.

TABLE XIV.—LABOR COST OF DRIVING STEEL SHEETING FOR SHAFTS.

Item.	Total hours.	Rate, cts. per hr.	Total cost.	Remarks.
Preliminary handling of sheeting	125	..	$66.89	
Placing logs on bottom	50.5	..	21.14	
Total	175.5	50	$88.03	
Superintendence, etc., 12.1 per cent..	21.2	..	[illegible]	
Grand total	196.7	50	$98.68	
DRIVING STEEL SHEETING.				
Driving steel sheeting	2,560.5	.	$1,675.08	Av. 8.2 pieces per 8-hr. day; 18.3 hrs. per ton; $11.99 per ton.
Prorated charge	454	.	245.00	
Total	3,014.5	64	$1,920.08	
Superintendence, etc., 12.1 per cent..	364.8	.	232.33	
Grand total	[illegible]	64	$2,152.41	24.2 hrs. per ton, or $15.50 per ton.
Grand total, preliminary and driving	[illegible]	63	$2,251.09	Av. [illegible] pieces per 8-hr. day, 26.9 hrs. per ton; $16.80 per ton.

and 12x12-in. pine spaced about 5 ft. apart, as the excavation proceeded. The material encountered, down to elevation —45 was blue clay and could be removed with shovels. It was first hoisted by means of tripods and windlasses and later with the derrick, the latter being more economical. The waste material was towed in dump scow to Lake Michigan for disposal. The price bid, 39¼ cts. per cubic foot, included the concrete work. The concrete plant will be described under item "K.

Table XV gives the labor costs for the concrete shaft foundations between elevation —20 and elevation —45.

"I"—CONCRETE SHAFTS BELOW ELEVATION —45.

The price bid for completed concrete shafts below elevation —45 was 47½ cts. per cubic foot. This price included the cost of excavation, removal of water, removal of boulders of less than 20 cu. ft. each, furnishing and placing all lagging and iron or steel rings, and all equipment for doing this work. This price did not include reinforcing bars. The total quantity of concrete placed was 14,601 cu. ft., the concrete being a 1:3:5 mix.

The part included in this item is the circular portion of the shaft foundation, the river pier

TABLE XV.—LABOR COSTS OF CONCRETE SHAFTS BETWEEN ELEVATIONS —20 AND —45.

Item.	Total hours.	Rate, cts. per hr.	Total cost.	Per cu. yd.* Hours.	Per cu. yd.* Cost.
PRELIMINARY WORK AND EXCAVATION.					
Loading and unloading from scow the lumber for bracing	32	.	$16.88	.....	.....
Excavating and bracing caissons from El —20 to El. —45	2,470.5	50	1,244.28	3.58	$1.80
Prorated charge	320		174.57	.....	.....
Total	2,822.5	51	$1,435.73	4.1	$2.08
Superintendence, etc., 12.1 per cent.	341.5		173.72	.....	.....
Grand total	3,164.0	51	$1,609.45	4.6	$2.32
CONCRETING BETWEEN ELEVATION —20 AND ELEVATION —45.					
Concrete shafts [illegible]	1,392.3	47	$649.41	2.0	$0.94
Prorated charge	60.1		26.90	.....	.....
Total	1,452.4	[illegible]	$676.31	.....	..
Superintendence, etc., 12.1 per cent.	175.7		81.83	.....	.....
Grand total	1,628.1	47	$758.14	2.36	$1.10
Grand total, excavation and concreting	4,792.1	49	$2,367.59	6.96	[illegible]

*Per cubic foot; 0.258 hrs.; $0.127.

and the anchor pier cylinders being 8 ft. and 7 ft. in diameter, respectively. These diameters were adhered to as closely as possible. The material down to elevation —60 could be removed with shovels; but from this elevation to bed-rock, at an average elevation of —81, it first had to be loosened with grubs. The clay overlying the rock was impregnated with gravel and small boulders, and directly above the rock there was a thin layer black sand, slightly water-bearing. The excavated material was disposed of in the same manner as in the upper part of the shaft, tripods and windlasses being first tried only to be discarded for the derrick, which hoisted the material from the four caissons. The lagging used was 3x6-in. tongue-and-groove lumber, in 3-ft. and 6-ft. lengths. The lagging was held in place by ¾-in. by 4 and 5-in. iron rings spaced about 3 ft. apart.

Table XVI gives the labor costs for constructing the concrete shaft foundations below elevation —45.

TABLE XVI.—LABOR COSTS OF CONCRETE SHAFTS BELOW ELEVATION —45.

Item.	Total hours.	Rate, cts. per hr.	Total cost.	Per cu. yd.* Hours.	Per cu. yd.* Cost.
EXCAVATION.					
Setting windlasses and building material platforms....	483	..	$200.31	.....	.....
Excavating caissons from El. —45 to rock	5,250	50	2,644.10	9.7	$4.90
Prorated charge	533	..	291.02	.....	.....
Total	6,266	50	$3,135.43	11.6	$5.80
Superintendence, etc., 12.1 per cent.	758.2		379.39	.....	.....
Grand total	7,024.2	50	$3,514.82	13.0	$6.50
CONCRETING.					
Concrete shafts below El —45	1,168.4	47	$544.37	2.16	$1.00
Prorated charge	408	..	206.62	.....	.....
Total	1,576.4	48	$750.99	2.92	$1.38
Superintendence, etc., 12.1 per cent.	190.7	..	90.87	.....	.....
Crand total	1,767.1	47	$841.86	3.27	$1.55
Grand total, excavatng and concreting	8,791.3	50	$4,356.68	16.27	$8.05

*Per cubic foot; 0.6 hrs.; $0.30.

"J"—HANDLING AND SETTING SUBSTRUCTURE STEEL.

The price bid for handling and setting substructure steel (furnished by contractor for the superstructure) was 3 cts. per pound. The amount of steel set was 102.27 tons.

The steel set consisted principally of four "knocked down" trusses spanning the caissons, two anchor columns, one floorbeam connecting these columns, and four large anchor bolts for each side of the river. The cofferdam bracing was designed so as to avoid interfering with the erection of these substructure trusses. The pieces were all handled by the derrick, being fastened together with turned bolts. These trusses were set in place before any part of the main piers was concreted. When the concrete reached the proper height the anchor columns and floorbeam were set, particular care being taken to set them accurately.

Table XVII gives the labor costs of handling and setting the substructure steel.

TABLE XVII.—LABOR COSTS OF PLACING SUBSTRUCTURE STEEL.

Item.	Total hours.	Rate, cts. per hr.	Total cost.	Per ton Hours.	Per ton Cost.
Handling steel from cars to site	177		$93.76	1.7	$0.92
Bolting and erecting	1,154	..	803.83	11.3	7.90
Total	1,331	..	$897.59	13.0	$8.80
Prorated charge	133		73.78	.....	.....
Total	1,464		$971.37	.....	.....
Superintendence, etc., 12.1 per cent	177.1		117.54	.....	.....
Grand total	1,641.1	67	$1,088.91	16.0	$10.65

Note.—Derrick and derrick scow were used about 10 days each.

"K"—CONCRETE IN TAIL PITS, OUTSIDE WALLS, SEWERS, ABUTMENTS AND FOOTINGS.

The price bid for the concrete in the piers, tail pits, outside walls, sewers and sewer outlets, abutments, footings, etc., including all labor, materials, forms, etc., was $7.25 per cubic yard. This concrete work did not include that in the caissons, which was let under a separate item. The quantity of concrete placed was 3,604 cu. yds.

The concrete used for this work and also for the caissons was a 1:3:5 mix. Part of the sand used was bank torpedo, hauled in by cars and teamed to the site; the remainder was Lake Michigan torpedo sand, brought to the site by a sand sucker and unloaded by two clam-shell buckets onto a moving belt attached to a 60-ft. boom. By this arrangement the sand was placed practically where it was wanted. The crushed stone used was brought in by teams and boats; that delivered in boats being loaded on skips at the quarry and unloaded at the site by a derrick.

The mixer on the west side of the river was set on top of the approach and the concrete chuted into place (see Fig. 5). The materials were measured in wheelbarrows, a batch being about ¼ cu. yd. The capacity of the mixer was ½ cu. yd. Most of the chutes were built of 2x12-in. plank and were unlined.

At the east side of the river an 85-ft. wooden tower was used. Figure 7 shows a view (looking from the south) of the tower and chutes used in concreting the substructure on the east side of the river. The sand was delivered by boat. The tower was 4 ft. 9 ins. by 6 ft. 3 ins., and was built of 6x6-in. posts and 2x6-in. braces. The main distributing chute was set at an angle of about 30° with the horizontal. The same mixer was used as for the west side.

The forms were built of 2x8-in. planks and 4x6-in. studs placed about 3 ft. apart. The forms on the inside of the pit and on the outside above the water line were *D* and *M* lumber.

Figure 8 shows a view (looking from the south) of the completed east piers and pit. The view also shows an anchor column and a part of the floorbeam.

Table XVIII gives the labor costs for the concrete work under item "K."

TABLE XVIII.—LABOR COSTS OF CONCRETE UNDER ITEM "K."

Item.	Total hours.	Rate, cts. per hr.	Total cost.	Unit costs—Per sq. ft.—Hours.	Cost.	—Per cu. yd.—Hours.	Cost.
MIXING AND PLACING CONCRETE.							
Mixing and placing concrete, 3,604 cu. yds.	7,228.5	46	$3,343.71	.		2.00	$0.93
Prorated charge	570.5		258.00	.		.	.....
Total	7,799	46	$3,601.71			2.16	$1.00
Superintendence, etc., 12.1 per cent	943.7	..	435.81	.		.	....
Grand total	8,742.7	46	$4,037.52	.....	...	2.44	$1.20
BUILDING FORMS.							
Building forms, 38,340 sq. ft.	3,929	60	$2,337.49	0.10	$0.061	1.09	$0.65
Prorated charge	303		186.00	....	...	...	.....
Total	4,232	60	$2,523.49	0.11	$0.066	1.17	$0.70
Superintendence, etc., 12.1 per cent	512.1	..	305.34	....	......	...	.....
Grand total (38,340 sq. ft., 3,604 cu. yds.)	4,744.1	60	$2,828.83	0.125	$0.075	1.32	$0.73
STRIPPING FORMS.							
Stripping forms	1,004.5	50	$ 505.28	..		0.28	$0.14
Prorated charge	77.3	..	39.06	....	...	....	.....
Total	1,081.8	50	$ 544.34	...	..	0.30	$0.15
Superintendence, etc., 12.1 per cent	130.9	.	65.87	.		..	.....
Grand total	1,212.7	50	$ 610.21	...		0.34	$0.17
Grand total, all items	14,699.5	51	$7,476.56	.....	......	2.9	$2.10

"L"—CEMENT MORTAR FOR FACING AND WATERPROOFING.

The price bid for furnishing and placing the Portland cement mortar used for facing and for waterproofing the courses in the tail pits was $11.00 per cubic yard. The quantity of mortar placed was 357 cu. yds.

The mortar used for this work was a 1:2 mix. A 6-in. horizontal mortar course was placed at elevation —18, or about in the center of the tail pit floor, and extended from this course on the outside of the tail pit walls to elevation —2, where the thickness was reduced to 4 ins. From elevation +2 to the tops of these walls the thickness of the mortar course was 2 ins. The inside of the pit had a 3-in. mortar finish on the floor and a 2-in. course on the sides. The small walls on the outside of the main piers were merely spaded, as were the abutments.

The same plant was used for this work as for the concrete work. The mortar was held in place by 1-ft. mortar boards. These boards were set up against the forms, and the concrete was placed up to their tops. The mortar was then placed in the space between the forms and the mortar boards. The boards were then raised, and the operation was repeated. The concrete was thus placed in 1-ft. horizontal layers and the mortar placed against the concrete while the latter was still green.

Table XIX gives the labor costs of mixing and placing the mortar courses.

TABLE XIX.—LABOR COSTS OF MIXING AND PLACING MORTAR COURSES

Item.	Total hours.	Rate, cts. per hr.	Total cost.	—Per cu. yd.— Hours.	Cost.
Mixing and placing mortar, 357 cu. yds.	1,612	44	$ 706.92	4.5	$1.98
Prorated charge	411.6	..	212.40		
Total	2,023.6	45	$ 919.32	. .	. .
Superintendence, etc., 12.1 per cent	244.9		111.24	. .	.
Grand total	2,268.5	45	$1,030.56	6.3	$2.90

"P"—REINFORCING BARS FOR CONCRETE.

The price bid for furnishing and placing the steel reinforcement in the concrete cylinders, shafts, foundations, piers, pit constructions, outside walls, and abutments was 2 cts. placed was 72.85 tons. (See Part I in Oct. 21, 1914 issue.)

The labor costs of this work are given in Table XX.

TABLE XX.—LABOR COSTS OF PLACING REINFORCING BARS.

Item.	Total hours.	Rate, cts. per hr.	Total cost.	—Per ton— Hours.	Cost.
Unloading, sorting and miscellaneous handling of bars	160	..	$ 78.53	.....	.....
Placing bars, 72.85 tons	927.5	.	437.68	.....	.....
Total	1,087.5	47	$516.21	14.9	$7.07
Superintendence, etc., 12.1 per cent	131.6	..	62.46	.....	.....
Grand total	1,219.1	47	$578.67	16.8	$7.95

"S"—DIVERTING AND EXTENDING SEWER.

The price bid for diverting and extending the 5-ft. sewer (two-ring brick construction), including excavation, was $9.60 per linear foot.

As the sewers on each side of the river were located in the center of the street, it was necessary to make a temporary diversion to the side while construction was being carried on. The above price, $9.60 per linear foot, included the connection from the original sewers to the new outlets. The cost of the temporary diversion was included in item "C." On the west side of the river 50.1 ft. of new sewer were built, and on the east side 19.5 ft. were constructed. Practically all of this construction was on a curve and through a 15-ft. cut. The cost of this work is given in Table XXI.

TABLE XXI.—LABOR COST OF CONSTRUCTING 5-FT. SEWER.

Item.	Total hours.	Rate, cts. per hr.	Total cost.	Per linear ft. Hours.	Cost.
Excavating in clay, bracing trench, pumping, laying 2 ring, 5-ft. sewer, and backfilling	1,498	52.5	$787.41	22	$11.31
Superintendence, etc., 12.1 per cent	181.3	....	95.28	..	......
Total	1,679.3	52.5	$882.69	24	$12.70

TABLE XXII.—LABOR COST OF PLACING OAK TIMBER.

Item.	Total hours.	Rate, cts. per hr.	Total cost.	Per 1,000 ft. B. M. Hours.	Cost
Placing 5,564 ft. B. M. oak	375	63	$236.17	68	$42.50
Prorated charge	11	..	4.66	..	......
Total	386		$240.83	69	$43.00
Superintendence, etc., 12.1 per cent	46.7	..	29.14	..	......
Grand total	432.7	63	$269.97	78	$48.50

"E"—OAK TIMBER IN PLACE.

The price bid for the oak timber used in constructing docks and pier protection, bumping timbers in tail pits and permanent sheet piling, including all labor, timber, tools, bolts, nuts, washers, spikes and other appurtenances, was $62.40 per M. ft. B. M. in place. The quantity of oak lumber placed was 5,564 ft. B. M.

The high cost of this work, given in Table XXII, is mainly due to the small quantity of timber used and to the large amount of cutting and fitting required. The excavation required for a part of the sheeting also increased the cost considerably.

"F"—PINE TIMBER IN PLACE.

The price bid for the pine timber used in constructing docks, pier protections, etc., including all labor, timber, tools, bolts, nuts, washers, spikes and appurtenances, was $39.00 per M. ft. B. M. The amount of pine timber placed was 5,128 ft. B. M.

The high cost of placing this timber, given in Table XXIII, was due to the same causes as were given for placing the oak timber.

TABLE XXIII.—LABOR COST OF PLACING PINE TIMBER.

Item.	Total hours.	Rate, cts. per hr.	Total cost.	Per 1,000 ft. B. M. Hours.	Cost.
Placing 5,128 ft. B. M. pine	526	57	$ 284.48	97	$55.50
Prorated charge	11	..	22.30	...	......
Total	537	..	$306.78	105	$60.00
Superintendence, etc., 12.1 per cent	65	..	37.12	...	......
Grand total	602	57	$343.90	117	$67.00

"R"—FURNISHING AND ERECTING STRUCTURAL STEEL.

The price bid for furnishing and erecting structural steel was 2.5 cts. per pound. The quantity placed was 9,136 lbs.

This steel was principally chains for the pile clumps, and the work was so closely allied to the pile driving that its labor cost was merged with the cost of pile driving.

PERSONNEL.

The work described in this article was in charge of John Ericson, city engineer of Chicago; Thos. Pehlfeldt, bridge engineer; Alexander von Babo, designing engineer; Clarence Rowe, construction engineer; and Carl O. Johnson, resident engineer. The Byrne Bros. Dredging and Engineering Co. was the contractor for the substructure, for whom Thos. Lynch was superintendent.

Erection of Steelwork of Building by Locomotive Crane.—The New Jersey Zinc Co. has recently constructed a bag house at Palmerton, Pa., the steel framework of which was erected by a "Bay City" 25-ton locomotive crane fitted with a long boom. The building is 64½x805 ft. and has a height, to the lower chords of the roof trusses, of 59 ft. The framework is comparatively light and none of the pieces handled in the field weighed more than 2 tons. In erecting the heavier pieces the locomotive crane moved along a longitudinal track which extended from one end of the building a short distance beyond the other end. The lighter pieces of the framework were erected by hand. The building was erected by the Pittsburgh Construction Co.

Orders were recently placed in Pittsburgh for 100 miles of 80-lb. steel rails for a South American railway. This is the largest South American order for steel rails as yet received in this country.

SEWERAGE

Design of the Sewerage System for the Panama-Pacific International Exposition, San Francisco, Calif.

Contributed by William Clyde Willard, Assistant Engineer, Division of Works.

DESCRIPTION OF SITE.

The site of the Panama-Pacific International Exposition is in that section of San Francisco, locally known as Harbor View, lying at the westerly end of the city's north waterfront and on the shore of San Francisco Bay, just within the Golden Gate, and covering an area slightly in excess of 600 acres including some 85 city blocks and a nearly equal area lying in the United States Government Military Reservations of "The Presidio" and "Fort Mason."

When this location was selected as the site for the Exposition, an area covering some 20 city blocks at Harbor View was a tidal basin over most of which the water was from 5 to 12 feet deep at mean high tide. Also, a second area covering almost two-thirds of that portion of the site lying within the Presidio, was a marsh in which the depth of water ranged from 2 to 5 ft. The first work towards preparing the site for the Exposition consisted of filling in these water areas with material hydraulically dredged from the bay, and although completed about 18 months ago, a slight settlement is still observed over the deeper portions of the fill.

Approximately three-fourths of the site is on nearly level ground lying at an elevation of only 3 to 6 ft. above high tide. The settling hydraulic fill and the low elevation, together with the decision to have but one sanitary outfall sewer for the entire site, offered serious obstacles to the design and construction of the sewer system.

In all work of the Exposition the San Francisco City datum plane was taken as zero for elevations. This datum is 11.768 ft. above mean low water, the datum of the U. S. C. & G. S., the U. S. Army Engineers and the California State Harbor Commission. Much of the ground was below city datum so it was necessary in almost all work to deal with positive and negative elevations.

From data collected at the Presidio, covering the time interval 1897-1907, the elevations of different tide stages, below city datum, were as follows:

Lowest tide observed	14.360
U. S. C. & G. S. zero	11.768
Mean of all low waters	10.624
Mean sea level (U. S. G. S. zero)	8.666
Mean of all high waters	6 621
Highest tide observed	3.600

At the time the site was taken over by the Exposition, three city outfall sewers existed in the Harbor View section of the grounds. These paralleled each other running from south to north. The easterly one, a 6-ft. circular sewer in Laguna Street, emptied into a small cove of the bay; the second one was in Pierce Street, about 2,500 ft. further west and discharged through a 6-ft. circular section into the tidal basin; the third one, having a 5-ft. circular section, was in Baker St., some 1,900 ft. still further west, and discharged directly into the bay. The outer 500 ft. of the last sewer was in poor condition and the sewage was really discharged through a break several hundred feet from the end, the sewer being filled with sand from that point to the outfall. Besides these trunk sewers there were over 15,000 ft. of branches ranging in size from an 8-in. vitrified pipe to a 2-ft. 6 in. by 3 ft. 9 in. brick, egg-shaped section. All these carried both sanitary sewage and storm water.

In the Presidio section of the site there existed a 12-in. ironstone pipe and a 12-in. cast iron pipe, both sanitary sewers and both discharging into the bay beneath a wharf; a 10-in. ironstone pipe storm sewer emptied onto the ground above the east end of the marsh; a 3-ft. x 3 ft. 6 in. timber box storm sewer and a 24-in. ironstone pipe storm sewer emptied into the marsh.

Immediately after the filling of the tidal basin at Harbor View was completed, the city commenced construction of an extension to the Pierce St. sewer from its former end, at Bay St., 1,773.75 ft. north across the fill to the bay, making a submerged two-compartment outlet having a flowline elevation of —23.00 ft. at the outfall. This sewer is of concrete, circular in section and increases in diameter from 7 ft. to 7 ft. 6 ins., and then to 8 ft.

The city next constructed two intercepting sewers, one west from the Laguna St. sewer to the Pierce St. sewer and the other east from the Baker St. sewer to the Pierce St. sewer. The former is approximately 3,100 ft. long, with sections 2 ft. 6 in. x 3 ft. 9 in. at Pierce St., this section being egg-shaped with flat top, and diminishing in size to a 12-in. ironstone pipe at Laguna St. The second intercepting sewer is approximately 1,935 ft. long with sections flat roofed, egg-shaped 2 ft. 6 ins. x 3 ft. 9 ins. at Pierce St., 2 ft. x 3 ft. intermediate, and 21 in. ironstone pipe at Baker St. The intercepting sewers were designed to carry only the dry weather flow of the two trunk sewers, together with such sewage as they picked up along their run. The sewage was deflected from each trunk into its intercepting sewer by a weir placed just below the point of junction. The storm water flow, carrying the diluted sewage, passes over the weir and on down the respective trunk sewer to the bay.

The intercepting sewer west of Pierce St. was planned to cross an arm of the Yacht Harbor which it was originally intended to fill. However, it was found cheaper to carry

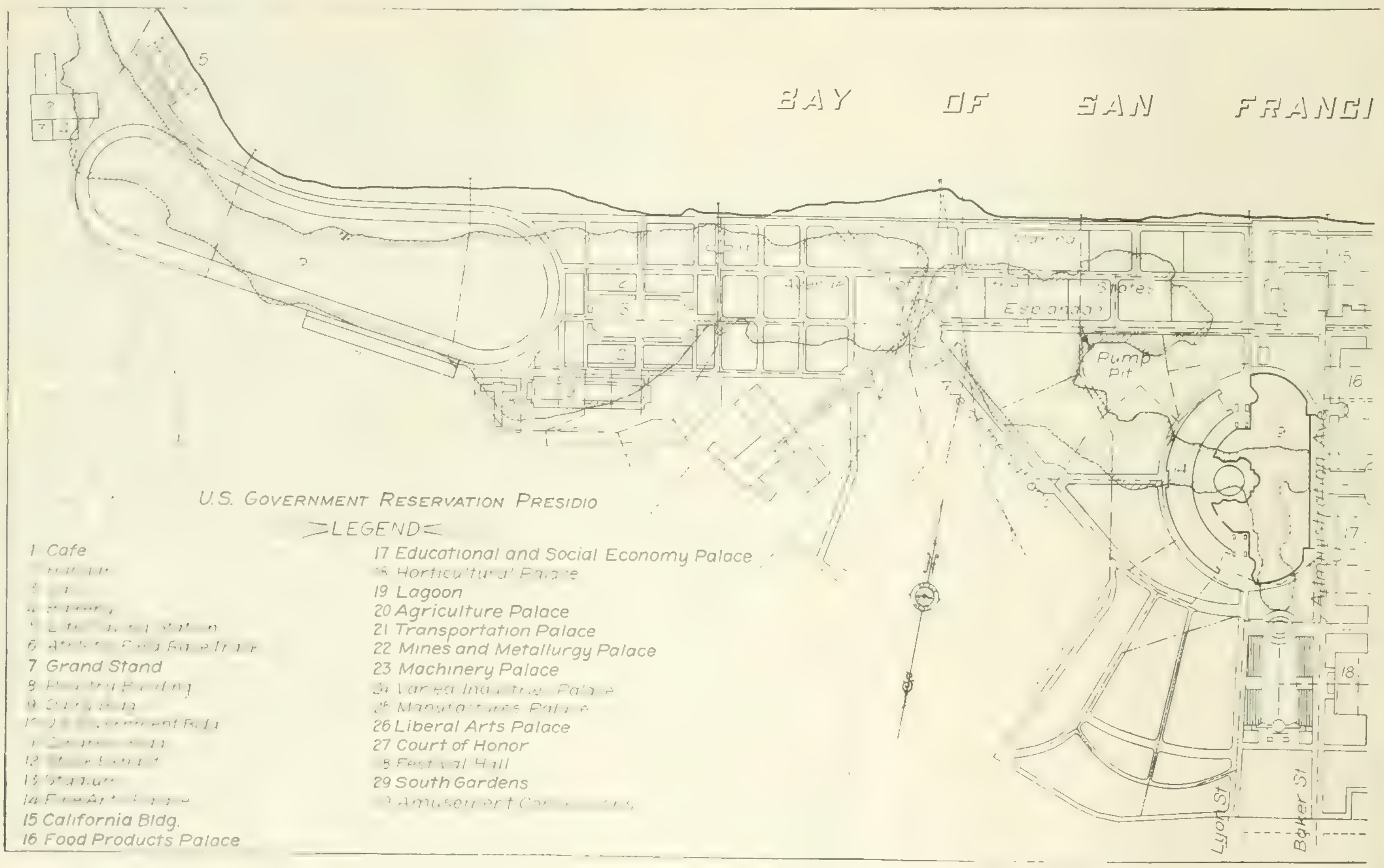


Fig. 1. Block Plan Showing Sewerage System for

the roadway, in which the sewer was to be laid, across this arm of water on a trestle than to make a solid fill of it, so a 30-in. wood stave pipe, with staves 3 ins. thick, and hung from the trestle, was substituted for 280 ft. of the 2 ft. x 3 ft. concrete section.

A short time after the completion of the two intercepting sewers, the city awarded a contract for removing the damaged outer portion of the Baker St. sewer and replacing it with 713.75 ft. of concrete sewer having a 5 ft. 6 in. x 6 ft. section, and a submerged outlet with a flow line elevation of —22.00 at outfall.

The construction of the two intercepting sewers leading into the Pierce St. sewer was the result of an understanding between the Exposition Company and the city to make the Pierce St. sewer the outfall for the dry weather flow of the entire exposition site, and the adjacent sections of the city. The reason for this was the fact that the Pierce St. outfall is far more advantageously situated with respect to tidal currents than either the Laguna St. or the Baker St. outfalls and the sewage will be carried away by the tides, thereby preventing the long beach line of the Exposition—which is to be made a very attractive feature—from becoming fouled, the storm water going directly into the bay through the four different outfalls shown and the sanitary sewage being pumped into the Baker St. sewer above the deflecting weir at the west intercepting sewer, through which it finally reaches the Pierce St. outfall.

GENERAL ASSUMPTIONS.

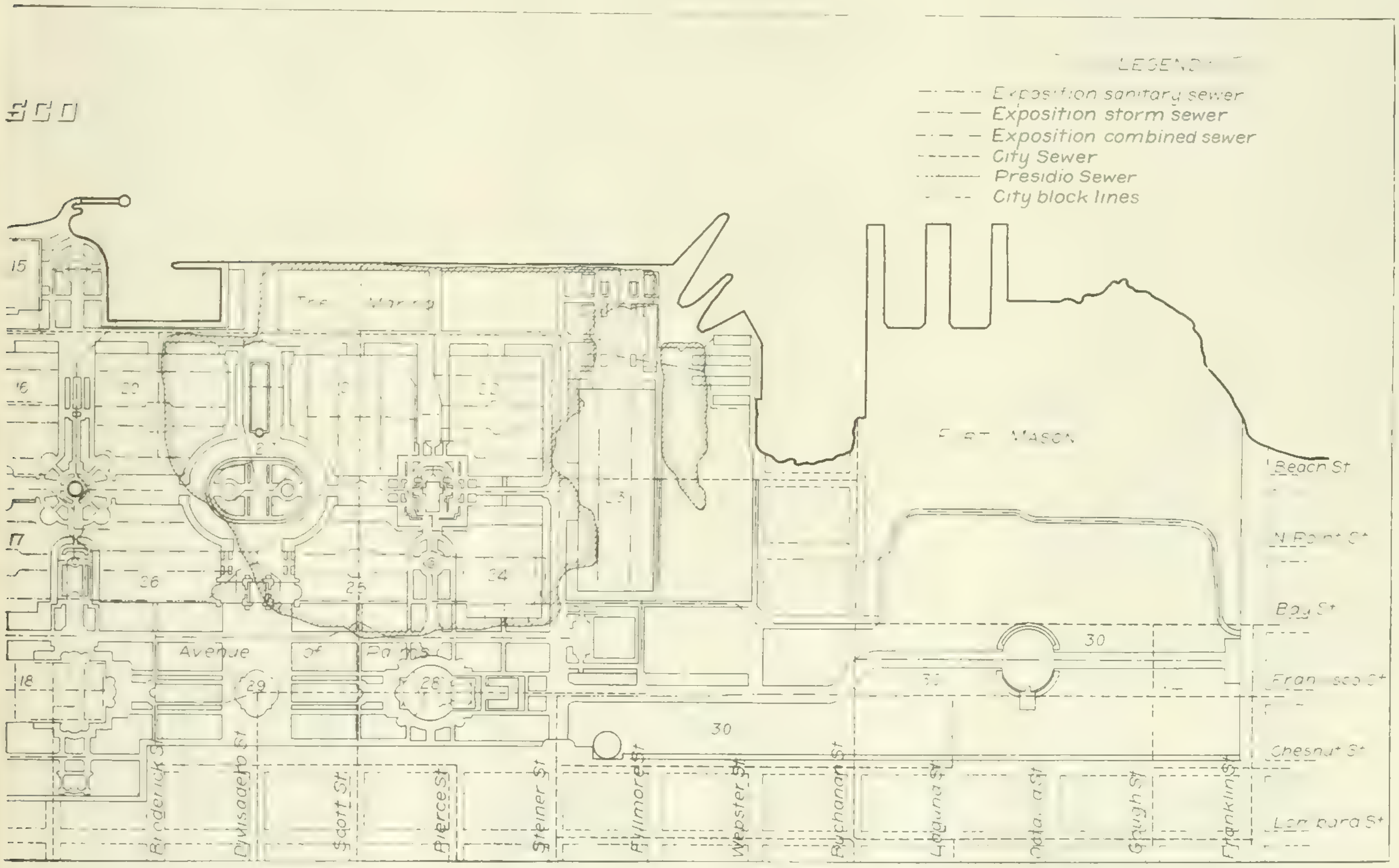
In computing the sewer sizes the rainfall rate table prepared several years ago by the San Francisco City Engineer's office and still used by them, was used. The curve in Fig. 2 was plotted from this table. Referring to Fig. 1 for the position of the various districts named, the runoff was assumed as shown in the above tabulation.

District.	Runoff. Per cent of rainfall.	Interval of rain giving maximum run-off, minutes.
Amusement concessions	70	5
Areas covered by exhibit palaces	100	5
Paved areas of courts, roadways, entrances, etc	100	5
All gardens	25	5 to 10
Entire area for state and foreign pavilions	40	5 to 1[illegible]
Stock exhibit district	70	5 to 9
Drill ground and athletic field	30	5 to 1[illegible]
Steep hillside above drill ground	40	10
Race track	[illegible]	5 to 10

From an investigation of the United States tide tables for San Francisco Bay, the curve in Fig. 3 was plotted. This gives the average normal time in hours per day at which the tide is at or below certain elevations. Since much of the Exposition site is on low ground, in some places the finished surfaces of roadways and gardens being but a few inches above the top of the large city sewers, it was impossible to design a system which would not be subject to hindered or impaired operation at the highest tide during a heavy rain. The use of pumps to remove this condition would have largely increased construction costs, and many times increased operating costs. The City Engineer's office, in designing the Pierce St. sewer extension, computed a hydraulic grade line elevation of —2.00 ft. at a point some 200 ft. from the outfall, which will only be obtained when the tributary area of the city is built up; the conditions imposed by the presence of the Exposition, however, undoubtedly more than equal this.

The finished surface of the Exposition at the point above mentioned on the Pierce St. sewer roadways and gardens is —1.5 ft.

In the general design it was, therefore, necessary to assume that the possibility of simultaneous occurrence of extreme high tide and heavy rain fall was exceedingly small, and in any case could only prevail for a very short period during which the temporary flood condition resulting would not damage or inconvenience the Exposition to any extent as gardens and roadways only could be affected, the building floors being all set well above the zone of danger.



Panama-Pacific International Exposition, San Francisco, Calif.

inevitable result if numerous sanitary sewage outfalls existed. Therefore, in designing the sewer system for the Exposition site, where any sanitary sewage had to be cared for, the city sewers were used as the backbone of the system. A great amount of storm water was also emptied into them, but where their capacity was insufficient, the storm water was kept separate and carried directly to the bay by the separate storm sewer outfalls.

In Fig. 1 is shown the block plan of the Exposition site and on it the skeleton outline of all city, U. S. Government and Exposition sewers. The areas within the various shaded lines show the extent of the hydraulic filling In the sections given over to State and Foreign Pavilions and the Stock Exhibit Section, the ground being low in elevation, and remote from the Pierce St. sewer, the storm water and sanitary sewage were separated; the

From Fig. 3 it will be seen that on an average the tide is at or below —6.0 for 23 hrs., 34 mins. per day; at or below —7.0 for 12 hrs. 54 mins. per day; at or below —8.0 for 12 hrs. 12 mins. per day; at or below —9.0 for 6 hrs. 36 mins. per day, and at or below —10.0 for 3 hrs. 10 mins. per day. From a study of these elevations and durations, a height of —7.0 (approximately 0.4 ft. below the Presidio record of the mean of all high waters) was taken as the point from which to start the hydraulic grade line for obtaining the required sewer sizes, where the sewer in question emptied directly into the bay. In case the sewer emptied into one of the city trunk sewers, the inside top of the city sewer was taken as the point from which to start the hydraulic grade line of the joining sewer. In the case of the two intercepting city sewers, the inside top of these sewers at their junction with the Pierce St. trunk, was taken as a point on the hydraulic grade line and their capacities computed back therefrom.

Besides receiving a small quantity of sewage from the Baker and Laguna Street sewers, the two interceptors will be required to carry the roof water and sanitary sewage from the Agriculture, Mines and Machinery Palaces and the dry weather, or sanitary, flow from one half the Liberal Arts, the Education and Social Economy, Horticulture and Food Products Palaces. The roof water from the Machinery Palace alone is estimated at 17 second feet. The flow from these palaces taxes the capacity of these two sewers, and in the Court of the Four Seasons it was necessary to include a separate storm system emptying into the Yacht Harbor.

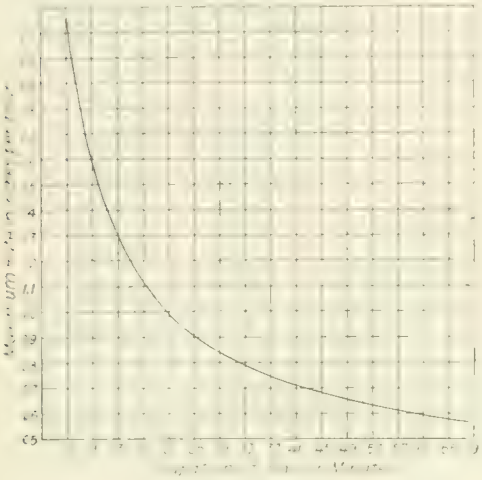


Fig. 2. Curve Showing Maximum Rate and Duration of Rainfall for San Francisco.

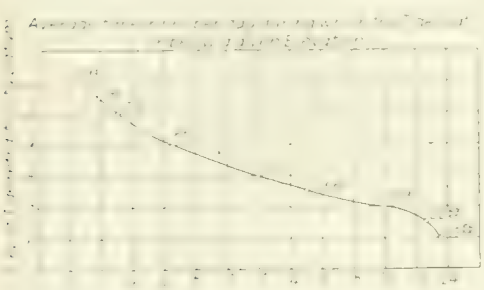


Fig. 3. Curve of Tide Duration at the Site of the Panama-Pacific International Exposition, San Francisco.

The runoff from the roofs of all Exhibit Palaces was computed for a 5-minute interval and 100 per cent runoff. The mouths of down spouts were flared enough theoretically to take the capacity of the vertical pipe. One square inch of cross-section of downspout was provided for each 500 sq. ft. of roof surface, but no downspout less than 5 ins. in diameter was used in any Exhibit Palace except for the Fine Arts and Horticulture, where as small as 2 ins. was used.

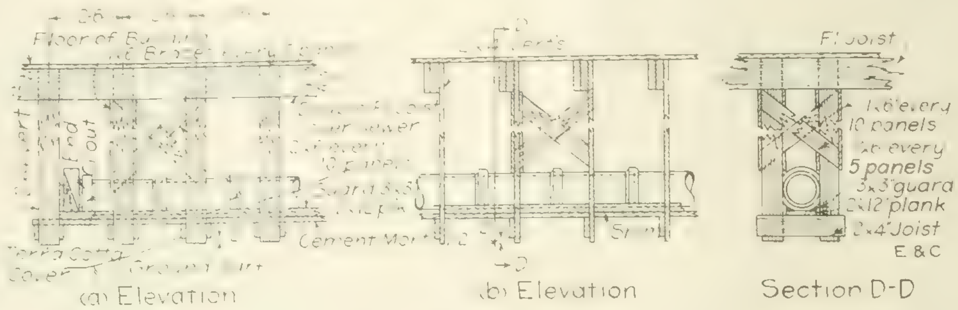


Fig. 4. Method of Supporting Sewers by the Piling Under Buildings on Filled Ground.
(a) Sewer line parallel and (b) sewer line perpendicular to floor joists.

UNIQUE ASPECT OF PROBLEM.

In designing the structures for the Exposition sewer system, the problem to be solved was entirely different from that for any municipal sewer system. First, all parts of the system, with but a few exceptions described later, were to be temporary, their usefulness expiring with the close of the Exposition in December, 1915. To provide a more permanent system would add unnecessarily to the initial cost. Besides, much of the land leased by the Exposition Company was leased with the agreement that at the close of the Exposition the Company should restore the ground to its original condition. Therefore, the more temporary the sewer structures, the more easily can they be removed later. Second, the cheapest material to meet the above conditions had to be selected. Third, where the sewage is pumped, and also for some combined sewers, the pipes had to be laid in sand at a depth ranging from 0 to 10 ft. below the normal elevation of the ground water, so that the material used, though temporary and cheap, must be as watertight as it would be possible to make any permanent sewer.

After considering these requirements, second quality vitrified ironstone pipe, both dipped and undipped machine banded fir wood-stave pipe, continuous-stave redwood pipe and timber box culverts were used to fit conditions in different sections of the site. From an investigation of all available records it was found, before determining upon the use of wood-stave pipe, that such pipe had been used in a few places to a very limited extent for sewer purposes. The total length of wood-stave pipe sewers built by the Exposition is approximately 21,720 lin. ft., or over four miles. This, undoubtedly, establishes a new record in this line of construction for wood-stave pipe.

The required size of the ironstone pipe sewers was obtained from the Williams and Hazen tables for tile sewers. The size of the wood-stave pipe was obtained from diagram published by Theron H. Noble based on the formula,

$$Q = 1.28\ D^{2.65}\ H^{.555}, \text{ where}$$

Q=discharge in cubic feet per second
D=internal diameter in feet
H=head in feet in 1,000 feet.

In all cases, with a few exceptions under some of the Exhibit Palaces, in the State and Foreign Pavilions districts and the Stock Exhibit district, both the ironstone and wood-stave pipes were laid on the following minimum grades:

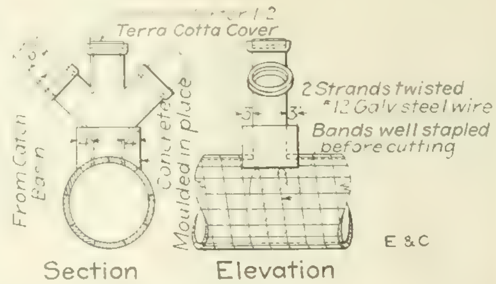


Fig. 5. Details of Double Catch Basin Connection for Wooden Stave Pipe.

Size of pipe, ins.	Minimum per cent of grade.
6	1.0
8	0.7
10	0.5
12	0.4
15	0.3
21	0.2
24 and over	0.1

The exceptions to this practice occurred at

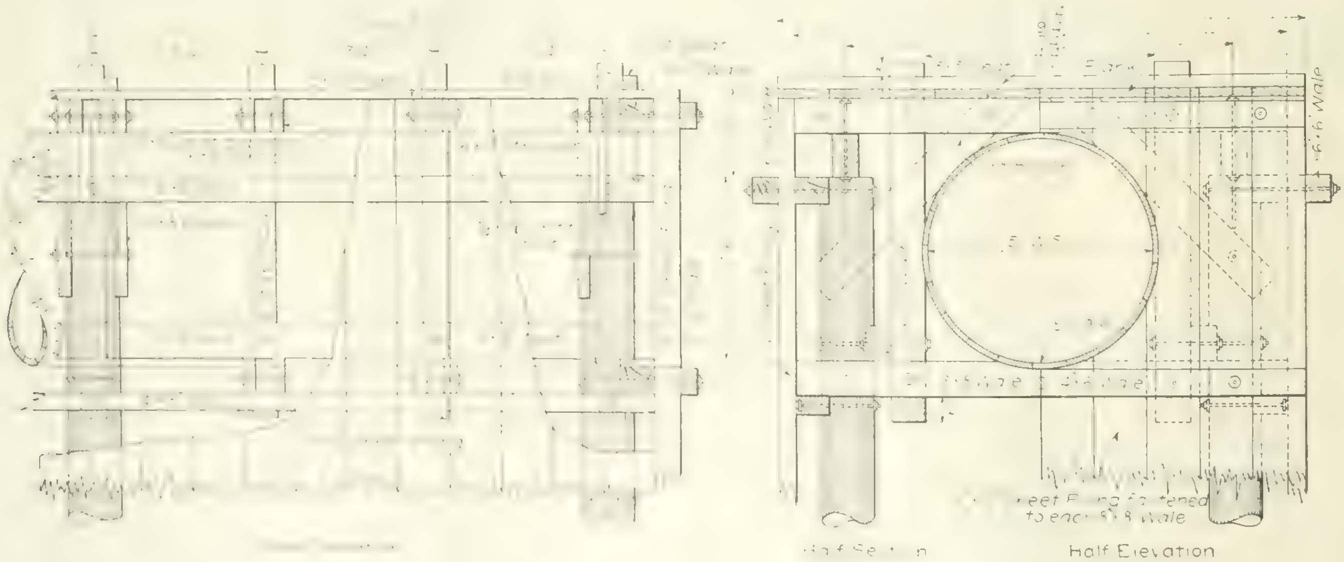


Fig. 6. Section and Elevations of Outfall Structure for 51-in. Sewer, Panama-Pacific International Exposition.

the upper ends of a few sewers where, by placing the small pipes on slightly flatter grades than the above, the cost of the system could be considerably reduced by reducing the depth necessary for the lower sections.

SEGREGATION INTO DISTRICTS.

Amusement Concessions.—This district offered no difficulties. City sewers existed in many of the former city streets and the ground lies higher than in any other portion of the site. Five separate systems carrying combined sewage were installed. The size ranged from 6-in. to 14-in. ironstone pipe, and in every case the grade was good and the sewers operate under no head.

Main Exhibit Palace District.—The three city trunk sewers, with their several branches, are in this district and were used as a basis for the Exposition systems. Twelve Main Exhibit Palaces, the Festival Hall and Service Building drain all roof water and sanitary sewage ultimately into these sewers, and in addition, about three-fourths of the ground

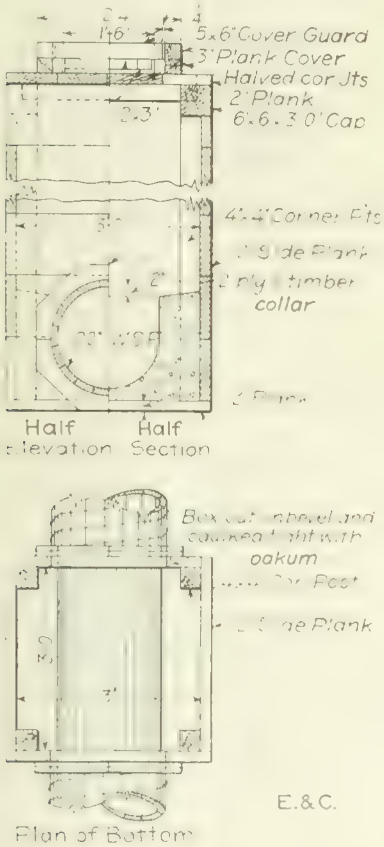


Fig. 7. Details of Standard Timber Manhole, Panama-Pacific International Exposition Sewerage System.

surface runoff of the district reaches them. The roof water from the above structures alone is estimated at 158 cu. ft. per second.

In constructing the sewers, all those under buildings and including connections to the nearest manhole, were included in the plumbing contract for the respective building, while the portions outside the buildings were let in the form of sewer contracts. A total of 46,071 lin. ft. of sewers, ranging from 6 ins. to 28 ins. in diameter, will be constructed in this district, 38,277 lin. ft. of which, ranging from 6 ins. to 24 ins. in diameter, are included in the Fine Arts, Education and Social Economy, Food Products, Agriculture, Liberal Arts, Manufactures, Transportation, Mines and Metallurgy, Varied Industries, Machinery and Horticulture Palaces, the Festival Hall and the California State Building.

In all cases ironstone pipe was used for 6 in. and 8-in. sizes, and in some cases for sizes as large as 24-in. where the ground was solid. The structures erected over the filled ground have piled foundations because of the constant settlement of the fill, and the sewers under these structures are hung from the building floor on timber brackets, as illustrated by Fig. 4. All sewers thus supported, ranging from 6 ins. to 24 ins. in diameter, are ironstone pipe. Wood-stave pipe would have been much lighter, also cheaper for the larger sizes, but the difficulty of making numerous taps for service connections, a "Y" branch being left each 25 ft., decided in favor of the ironstone pipe. For the larger sizes outside the structures wood-stave pipe was used.

There are few direct connections to the wood-stave pipe, as most catch basin and service connections are made at manholes, but where it is necessary to do so, direct connection is made by using a concrete collar, as shown in Fig. 5. Under the eight central exhibit palaces the ground was borrowed to a depth of several feet below the level of the ground outside the structure, in many places the borrow going down to the level of the ground water. Where it seemed probable that water would seep in and stand under the structure, a 6-in. vertical stand pipe was put on the sewer and made of such length as to extend well above the computed hydraulic grade line. (Many of the sewers supported under the buildings will normally discharge under a head of from 1 to 4 ft.) A removable, tight cap was placed over the end of the

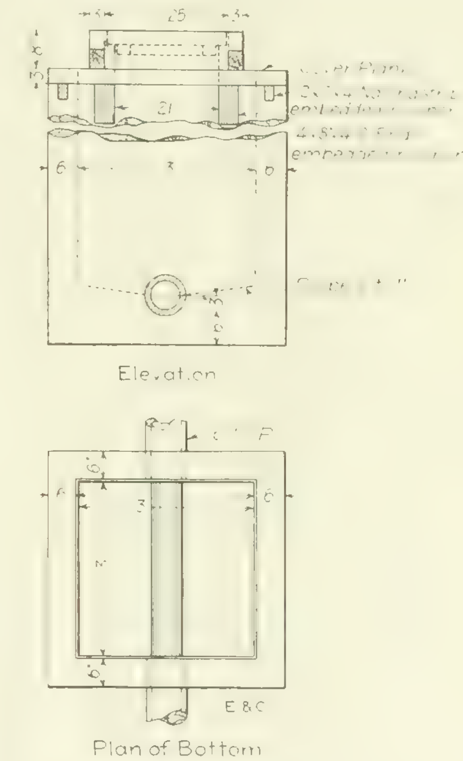


Fig. 8. Standard Shallow Manhole of Concrete With Timber Top, for Sanitary Sewers, Panama-Pacific Exposition.

vertical pipe. Each line of sewers under the exhibit palaces has several downspout connections—(the downspouts being brought down the building columns)—none of which is trapped as it is intended to let the downspouts act as sewer vents, no other vents being provided for the exhibit palaces.

This section of the grounds contains numerous pools and fountains each of which has a drain connected to a sewer. In practically every case the bottom of the pool is below the hydraulic grade line for storm conditions, making it necessary to provide each drain with a check valve.

State and Foreign Pavilions District.—The system in this district consists of one combined, one sanitary and three storm systems. The southern end of the district is on higher ground, making it possible there to combine the sewage and storm water and discharge it by gravity into the Baker St. sewer. The larger part of the district, however, is low and flat making it impossible to get the sanitary sewage to Baker St. without pumping. Therefore, for this low ground, separate systems were provided for the storm water and sanitary sewage, the latter draining by gravity into a concrete sump centrally located in the district and having a bottom elevation of —13.4 ft. Some 40 feet, south of the sump was sunk a redwood tank, weighted down against flotation with a concrete casing, and containing two 10-in. horizontal Krogh centrifugal pumps, each connected by a silent chain drive to a 25 HP., 3-phase, 60-cycle, 220-volt, 1,800-r.p.m. Westinghouse motor mounted at the same elevation as the pumps. The pump speed was specified as 500 r.p.m.

The pumps are mounted sufficiently below the elevation of the sump bottom so that each pump is fed by gravity through an 18-in. wood-stave pipe from the sump. A 6-in. stand pipe carrying a float which operates a float switch, rises from each feed pipe just outside the pump pit. The floats are so arranged that the two pumps are thrown into action at different elevations. As the sump is pumped out, the switch cutting out the motor is thrown as the float descends and the motor is at rest till the sewage again rises in the sump and stand pipe and the motor again

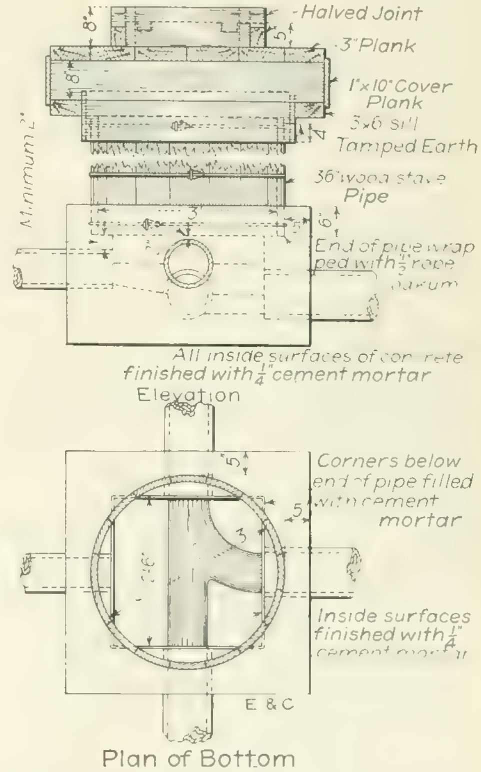


Fig. 9. Standard Deep Sanitary Sewer Manhole of Wood Stave Pipe With Concrete Bottom and Timber Top, Panama-Pacific Exposition Sewerage System.

starts as the rising float throws the switch. The specifications required each pump to have a capacity of 2,500 gals. per minute, against a 10 ft. head. The discharge from the pumps is through a 20-in. machine banded fir, asphalt dipped, wood-stave pipe wrapped for a 50-ft. head and being 1,168 ft. in length.

The sump is constructed in the form of a double manhole with a rectangular flap gate connecting the two compartments at —4.75 elevation. One compartment acts as a manhole on a 40-in. storm sewer running directly to the bay. In case the power operating the pump is shut off or the pumps are inactive for any other reason, the sewage will rise in the sump till it reaches the flap gate and will then overflow into the storm sewer. The flap gate acts only in one direction, so that should a combination of rain and tide raise the water in the storm sewer above —4.75 elevation, it will not overflow into the sump.

In determining the required capacity of the pumps, 1,800 gals. per minute was computed as coming from the two Presidio sanitary sewers, which were connected with the Exposition system and 900 gals. per minute from the Stock Exhibit District. For the State and Foreign Pavil[illegible] that there would be 70 buildings, each with eight fixtures fed by a ½-in. water pipe under a 30-lb. pressure at the main 200 ft. distant. Such a pipe would discharge about 2.5 gals. per minute for the 70 buildings, making the total amount of sewage to be pumped as 4,100 gals. per minute.

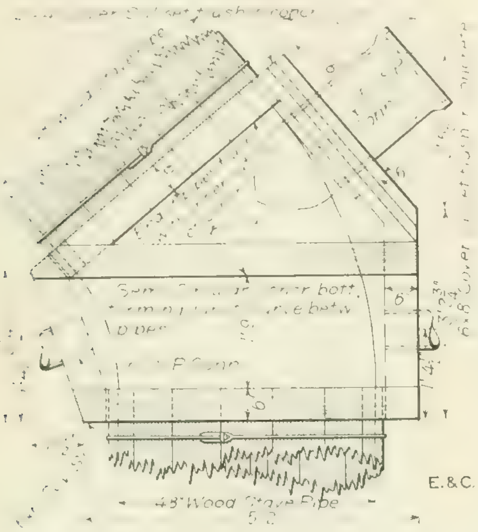


Fig. 10. Plan of Special Manhole of Concrete With Timber Top for Use on Large Wood Stave Sewers.

This sanitary sewer is laid at from 1 ft. above to 8 ft. below the level of the ground water, and to reduce to a minimum the leakage into the pipes and correspondingly the work of the pumps, wood-stave pipe was used for all sizes 10 ins. in diameter and over. The service connections will be infrequent in this district, and where connections cannot be made at manholes a direct connection will be made in a manner similar to that shown in Fig. 5.

The 3 ft. x 3 ft. 6 in. storm water box previously mentioned was in poor condition at the time the Exposition Co. started work, and in making the hydraulic fill in the marsh the box was allowed to fill with sand. In all other sections of the site the sewer system is of temporary construction, the intention being to abandon it at the close of the Exposition period. The United States Government, however, required that all storm sewers in the Presidio which connected in any way with their existing drainage system, be of permanent construction. To meet this requirement continuous-stave redwood pipe with galvanized bands and concrete manholes was used. The storm water formerly carried by the abandoned box is now carried by a continuous stave redwood pipe 46 ins. in diameter, at the upper end and 51 ins. in diameter at the outfall. This 51-in. pipe is the largest sewer built by the Exposition.

In constructing this large storm water pipe it was found that the 12-in. ironstone sanitary sewer from the Presidio interfered at one point. The 12-in. pipe was broken at this point and connected to the storm sewer through which the sanitary sewage will discharge into the bay, till the sewer pumps are started. The old 12-in. cast iron sewer from the Presidio was supported on piles and was found to be above finished Exposition grade, so that it had to be entirely removed. Where this sewer reached the high ground, a connection was made to the same storm sewer mentioned above and the sanitary sewage is handled in the same way as for the other 12-in. pipe. During the Exposition period these two sanitary sewers will be connected to the Exposition sanitary system, and their sewage will pass through the pumps and eventually reach the Pierce St. outfall. After the close of the Exposition the two sewers will again be connected to the storm sewer which will thereafter permanently act as a combined sewer for the Presidio.

During the first heavy rains of the past winter, a length of about 100 ft. of the 46-in. section of this wood-stave storm sewer floated to the surface carrying a foot or so of backfill up on top of it. This caused some consternation and a hasty going over of computations to see how much more of the various wood-stave pipe sewers might be expected to come floating up. In designing the sewers this danger was thought to have been amply cared for, and no other trouble of the kind has occurred. Several conditions seem to have contributed to the one failure. The system was then incomplete. The drainage of the newly filled area was imperfect, and the filled marsh became so thoroughly saturated that it was practically a liquid offering very little resistance to the raising of the pipe. In order to prevent a recurrence, a timber platform was placed on the section affected and the deck loaded with sand. No further trouble was experienced.

Immediately east of the Fine Arts Palace lies a lagoon covering 6.08 acres, into which is drained the roof water of the Fine Arts Palace, itself covering 6.13 acres, and the gardens surrounding the lagoon with an area of 1.4 acres, comprising a total area of precipitation of 13.61 acres. The surface of the lagoon was fixed at elevation —5.0 ft. and the bottom at —7.0 ft. and the allowable rise of lagoon level due to run-off from storm was fixed at 2 to 3 ins.

To meet this condition there was installed an underground overflow weir having a length of 15 ft. with a crest elevation of —5.0 ft. and receiving water from the lagoon through a wooden box intake 1 ft. high by 4 ft. wide, provided with wooden bar screen at the lagoon end and set with flow line at —6.0 ft.

The weir itself is in the form of a "hollow square" built within a box 6 ft. 3 ins. x 6 ft. inside dimensions, the water from the lagoon surrounding the inner weir and discharging on three sides over the crest to an inner box through which it is conveyed to a connection with a 36-in. wood-stave storm sewer, by means of a wood box drain 1 ft. 5 ins. high by 2 ft. 6 ins. wide inside dimensions, and about 170 ft. long. The flow line elevations of the upper and lower ends of the box drain are —6.0 ft. and —6.45 ft. respectively. At the lower end of the drain which discharges into a concrete chamber opening to the storm sewer, was placed a timber and iron tide gate or flap valve to prevent water from backing up in case of high tides, which occasionally rise to elevations considerably above the level of the overflow weir.

This system will be sufficient to carry away all rainwater at normal tide stages without causing a rise in lagoon exceeding the allowance mentioned above.

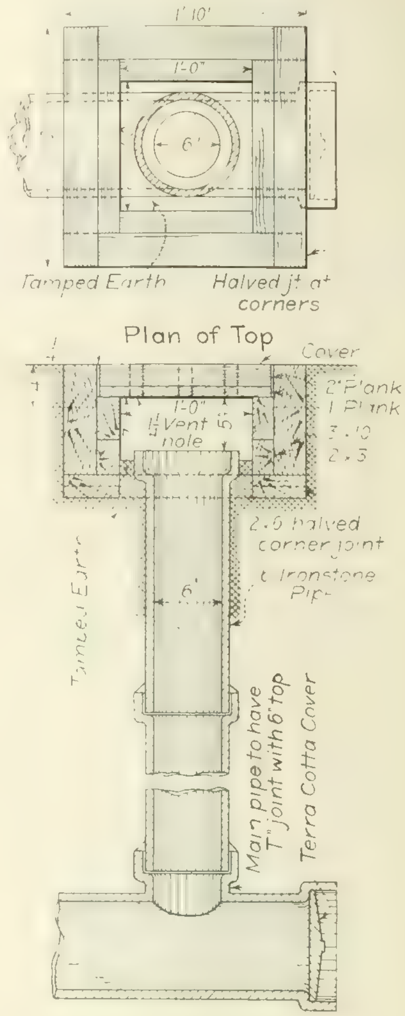


Fig. 11. Plan of Top and Section of Standard Lamphole, Sewerage System, Panama-Pacific Exposition.

When a storm occurs at a time of high tide, the storage capacity of the lagoon is sufficient to care for a short period of run off.

The outfall structures for the large storm water pipes consist of a boxed-in pile structure, similar to that shown in Fig. 6, the dimensions varying with the size of pipe. Six of these structures were required. Smaller outfalls in the Main Exhibit Palace district were hung from wharves.

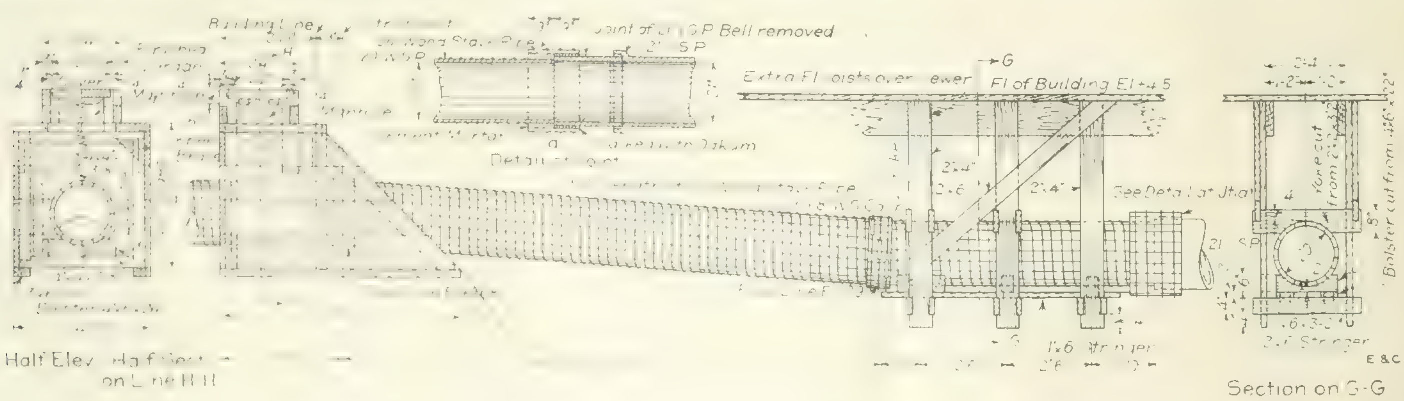


Fig. 12. Typical Transition or Junction Structure Where Sewer Passes From Settling Ground to a Rigid Support.

Stock Exhibit, Race Track, and Drill Ground Districts.—The sanitary sewage from the Stock Exhibit District is conveyed to a connection with the sanitary system in the State and Foreign Pavilions District, passing through the pumps and eventually reaching the main outfall sewer in Pierce Street. These sewers range in size from 6 ins. to 16 ins. in diameter, those of 10 ins. and over being of wood-stave pipe to prevent excessive infiltration of ground water, while the smaller sizes are of ironstone pipe construction.

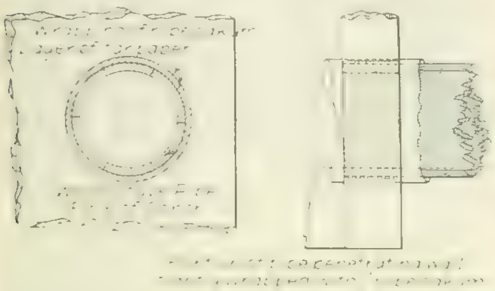


Fig. 13. Standard Joint Where Wood Stave Pipe Enters Concrete Manhole.

An overflow manhole of special design, with a 3-ft. weir having a crest elevation of —5.0 ft. discharging directly into the main outfall storm water sewer of this district, and provided with a tide gate, was installed so that in case the pumping plant was idle for any cause, a temporary outlet for the sanitary sewage would be found through the storm water system.

The storm water sewer system for this district was divided into three parts, the Westerly States sites and Live Stock section being drained through a main outfall sewer of wood-stave pipe with diameters of from 30 ins. to 40 ins., discharging into the bay with branches ranging from 8 ins. to 22 ins., being of ironstone pipe and those of 18 ins. or more of wood-stave pipe. Existing 24 and 21-in. sewers coming from the Presidio were connected with this system and the overflow connection with the sanitary system previously mentioned was provided.

Immediately south of the Live Stock section, a small area of level ground receives the drainage from the low hills of the Presidio. Around the foot of these hills an open wood box drain 2 ft. in width and 850 ft. long with a level flow line at an elevation of —5.70 ft. was built. Connections were made at the easterly end with the storm water system of the Westerly States sites, and at the westerly end with that of the Race Track section. Connections were made with the U. S. Government drains from the Presidio above and sumps placed to intercept surface flow in the gullies. To lower the water plane of the level tract mentioned the joints of the box were purposely left open to allow seepage water to enter more readily.

The storm water drainage of the Race Track and Drill Grounds is taken care of by two lines of 18-in. wood-stave pipe and timber outfalls.

Timber sumps with grating tops were placed at points adjacent to the margins of the track and the surface drainage conveyed thereto by open ditches in gutters 4 ft. inside the line of the track. Some lines of agricultural tile drain were laid to carry away excessive ground water and connected with the sumps.

West and south of the Race Track section lies an area of hillside drainage comprising about 218 acres, all of which concentrates in a gulley discharging near the southwesterly turn of the track. To intercept this drainage an intake of sheet piling with a floor of large broken stone was installed at the mouth of the gulley, from which a wood box drain 2 ft. high by 4 ft. wide and 1,250 ft. in length carries the storm waters to the bay. This box was laid on a slope of 0.0039 and has a capacity of about 50 second feet.

After the above described drain was installed, the U. S. Government took steps to lay out a site for stables in the valley south of the intake. The sewer system planned for the site will discharge into the intake end of the drain, thus finding an outlet to the bay and obviating any necessity for further provision being made by the Exposition Company for draining this section.

Structures.—Several different types of manholes were used, the first consideration being cheapness of cost, and the second consideration, particularly so with the sanitary system connected with the pumps, being water-tightness. Also, permanence was required on sewers to be used afterward by the U. S. Government. Several types were tried out before the three forms in Figs. 7, 8 and 9 were adopted as standard. The first of these is all timber and is used where the seepage is moderate and watertightness and permanence are unessential. The second is of concrete with timber top and is used where permanence is required or for shallow depths where seepage is excessive. The third has a concrete bottom, a redwood stave pipe body and detached timber top and is used for depths over 6 ft. where watertightness but not permanence, is required. For the large wood-stave pipes, special concrete manholes were constructed, a typical one being shown in Fig. 10.

At the time these manholes were designed the character of the roadway pavement had not been determined upon. As the height of the crown would depend on the width and character of the pavement, the manhole tops were designed so that the covers could be raised or lowered to fit the pavement elevation, with very little difficulty. As a rule lampholes were so installed that no pipe run would be over 300 ft. The standard lamphole is shown in Fig. 11.

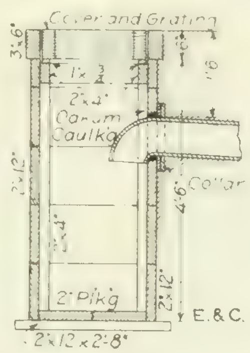


Fig. 14. Section of Wooden Catch Basin, Panama-Pacific Exposition, Sewerage System.

As before mentioned, the sewers under buildings located on hydraulic fill are swung from the building floors and consequently receive a positive and unyielding support from the piles forming the building foundation. Where the sewers leave the building and traverse the fill, the portion outside the building is liable to certain settlement, while the supported portion under the building remains fixed. The reverse condition also exists in several places, particularly where Exposition sewers in the settling fill connect with city sewers, all the latter being supported on deep pile foundations and not liable to settlement. To meet this condition of transition from fixed to yielding foundation for sewer conduit, a special flexible short section was designed to connect the fixed conduit with the portion subject to settlement, the latter being set arbitrarily at a lower elevation and the flexible section set at a sufficiently greater slope to connect the two sections. It was necessary to assume that the run of unsupported sewer would be subject to approximately uniform settlement throughout a given zone and the increased grade in the flexible section was determined as nearly as possible from the observed settlement of the adjacent part of the hydraulic fill. All these junctions so far have proved satisfactory. Through them a drop of from 10 to 15 ins., according to location, is allowed for in from 14 to 20 ft. The necessity for providing these settlement joints is shown by the fact that about the middle of March, when the unsupported sewers from the Mines and Metallurgy Palace were connected to the main sewer it was found that the former were from 8 to 11 ins. lower than the elevations shown for them on the plans, none of them having been in the ground more than six months. Fig. 12 shows a typical transition or junction structure where sewer passes from settling ground to a rigid support.

Where wood-stave pipe, particularly the larger sizes, enters concrete manholes, the end of the pipe penetrating the wall of the manhole was wrapped with rope oakum protected by a layer of tar paper to prevent absorption of grout from the concrete while being poured; it being desired that the rope oakum would remain flexible and take up the expansion of the wood-stave pipe due to moisture and prevent cracking of the concrete manhole. Observation so far indicates the efficiency of this design. All joints inspected are watertight and no cracking of concrete walls has occurred. Fig. 13 illustrates this circumferential expansion joint.

The catch basin design also underwent a growth before a standard was adopted. The first design was nothing more than an open manhole on a storm sewer with a grating top so that the water drained directly into the sewer. Fig. 14 shows the next form tried, a number of which were built. It was expected that the quarter curve of the ironstone pipe extending downwards into the rectangular timber catch basin would trap the sewer gas, but as the soil was principally sand, it was impossible to build the catch basin sufficiently water tight to maintain a continuous seal. A 6-in. cast iron backwater valve was then substituted for the quarter curve ironstone pipe. The valve kept out the sewer gas, but about doubled the cost of the catch basin. The last and most satisfactory design is shown in Fig. 15. In this design an oak oil or liquor barrel is used for the sump, a circular hole is cut in it and a quarter bend of ironstone pipe used for the trap. The barrel is watertight and a continuous seal is maintained. The grating top is the same for all types. Besides the other advantages mentioned, the barrel catch basin proved to be cheapest in first cost, which was, set complete, $7.75, while the timber catch basin with backwater valve cost approximately $13.

This cost of catch basins and pipe connections is not included in the sewer items, but is taken to be a part of the work of constructing the roads and pavements of the Exposition.

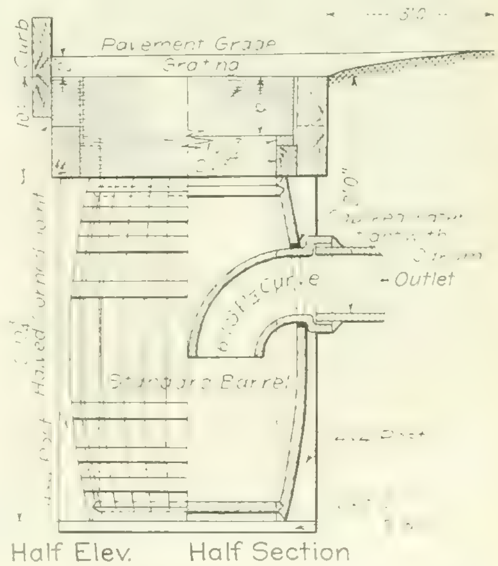


Fig. 15. Detail of Catch Basin With Barrel Used for Sump, Panama-Pacific International Exposition Sewerage System.

GENERAL.

In all cases lump-sum bids were called for, covering all work included in the plans and specifications. Whether it was due to this method of bidding or to the scarcity of other construction work at the time in the city, the majority of contracts were let at unusually low prices.

All ironstone pipe specified was of second quality. Where wood-stave pipe was used the specifications called for standard machine-banded fir pipe wrapped for a 25-ft. head. This was used for all obtainable sizes, the maximum being 22 ins. in diameter. Continuous stave pipe was used for larger sizes. Exceptions to these general rules have already been described.

Since the construction of many of the sewers the block plan of several of the districts, particularly that of the State and Foreign Pavilions, has been entirely changed and the sewer locations which were made to fit the former plans, are now not so advantageously located as could be desired.

The total length of the sewer system as planned and now practically constructed, including catch basin connections and city sewers within the Exposition site, is 149,977 lin. ft. or approximately 28.4 miles. The total mentioned is made up as follows:

	Lin. ft.
Exposition sewer system proper—not including catch basin connections	101,758
Catch basin connections, 4, 6 and 8-in. pipe	28,34[illegible]
City sewers within th[illegible]	19,8[illegible]

Cost.—It is impossible to state with absolute accuracy the cost of the Exposition Sewer System, as many drains still remain to be installed at this date, the plans for which have not yet been completed and which will be perfected only as conditions requiring sewer service are known. The bulk of the work in this connection is, however, accomplished and the following approximate costs are given:

1. Sewers installed under sewer contracts proper—18 separate contracts having been awarded—which included the installation of approximately 53,000 lin. ft. of pipe	$ 95,780
2. Sewers installed in connection with building contracts, which work was done under the plumbing item for the respective buildings, approximately 44,000 lin. ft.	35,000
3. Sewers installed by exposition working forces under the direction of the Bureau of Civil Engineering, approximately 32,000 lin. ft., 28,000 lin. ft. of which consists of 4, 6 and 8-in. catch basin connections; therefore sewers proper laid by the exposition forces amount to about 4,000 lin. ft.	8,313
4. Catch basin connection installed consist of 28,344 lin. ft. of 4, 6 and 8-in. pipe. These were installed entirely by the exposition forces, but were charged to roads and pavements and the cost was not segregated from other items of road construction. Therefore no segregated cost can be given	....
Total	$139,093

The apparently low unit cost for sewers installed under Item 2 as compared with Item 1, is accounted for by the fact that a comparatively large proportion of the sewers included in building contracts were of small diameter and their installation was accomplished with a small amount of excavation, the space under the building permitting this.

The apparently high-unit cost of the sewers installed by the Exposition—Item 3 above—is accounted for in part by the proportion of redwood box culverts installed; and further, the fact that the sewers installed by the Exposition consisted of small isolated sections generally in bad ground and which could not consistently be included in contracts.

The plans of the Sewer System were prepared and the work executed by the Division of Works of the Exposition Company, of which Mr. H. D. H. Connick is Director of Works and Mr. A. H. Markwart, Assistant Director of Works. A large part of the work was designed by the writer under the immediate supervision of Mr. E. E. Carpenter, Chief Civil Engineer.

ROADS AND STREETS

Road Building With Convict Labor in Fulton County, Georgia.

Contributed by W. T. Wilson, County Engineer of Fulton County, Atlanta, Ga.

Fulton county, Georgia, the county seat of which is Atlanta, the capital of the state, has been using convicts for road construction for a number of years. At the present time approximately 700 convicts are employed on the county roads. This article describes the equipment and methods of construction used in Fulton county and discusses the cost of accomplishing work with convict labor.

Both white and colored convicts are employed but the races are segregated. One camp is composed entirely of whites. Six other camps of about equal size are composed entirely of negroes. These camps are permanently located at advantageous points 6 to 8 miles apart—the county is about 30 miles long and 8 miles wide—and comprise barracks, stables, kitchen and guard living quarters of wooden construction.

Equipment.—The present equipment includes the following:

1 Thew steam shovel.
6 5½-ton motor trucks.
1 traction engine.
[illegible] trailer cars.
12 Maney 4-wheeled scrapers.
4 15-ton road rollers.
[illegible] stone crushers.
2 concrete mixers.
[illegible] asphalt heating wagons.
1 bituminous mixing plant.
1 pile driver.
350 mules.
[illegible]

R[illegible] sprinkling wagons, [illegible] road dr[illegible] [illegible] tools, [illegible] and other necessary small tools.

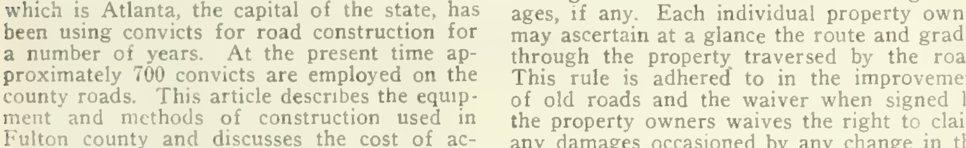


Fig. 1. Typical Cross Section Used on Roads in Fulton County, Georgia.

[illegible]

The first step taken to build a new road is to make the preliminary survey and profiles, which is done after carefully looking over the section of the county in which it is proposed to build the road and deciding on one or more routes. Preliminary lines and profiles are run and an estimate made considering the comparative amount of material to be moved, grades, culverts, drains, etc. Usually the line showing the lowest cost of construction is adopted. After the adoption of the line, the location is made and the road side-staked. A map and profile is attached to the petition, containing waiver of claims for resulting damages, if any. Each individual property owner may ascertain at a glance the route and grades through the property traversed by the road. This rule is adhered to in the improvement of old roads and the waiver when signed by the property owners waives the right to claim any damages occasioned by any change in the alignment or alteration of the existing grade. This method of procedure eliminates the possibility of any delay being caused in the construction of the road, by the filing of injunctions or protests with the Board of County Commissioners, asking for changes in the alignments and grades.

The maximum grade allowed on any of the highways in the county as fixed by resolution of the Board of Commissioners is 4 per cent, but in remote instances this maximum has been increased to as high as 8 per cent in very rugged localities. There is no fixed rule as to the degree of curvature adhered to in the location of the curves, this being left to the discretion of the engineer.

CLEARING AND GRUBBING.

After the right-of-way has been secured, the location having been previously made and the road side-staked, the convicts are placed at work clearing and grubbing the right-of-way of all trees and stumps. The timber is cut and corded, and hauled to the different camps for fire wood, unless it is stipulated in the waiver that the property owner is to have the use of the wood. After cutting the timber the stumps are blasted, this being the easiest and quickest way of disposing of them. These are gathered and placed in the fills, as this provides for their disposal and removes the unsightly stumps from along the side of the finished road.

When the clearing and grubbing have been completed, the drains are located and constructed, all small streams and dry runs being taken care of by laying vitrified clay pipe of sufficient size to provide for the natural drainage. This pipe is laid with Portland cement joints and proper catchbasins and headwalls constructed of rubble masonry, laid in cement mortar. The stone used in the construction of the basins and headwalls is secured either on the line of work in making the excavations or from some suitable quarry located convenient to the work.

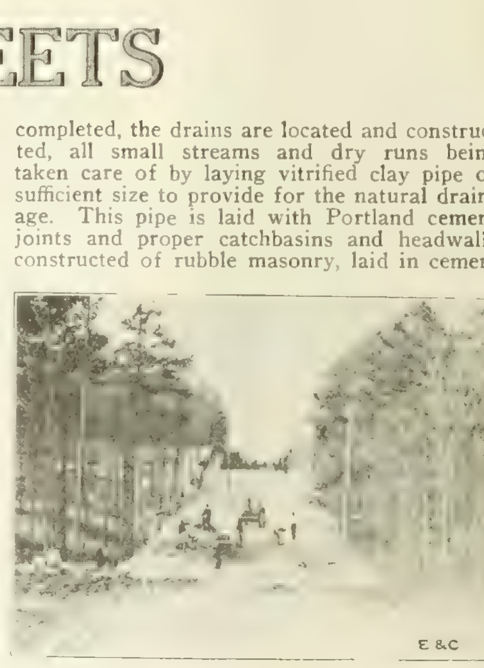


Fig. 2. Method of Grading. Note Vertical Sides of the Cut.

The larger streams are spanned by concrete or wooden bridges, these bridges often being 100 to 200 ft. in length. The bridges in the county, with five or six exceptions, are of wooden construction, being built either upon piles or land bents. The repairs to these wooden structures amounts to about $5,000 a year. They are being replaced as rapidly as possible with reinforced concrete bridges.

After the drainage structures have been placed the men and teams begin the work of grading the road to the width and profile as furnished, and upon the completion of the grading the road is allowed at least one year to settle before the work of paving begins.

QUARRYING AND CRUSHING STONE FOR PAVING.

Preparatory to paving, a quarry containing suitable stone is located near the road to be paved and a contract is entered into with the owner or agent of the property upon which the quarry is located, to allow the county to strip the quarry, if new and never before used, or to quarry stone out, if already opened. The contract usually calls for the payment of a stipulated amount per cubic yard, averaging about ten cents a yard, to be paid the owner or agent after the work of paving has been completed. All payments are made upon measurements and estimates of the completed work. The crusher is moved to the site of the quarry and after the stone is quarried and crushed it is hauled by the trucks and teams and deposited on the road and used in laying the foundations, gutters and top surface of the pavement.

The quality of stone used is granite or gneiss, the only stone available in this locality. The cost of quarrying, crushing and hauling the stone from the crusher to the paving job costs on an average of 60 cents a cubic yard, but consideration should be given to the fact that the labor employed is only that of the convicts, with the exception of the engineer at the crushing plant, the powder man at the quarry and the truck and team drivers, who are white and free laborers.

Fig. 3. Type of Maney Scraper Extensively Used with Convict Labor.

Fig. 4. Crushing Plant. Note Method of Piling Uncrushed Stone Around the Plant.

GUTTERS AND PAVEMENT.

The first step in laying the pavement is the placing of the gutters, Fig. 1, which are of rubble, and, since the average width of the roadways in the county is 30 ft., the gutters are laid for a water-bound macadam of a varying width. On the summit of the grades the gutters average 3 ft. in width, gradually widening until the foot of the grade is reached, where they are 5 to 6 ft. in width, according to the length of the grade down which the water flows.

After laying the gutters the macadam surface is placed and finished with a thin layer of granite screenings. The cost of the water-bound macadam road, including the rubble gutters completed, is 75 cents a square yard.

OILED MACADAM ROADS.

The year following the completion of the water-bound macadam road, during the summer months when the surface is heated by the sun, the road is gone over with a revolving brush broom, such as used in street cleaning, and the accumulated dust is swept into the gutters, where it is allowed to lie until the application of the hot road oil is completed. Heavy liquid petroleum asphalt, containing not less than 80 per cent of asphalt, is applied at a temperature of 225° F. at a rate approximating ½ gal. to the square yard on the initial treatment. Simultaneously with the spraying of the road with the hot oil an air blast, driven by a fan, blows all of the remaining particles of dirt and dust out of the interstices of the pavement, allowing a close incorporation with the road surface, and the asphaltic oil is applied under steam pressure. Following the spraying machine, the convicts cover by hand the entire surface of the oiled road with granite screenings, which are hauled in the trucks from the crushers and deposited at intervals alongside the road for this purpose. The cost of the oiling is 6½ cents a square yard for the initial treatment, and 3½ cents a square yard for each successive treatment, using one-half the amount of oil to the yard applied on the initial treatment on successive treatments. Constant vigilance is the price of an oiled road, and such roads are being gone over continually and carefully watched. At the least sign of cracking or disintegration, a repair squad is immediately sent out and the necessary repairs made.

In Fulton County, there are over 50 miles of oiled roads. Each summer a light treatment is given each road previously oiled and the roads completed the year before are given their initial treatment.

ASPHALT MACADAM AND ASPHALTIC CONCRETE.

The usual process is employed in laying gutters for asphalt macadam pavements, except that the width of the gutters is uniform, instead of varying, as in the case of water-bound roads. Two years ago a policy was adopted of laying the rubble gutters 8 ft. in width on each side of the road, leaving in the center 14 ft. to be paved. The object in laying the gutters a uniform width of 8 ft. is to allow the travel on the road under construction to continue uninterrupted while the center is being laid. After the gutters are down the traffic proceeds along the road, keeping to the right, and in this manner no interference is had with laying the hot surfacing material in the center of the road.

The binders used by the county in constructing asphalt macadam roads include Texas, California, Bermudez, Trinidad and Cubanal asphalts and Standard Oil asphalt. We have found by experience that the binders

Fig. 5. Finished Roadway and a Grade Crossing Elimination.

Fig. 6. Concrete Arch Highway Bridge Built by Convicts.

ANALYSIS OF ASPHALT BINDER.— —NAME AND NUMBER OF CAR.——WHERE USED AND DATE.

Date of Analysis	Initials of Car	Car No.	Date Received	Melting Point Degrees F.	Evaporation 7 hrs. 325° F.	Flash Point Degrees F.	Burning Point Degrees F.	Soluble in Carbon Disulphide	Soluble in 88° Penn. Naptha	Mineral Matter	Penetration 32° F. 200 grms. 1 min.	Penetration 77° F. 100 grms. 5 sec.	Penetration 115° F. 50 grms. 5 sec.	Road Where Used	Manufactured by

(Original size of form 8½ x 16½ inches.)

Fig. 7. Form Used in Recording the Analysis of Asphaltic Binder.

used in the east will not meet climatic conditions in the south, since owing to the extremely long hot season a binder with a higher melting point must be used here. No binder is used in this county that has a lower melting point than from 65° to 85° F., experience having shown that a binder with a lower melting point will bleed and run during the hot season.

As each car of asphalt is received samples are taken from the shipment and sent to the laboratory for a chemical analysis and a record is kept of each car, showing a complete analysis.

By way of an experiment last year the county paved several roads with two different brands of asphalt mixed in the proportion of 60 per cent of one kind and 40 per cent of the other. After one year's traffic the roads on which the mixture was applied are showing very satisfactory results. The idea in trying the mixing of the binders was to obtain an average between an asphalt with a low melting point and one with a higher melting point, in order that the melting point of the binders, averaged, would not be lower than the minimum desired.

There are at present in Fulton County, in addition to the oiled roads, approximately 50 miles of asphalt macadam and bitulithic pavements and 190 miles of dirt roads.

REPAIR OF DIRT ROADS

The dirt roads are under the supervision of an assistant superintendent, who has detailed under him from each of the camps a squad of 15 convicts, 1 guard and 2 teams with drivers.

Supplied with road machines, steel road drag, scrapes and plows, the repair squads working from each camp devote their time to resurfacing and ditching the dirt roads. After heavy rains the repair squads go to work immediately filling in the holes, cleaning out the ditches and crowning the center of the road.

COST OF CONSTRUCTION BY CONVICTS AS COMPARED WITH HIRED LABOR.

On an average haul of 500 to 600 ft., earth excavation costs an average of 20 cents a cubic yard. The same material moved with hired labor at $1.50 per day (the prevailing rate for labor) would cost at least 40 cents per cubic yard. The method used in moving earth in cuts too shallow to allow the use of the steam shovel is that of working the cut to a face, sinking drill holes, and shooting with light charges of explosives, after which it is loaded by the convicts into the wagons. In heavy cutting, it is much more economical to operate the steam shovel and load direct into the wagons.

The cost of rock excavation figures close around $1.00 a cubic yard with convicts and $2.00 with hired labor. The building of concrete retaining walls, abutments, culverts and work of a similar nature costs $6.50 a cubic yard, while work of the same character performed by hired labor costs $9.00 to $11.00 a cubic yard.

Water-bound macadam averages 75 cents a square yard when the convicts are employed, as against $1.50 a square yard, the ordinary local contract price.

Notes on the Relative Values of Paving Materials.

The selection of the type of paving material best suited to the needs of traffic is one of the important problems of the city engineer. A statement of the general laws controlling such a selection is made by Geo. W. Tillson in a paper before the Cleveland Engineering Society, which paper is given here in part.

STONE BLOCK PAVEMENTS.

Granite and the harder sandstones are principally used in the stone block pavements of this country. The particular kind to be used will depend upon the availability and cost. It would be foolish to consider granite as a material where it must be obtained at great expense and where a good sandstone is available. In this study however, granite is considered, as it is a material that is used in New York City and one with which the author is most acquainted.

In this connection it should be stated that the figures arrived at in this study by the author must be varied in every locality and must vary to a great extent according to the judgment of each individual engineer. He wishes it understood, however, that he does not consider the exact figures as of so much importance as the method of obtaining the results. So that too much importance should not be given to the results shown.

The granite pavements of today are very much better than those that were laid even four or five years ago. It has been found that on account of their being laid on a concrete foundation it has been possible to reduce the depth of the blocks and make them of somewhat smaller size otherwise, thus rendering it possible to get a better cut block at the same expense as before and allowing the blocks to be laid with a closer joint, thus reducing abnormal wear.

The blocks are laid on a sand cushion on a concrete base, the joints being filled with cement grout, tar and gravel, tar pitch alone, and sometimes a combination of pitch and sand. The author believes that with a small joint and a combination of good pitch and sand the best results will be produced. While a cement grout joint makes a smooth pavement, it is often difficult to close the street long enough to allow the cement to set, and it also makes a pavement that is exceedingly difficult to repair after it has been torn up for any substructure purposes.

In considering the value of different materials as applied to different streets it is assumed that an intelligent selection of the different materials has been made, as upon that will depend the results entirely. For instance, a granite pavement could be laid on a residence street with light traffic, where its durability would be long and the cost of maintenance practically nothing. On the other hand, an asphalt pavement could be laid upon a heavy business traffic street, where its life would be short and the cost of maintenance enormous. If, however, an intelligent selection of material is made for all streets of the city, what will be the natural life and cost of maintenance will be found.

In 1913 in the Borough of Brooklyn the granite pavements cost 3.9 cts. and the Medina sandstones 6 mills per square yard, all on concrete.

WOOD BLOCK PAVEMENTS.

Wood pavements have been laid at intervals in this country for some 70 years. The first pavements were not only of untreated wood, but of wood selected without much regard for its natural durability. The result was failure, as could have been expected. The repeated failures of the different kinds of wood, however, although they delayed, did not prevent entirely the establishment of wood in this country as a standard material. The success of the wood pavements in Europe made it positive that they could be laid successfully in this country under proper conditions.

The first treated wood pavement in this country was laid on Tremont St., Boston, in 1900. This pavement has been in use during this entire period with very small repairs, and is in good condition at the present time. It is composed of blocks treated with a composition made up of one-half creosote oil and the other half resin. Pavements of this character were laid in New York and other cities, but on account of the increased price of resin it was dropped out of the mixture and creosote oil only used.

There has been a great deal of controversy as to the character of the creosote oil to be used for this purpose, the principal point being the specific gravity. The theory of mixing the resin with creosote was that, it being a more stable material, it would prevent the volatilization of the creosote and so preserve the blocks from decay for a greater length of time. The object of treating the wood is to prevent decay and also to make the blocks stable by preventing the absorption of water so that they will not shrink in dry weather or swell in wet and thus cause bulging. It was thought that by using a heavier oil this result could be obtained as well as by the use of a light oil with resin. The relative values of the heavy and light oils have never been determined, but the author has always been in favor of the heavier oil.

The present method of laying wood pavements in this country has not been in use long enough to determine what the cost of maintenance is, but figures obtained from St. Louis, Minneapolis and other cities indicate that it is exceedingly small. The first pavement of this character to be laid in Brooklyn was in 1902, and it has had practically no repairs for wear and tear since laid. It is, however, on a light traffic street. The total cost of maintenance of the wood pavements in Brooklyn in 1913 was 1.4 cts. per square yard on pavements that had been in use from 8 to 11 years. This cost, however, included, besides actual wear and tear, damage caused by openings in the pavement, although not the cost of repaving the openings themselves. In Paris in 1911 the cost of repairs was 26 cts. per square yard. In London the average cost is 20 cts. per square yard.

The fillings for joints in wood pavements are sand, asphalt or coal tar pitch, and cement grout. The practice in Brooklyn has been to fill the joints with sand, and first class results have been obtained. Some people, however, prefer the bituminous and others the cement filler; either will give good results when properly used, although the author does not look with much favor on the cement grout filler.

There is no question that wood block is an important paving material, and on streets where the traffic is heavy and noise is a great detriment it is most advisable. Its strongest point is its noiselessness and its weakest its slipperiness. It has been given a value as shown in the table.

BRICK PAVEMENTS.

The author has considerable hesitation in saying anything about brick pavements in such a community as this, where these pavements have been used so long and to so great advantage, and where the engineers know so much more about the subject than does the author. While he has not had so much experience, especially in recent years, with this class of material, he has kept in touch with the subject and recognizes fully the value of this material and the importance it has in the paving industry.

The use of burnt clay for pavements seems to illustrate as well as anything else how quick the genius of the individual is to take advantage of every gift of nature. In this central portion of our country, where there is no natural paving stone, there has been provided a quality of clay which when properly treated produces a material that is almost as good as that provided by nature herself, if not better.

The first brick pavements in this country were laid in Wheeling, W. Va., in 1870, but the material did not come into general use for some time. The author is free to confess that when his attention was first called to brick as a paving material he was not favorably disposed, but the success of the material after it was put into use was such that he was obliged to confess the error of his previous judgment. Many failures have occurred in brick pavements because people did not understand the difference between bricks, and it was not easy in the early days of the industry to determine previous to its use whether a certain brick would or would not make a good pavement, and then it was not known what was the best method of laying. Both of these matters, however, have been fully threshed out, and, in the judgment of the author, this is mainly due to the work of the National Paving Brick Manufacturers' Association, which has maintained a paid secretary to look after the interests of the brick manufacturers, not simply to enable the manufacturers to sell their product, but that every city should get the best possible brick pavement obtainable.

With the present knowledge of the art of making brick and the methods of testing and laying, it is as possible to determine in advance what the results will be with brick as with any natural material.

It is conceded, of course, that brick pavements, like all others, must have a good foundation so that the question at issue, after the bricks themselves have been determined upon, is principally the cushion on the concrete and the character of the joint filling. The National Paving Brick Manufacturers' Association has always been very strong in advocating a 2-in. sand cushion laid upon the concrete. Most engineers in the East, however, believe that only 1 in. is necessary and that no more should be used, the idea being that all that is required is to have a sufficient quantity to allow the brick to be well bedded and have an even bearing over its whole surface. If, however, experience demonstrates that a 2-in. cushion is better than a 1-in., the extra expense is negligible and it should be adopted.

Three kinds of joint filling have been used: sand, coal tar pitch, and cement grout. At the present time sand is not used to any great extent, as it is conceded that a material should be used that will protect as much as possible the corners of the bricks so that the wear may be in accordance with the principle previously laid down in this paper—as nearly vertical as possible.

The question of coal tar pitch and cement grout cannot be so easily dismissed, however. The advocates of both materials are very many and present strong arguments, and it must be admitted that good results have been obtained with both kinds of filling. The author believes, however, that if proper care be taken in the laying and in the application of the filler, cement grout will give the best results, as it will come more nearly to making the pavement a monolith. Some trouble has occurred in the past with this filler on account of the rumbling of the pavement under traffic. This trouble, however, has been nearly if not entirely obviated. There is the objection to a cement filler, however, that it is more difficult to open a pavement and replace it in case it becomes necessary on account of the subsurface work.

It is almost impossible to set any figure for the life of a brick pavement or the cost of repairs, as these depend almost entirely upon traffic. The figures given in the table have been made out principally from data received from large cities, and undoubtedly they would be modified if obtained from the state of Ohio, for instance. But as has previously been said, all figures of the table must be adapted to local conditions. The results determined upon are shown in the table.

ASPHALT PAVEMENTS.

Under the head of asphalt will be considered sheet asphalt, asphalt blocks and bitulithic, although the latter is perhaps more often used with coal tar pitch than with asphalt.

Sheet Asphalt.—The first pavement of any note of this character was laid on Pennsylvania Ave., Washington, D. C., in 1876. So great was its success that it soon came into general use all over the country. While called asphalt pavement, it is almost entirely composed of sand, as the standard pavements have but 10 to 12 per cent of bitumen, which is the valuable property of the asphalt, the rest being made up of sand and a small portion of stone dust. The pavement is pleasing in appearance, smooth, not noisy, and on light traffic streets seems to be almost ideal. It is more slippery than the hard block pavements, and in the coast cities it is not generally laid on grades over 3 or 4 per cent. In the interior, however, where the atmosphere contains less moisture, it is often used on grades as high as 7 per cent without trouble.

Data collected from the cities of Brooklyn, Boston, Buffalo, Chicago, New York, Philadelphia, St. Louis and Washington shows that these cities in 1890 had a total of 246 miles of asphalt pavement, and in 1911 2,348 miles. This gives an idea of the popularity of the pavement, although it must be taken into consideration that this was during a period when there was great activity in laying new and smooth pavements. In Brooklyn, for instance, in 1895, there were 18 miles of asphalt pavement, while at the present time there are 540 miles. Brooklyn is a residential city, without many steep grades, and one to which this material is particularly adapted.

The cost of repairs to the asphalt pavements in the Borough of Brooklyn in 1913 was 3½ cts. per square yard, and in the Borough of Manhattan for 1912 it was 14.1 cts. per square yard, this being due to a great extent to the difference in traffic in the two cities. In the city of Paris in 1911 the cost was 19½ cts. per square yard.

In Berlin asphalt repairs are made by contract. The price paid is for streets from 5 to 20 years old, 10 cts. per square yard; from 20 to 30 years old, 12½ cts. per square yard; from 30 to 40 years old, 15 cts. per square yard.

In London the contract price on Cheapside per yard per year was 56 cts. for 15 years beginning after the pavement had been down 2 years. The average cost of repairs in London is 30 cts. per yard.

Asphalt Block.—This is another form of asphalt pavement and consists of blocks, made under heavy pressure at a central plant, composed of asphalt and broken stone aggregate rather than sand, as is used in the sheet pavement. The mixture of the material can be absolutely regulated and the pressure made uniform, so that the blocks produced should be uniform in density and quality. On account of the stone aggregate being coarser (say from ¼ in. downward) than the sand, the surface of the pavement affords a better foothold than the sheet asphalt, and also on account of the joints between the blocks. So that where a smooth pavement on grades is desired asphalt blocks are particularly desirable. They are used, of course, to a great extent on streets of light grades.

An objection to them is that as they have to be manufactured at one location the entire surface of the pavement must be transported from the plant to the street, while with sheet asphalt the plant can be located at a convenient point so that the haul is not so large. This makes a difference in the expense against the asphalt blocks. On the other hand, however, an asphalt block pavement can be repaired without the use of a mixing plant, as the blocks can be purchased and brought upon the street and used when desired.

Asphalt block pavements in the Borough of Manhattan in 1912 cost 9.8 cts. per square yard for repairs, and in the Borough of Brooklyn for 1913 1.2 cts. per square yard.

Bitulithic Pavements.—This pavement was first laid about 10 years ago. A gentleman who had formerly been interested in asphalt pavements conceived the idea of improving the then existing methods of laying a macadam pavement by filling a portion of the voids with a bituminous product or bitumen mixed with some other material. By the gradual elaboration of his original idea there was evolved a pavement which is now known as "bitulithic." It is essentially a macadam pavement of selected and graded stone, so that the voids in the stone shall be as small as possible, the binder being a bitumen, either coal tar or asphalt, both having been used. The pavement, being formed of coarse materials, can be laid on quite steep grades with satisfactory results. The writer has had no personal experience with this material, but has observed its construction and use in other cities. It has been laid very extensively in this country and would undoubtedly have been used to a greater extent if it were not patented. It is considered as standard and ranks with asphalt pavements.

TRAFFIC.

Knowing, however, the kind of material and the properties thereof are not sufficient for the official whose duty it is to determine the particular one to be used. He should also know the requirements of the streets to be paved. In order to do this he should have records of the kind and character of the traffic upon each street, or upon typical streets. Of course it is not necessary to get a total census of traffic on all residential streets, but of those where by an inspection it can be told to what class they belong.

And in speaking of traffic it should be understood that as at present considered the words "heavy," "medium," and "light" traffic mean very little, except with reference to any one particular city. There should be a standard unit of traffic, so that when the traffic on a certain street is given it could be distinctly comparable with traffic in another city. To do this it is necessary that the effect of traffic upon the different materials be known. Little attempt has been made to determine this, but within the last two or three years the English Road Board has constructed a machine for making this determination, and a somewhat similar machine was exhibited at the American Highways Association meeting in Detroit last fall.

It can be easily understood that a vehicle weighing with its load 15 tons will have an entirely different effect upon a pavement than 15 vehicles each weighing 1 ton. It makes a difference, too, whether the tires are steel or rubber, whether they are 1 in. or 3 ins. in width, and whether the vehicle is moving at a rate of 6 or 30 miles an hour. Experiments can be made so that the wear of the different vehicles under different loads can be ascertained and referred to one unit, and until this is done the adjectives "heavy," "medium," and "light" must be considered very indefinite.

I think on the streets that will be subjected to the heavy traffic, it will undoubtedly be necessary in the future, if motor trucks are to be continued with heavy loads as they undoubtely will, to lay heavier foundations on streets that will be subjected to that traffic. It may be necessary in the large cities to have certain streets assigned for those heavy trucks. I do not think it is fair to a city or the property owners to be compelled to lay pavements with a strength sufficient to hold up those heavy loads, although I was quite surprised in Glasgow in speaking to the engineer about their extreme loads when he said that even under the heaviest conditions they did not use more than six inches of concrete, but in London, on the heavy traffic streets they are using eight inches. If we are going to have these enormous loads, as we undoubtedly are, some special provisions must be made for them.

In connection with these loads, the city consulting engineers of New York had in mind for some time the regulating of these loads by ordinance, and having the people who use those excessive loads pay a license fee for their use. In making investigations, it was found that the large trucks with the wide tires sometimes had a load of 1,500 to 1,800 pounds per inch of tire on the streets, and it was the intention not only to regulate the total load, or load per wheel, but the load per inch of tire. An ordinance was drawn up to accomplish this, but it was finally decided that such an ordinance would be in the nature of a tax, and that these trucks were taxed once by the city, and that they could not be taxed again.

The borough engineer of Fulham, London, has established what he thinks is the wear that will take place on wood pavements under a certain traffic, and, having observed the traffic on any particular street, he figures out how long a wood pavement should last. This, however, is indefinite for the reasons before given.

And even after the value of the traffic unit has been established it will be difficult to apply it positively, as in every case the weight of the vehicles upon it must be estimated.

Then after all that has been determined there are certain local conditions which must also be taken into consideration. For instance, if the traffic requirements are such that a brick or stone pavement should be used for economic reasons, it is possible that hospitals, school houses or churches may be situated on certain portions of the streets, so that it would be necessary to lay wood on account of its noiseless property. Then, too, the official will learn that the wishes of the users of the street and those doing business on it must also be taken into consideration, and he often finds that the two will conflict, as the tradesman cares nothing about the noise and the business man cares little for tractive or non-slipper properties. So that, despite all information that can be obtained, in order to arrive at a satisfactory result the different conclusions must be treated together and intelligently. If, however, all these matters are taken into consideration, it is seldom that an improper determination will be made.

FUNDS FOR MAINTENANCE AND REPAIR.

It might be in order to discuss to a certain extent the economics of the different kinds of pavement. When an original pavement is paid for by assessment upon the abutting property, with repairs and repaving done by the city at large, it often becomes necessary to establish legally just when a street should be repaved. This is more easily determined by inspection in a block pavement than in a sheet pavement, as it can easily be seen when the blocks are worn out, but with a sheet pavement patching can be carried on for a long time and to a great extent without there being any formal repaving. Take for instance the case of an asphalt pavement, and assume for the sake of the illustration that the original pavement is paid for by a bond issue continued during the life of the pavement, which in this case is assumed at 18 years. The items of cost in the maintenance of this pavement on a street are: (1) First cost; (2) Interest on the bonds; (3) Annual repairs; (4) A sinking fund to be collected each year to pay for the bonds when they mature.

Assume that an asphalt pavement will cost $2 a square yard, that the interest on the bonds is 4 per cent, and that it will cost on an average 4 cts. per square yard per year for repairs. This can be shown in a formula, such as:

$$A + CI + \frac{R}{N} = \text{annual expense},$$

When A equals sinking fund charges, C equals first cost, I equals rate of interest, R the estimated cost of total repairs during the life of the pavement, and N the life of the proposed pavement.

Substituting these values in equation we have:

$$.078 + 2.08 + \frac{72}{18} = .198 \text{ for the first period};$$

—that is, the cost to be raised by the city every year to maintain this pavement would be 19.8 cts. per square yard. When, therefore, the annual repairs on a street approximate this amount the question of repaving should be carefully considered. If, however, the same pavement is continued upon the street, succeeding pavements would cost less, as the foundation must have a material value.

The following table shows the cost of different kinds of pavement for a period of years, assuming that granite has a life of 25 years, wood 20 years, brick 15 years, and [illegible] 18 years. This life, as has been intimated before, is probably too small for Ohio brick pavements or those in [illegible]

Material.	[illegible]	Expense per [illegible]	Average expense [illegible]
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]

The Application of the Merit System to the Appointment of Road Officials

[illegible] and maintenance is generally understood among engineers. This is especially true where extensive construction projects are involved. Practical difficulties [illegible], exist in the working out of any general plan that are intensified by well established customs and methods of administration in use in this country. In a paper before the [illegible] Road Congress, F. S. J. [illegible], assistant director of the United States Office of Public Roads, points out some of the difficulties involved in the question and suggested methods of surmounting these difficulties, which paper is given here in part:

Continuous employment must be made a basic feature of our road system if efficiency is to be obtained. It has often been said that no man can serve two masters, and it is inevitable that where such a condition exists the interests of one or the other will be sacrificed. Private enterprise requires good management in order to insure success. Public enterprise may be neglected as a rule with impunity, because the people in the aggregate do not look out for their own interests. Therefore, in any divided responsibilities the people will be the losers. That the roads require constant attention is a fact which has been so emphasized and so insisted upon by all men familiar with road construction and maintenance that it seems scarcely necessary to mention it in this connection. Only by continuous employment can such continuous attention be obtained for the road.

Not only must men be continuously employed, but they must receive adequate compensation. The penny-wise and pound-foolish policy of having money by employing a man who will give his services for the least compensation should be abandoned wherever it is practiced. An adequate scale of salaries and wages would attract to the work men who could effect economies and increase efficiency to such a degree as to save the outlay due to their extra compensation many times over.

As a requisite to efficient road management the men who actually construct and maintain roads should be appointed instead of elected, and that these should hold office during efficiency instead of for a fixed term. The old political idea that office is a reward and that the reward should be passed around so that many persons should receive it within a given period of years should be stamped out utterly and entirely. In its place should be accepted the principle of public welfare. If a certain work is to be done for the benefit of the public, for which the public is called upon to pay, it should be a primary consideration that the public pay only for value received. If the engineer or road superintendent renders to the county certain services which the county has agreed to purchase it should continue to purchase and to pay so long as the services are worth the purchase price. An engineer should not be called upon to neglect his legitimate duties in order to obtain sufficient popular support to enable him to remain in office, nor should he be called upon to leave one lucrative position to accept another, which is so limited as to term that he can have no guarantee of continuance in office, even though he renders efficient service.

Summing up the various points which I have put forward, I would say:

First: That our road problem is one of such magnitude as to demand adequate attention.

Second: That the chief cause of inefficient supervision is extreme localization.

Third: That this localization is based upon [illegible] precedent which has crystallized in our road legislation.

Fourth: That the remedy for localization lies in state control and that this state control should extend not merely to the mileage of roads improved with the aid of state funds but to practically the entire road mileage of the state.

[illegible] road administration must [illegible] and the best way [illegible] through the establishment [illegible] a competitive merit system of appointment for subordinate officials and employes.

Sixth: That county road administration should be based upon the same principle of appointment by merit through competitive qualification tests.

Seventh: That employment must be continuous in order to secure adequate results [illegible] with the qualifications of the appointees must be provided.

Ninth: That the men who have direct charge of construction and maintenance must be appointed rather than elected and must hold office during their efficiency instead of for fixed terms.

Comparative Cost of Horses and Traction Outfit in Street Maintenance.—In a paper before the California League of Municipalities T. R. Trotter compares the cost of street maintenance, using horses and a tractor outfit. In the matter of maintenance of streets a comparison of cost of work done by horses as against the same done by a tractor or roller is interesting. An assumed equipment of 5 wagons requires the use of 10 horses and 5 drivers, and at present cost of feed it can be shown that it costs for the upkeep of the horses $12 per month for feed and shoeing horses, about $4.60 each day for 10 horses. The wages of the drivers will not be less than $2 per day each, or $10 per day, or a total of $14.60, while if a tractor or roller should be used you will require one engine man with a helper at a cost of say $5 per day for the two men, adding $6 per day for fuel and oil, etc., with about $1 per day for depreciation, making in all $12 per day for the machine as against $14.60 per day with horses and men, thereby saving $2.60 per day, and therefore being in favor of the tractor work as against the horses. Your wagons will cost alike both as to original cost and maintenance, so may be disregarded. Teams will cost $400 each, or $2,000 for the five, while the cost of a tractor would be from $3,200 to $3,500, which will last as long as the horse and wagon outfit, being given equal care. For the sake of argument let us suppose the tractor outfit costs the same as horses in the matter of upkeep, it will, while in use, help in the maintenance of streets by use of broad tires, while your horse outfits, with heavy loads, will constantly break down your road surface under equal loads, and, what is of paramount importance, will not produce per day an equal yardage of work as compared with tractor outfit. Another important factor in the matter of reducing expense is that in the season of no work the horse outfit is still an item of cost, while the tractor machine, not requiring food for its life, is practically of no expense.

Street Accidents in New York.—The Automobile Journal states that the causes of some of the deaths from accidents in New York and the vicinity are as follows:

What Caused Some of the Deaths in 1913.

	New York City.	New York State.	New Jersey.
Motorcycles	67	190	96
No light on wagon	..	10	3
No light on automobile	..	18	..
U. S. mail	23	3	1
Taxicab	49	14	.
Fire department	26	5	4
Ambulance	24	2	2
Police and other officials	5	2	2
Children playing in street, etc.	46	12	18
Woman driving automobile	10	14	13

Fatalities in New York City.

	1910.	1911.	1912.	1913.
Automobile	112	142	221	302
Trolley	148	169	134	108
Wagon	211	172	177	170

Remainder of the State of New York.

	1911.	1912.	1913.
Automobile	172	127	110
Trolley	67	79	79
Wagon	31	28	32

State of New Jersey.

	1912.	1913.
Automobile	91	124
Trolley	41	28
Wagon	16	26

The Safety First Society of New York has several active committees, which are constantly making recommendations looking toward the adoption of regulations for greater safety to the public. One of the notable accomplishments of the society in New York has been the adoption of recomendations which were made to Police Commissioner Woods. These include: One way traffic streets. Closing of certain streets to vehicle traffic between the hours of 2 and 5 in the afternoon, so that they may be used by children for play purposes.

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Co-operation Among Engineers.

At various times and generally in widely separated sections of the country an engineer has undertaken the discussion of the "Status of the Civil Engineer"—although he may not have used exactly that title. Those who were present when such an address was delivered have usually agreed in the main with the speaker's remarks, and on rare occasions the address has created written discussion. In most cases, however, the discussion has been purely local and its effect on engineers in general exceedingly small. The subject has been discussed in one form or another at different times before the various local engineering societies throughout the country, yet the individual efforts have had little effect, due to the lack of co-operation between the various organizations. In connection with the above remarks certain statements made by Hunter McDonald, president of the American Society of Civil Engineers, in his address before the annual convention of the society, are of interest. We quote the following from this address, believing that the force of his remarks should carry beyond the membership of that society:

Our total membership in all grades is more than 7,400. I believe there are fully 40,000 men in the United States who are eligible for various grades of membership, in about the same proportion as the list is now made up. It is hardly safe to assume that the reason they are not members is because they cannot afford to be. I think it is safe to assume that, though they value the prestige of membership, they feel that the benefits are too remote. The engineer's calling generally imposes isolation, and yet they all dearly love companionship and exchange of ideas with one another. What we need and ought to have is the establishment of state organizations and of sections in all important cities. This matter has been studied by a special committee and discussed at one of our annual meetings, but, because we felt that our duty ended with the welfare of our own members, as provided in the Constitution, the project met with no encouragement. It is true that we permit local members in large cities to band together and call themselves Associations of Members of the Am. Soc. C. E., but they, as organizations, have no voice in our government, cannot mingle with other local organizations, and, except in very large cities where no other sections already exist, the formation of prosperous associations is impossible. The history of such local associations is by no means flattering to our pride or good management. The result of our narrow policy has been that organizations, embracing members of all the callings allied to civil engineering, have sprung up in nearly all cities of 100,000 inhabitants, and other national engineering societies have their own local sections established in many of these centers, and are co-operating with other local organizations. Some of these societies are by no means local in scope. In the independent local organizations the membership in many instances has not been of so high a standard as our own because of the necessity for a large membership in order to meet expenses, and, like the German and Canadian Societies, all classes are taken in. Nearly all these organizations contain some of our members who have joined, sometimes as a matter of civic pride and more often because they found at home that companionship which could not reach them through the medium of our Transactions from New York. All but a few of these local organizations are struggling for existence, and are unable to publish their papers and discussions. Some are prosperous, and one in Boston antedates our own organization by several years. It is the duty of this Society to extend a helping hand to all who need it, and to take the initiative in an effort to amalgamate gradually all engineering organizations into one homogeneous and co-operative mass, permitting each, as far as possible, to retain its name and identity. Some sacrifice on the part of all would no doubt be necessary. Union has not been possible among religious sects, but as I have heretofore shown, engineers are of necessity managers, and, with their spirit of fair play and good sense, ought to be able to get together. Call it what you will, the American Engineer Confederation, or better still, the American Confederation of Civil Engineers.

I would not lower our standard of requirement for membership, but, by extending local societies limited financial assistance, where needed, we can raise the general standard, and, by doing this, can elevate the profession at large so as to make a printed code of ethics unnecessary.

The time has come for the engineer to possess his own. The public is awake to the advantages of sound engineering advice. We have it in our power, without taking sides in political contests, to exert a beneficial influence in National, State and Municipal affairs.

The force of the above remarks will at once be realized, especially since they emanate from the president of a national society which, to quote, has "felt that our duty ended with the welfare of our own members." Membership in this society is, without doubt, highly prized, yet there are not a few who have joined because they felt that their *professional standing* demanded membership in the national society, rather than that they expected any *tangible benefit* from such membership. The latter class are of little value to any organization as they do not bring to the society that "human element" which is needed to vitalize it. By the term "tangible benefit" we mean something which is of greater value to any man than mere financial return, although it is every man's duty to look to his own interests.

It is undoubtedly true that engineers have missed great opportunities to increase the value and pleasure of their work through lack of co-operation. In any attempt to secure unity of interests and action much can be learned from a study of what architects have done, and are trying to do; that they are co-operating to a much greater extent than are engineers is especially evident to those engineers who have specialized in building construction. Unless engineers make organized effort to protect their interests and to further their aims they will soon find that others more alert have invaded the field which they long have considered to be their own.

The Traffic Limits of Pavements Used on Country Roads.

By traffic limit is meant, according to current usage of the term, the amount and type of traffic a pavement will carry without undue wear. The measures used in determining traffic vary, no one of them being universally employed. In England traffic is frequently measured in terms of tons per yard width of roadway passing a given point in a day or a year. In France the unit of measure is the "collar"; a "collar" meaning a single horse-drawn loaded vehicle, an automobile being estimated as equivalent to three "collars." The amount of traffic is stated in terms of 1,000 "collars" a day for the entire width of pavement. In the United States the tonnage per day for the whole width is the most commonly used unit, although the yard wide unit is also used. In this country also a distinction is made in the type of traffic—i. e., whether horse-drawn, motor truck, pleasure automobile, etc. Traffic limits are by no means well established, but results as analyzed in a recent report of the New York Highway Commission, given in part in the issue of ENGINEERING & CONTRACTING for October 28, 1914, are of interest as indicative of the development of adequate and useful limits of traffic for various conditions.

In England it appears to be a current opinion that if a waterbound macadam road does not last two years without resurfacing another type of construction should be used. It is believed that a traffic of about 137 tons per yard width per day, or 50,000 tons per annum, is the limit for this type of pavement. Traffic amounting to 1,200,000 tons per yard width in one year will wear out the pavement within the year. Another criterion is that ⅔ ct. per ton mile of cost of maintenance is the limit at which further use of a macadam surface becomes uneconomical.

Tar painting appears to double the life of a macadam pavement; in France the life is believed to be prolonged 50 per cent by such treatment of the surface. Tar painting is most effective with light, rapidly moving traffic; with heavy loads it adds little to the life of the road surface. The economic traffic limit on a daily basis as estimated in Germany is 400 single horse-drawn vehicles, 40 automobiles and 20 heavy motor trucks, which is equivalent to about 640 "collars," and is much less than English estimates of 200,000 tons per yard width per year.

Bituminous penetration macadam and bituminous concrete appear to have a traffic limit as high as 1,000 tons per yard width per day, under which traffic the surface would probably need renewing every four years. This traffic is from five to ten times that existing on ordinary heavy traffic roads in the United States. In England, in one locality, a bituminous concrete surface seven years old bears a traffic of 70,000 tons a yard width a year, and it is believed can be maintained indefinitely under present conditions for 12 cts. a square yard a year. One estimate places the life of this pavement at 11 years under a traffic of 120,000 tons per yard width per year. Greater uniformity of surface and lessened cost of maintenance are secured with the use of bituminous concrete than when other methods of construction are employed.

The above statistics are contradictory when analyzed in detail and cannot be accepted as of great value. They do, however, indicate a possible satisfactory method of determining traffic limits. Carefully conducted experiments should in the course of time furnish engineers with well-established limits for a limited number of traffic conditions, the utility of which is readily appreciated.

The Interrelation of Inadequate Funds, Poor Work and Damaged Reputations.

The attempt to carry on engineering work with inadequate funds usually ends in poor work, if not in disaster, and in the damaging of the engineer's reputation. This is especially true of small works for the reason that large works are seldom undertaken unless there is good reason for believing that ample funds are available or will be forthcoming. Moreover, certain influences, such as the desire to be philanthropic or to help those not quite able to help themselves, often tempt the engineer to permit his name to become connected with small engineering enterprises which are likely to come to grief through insufficiency of funds. Thus the municipal engineer sometimes finds his name linked with the public works engineering of villages which are unable to carry out his first recommendations. Through the influence of acquaintances and the desire to be as helpful as possible with the means at hand, he sometimes, under protest, modifies his original recommendations so as to make possible the installation of works with the funds available. If such works turn out badly, and that is the way they usually do turn out, his first recommendations and his protests at their modification are forgotten and the failure, partial or complete, is charged to him. He finds that his reputation has suffered, the city is dissatisfied with what it has, and every interest in the matter is a loser.

Many engineers have had, to their everlasting chagrin, experiences along the lines sketched above. In this issue we publish an interesting account of the connection of an engineer with an enterprise which illustrates the point here under discussion. This engineer was nominally responsible for the engineering work for a small dam, costing $3,500, which failed. The engineer reader will note, of course, that this particular engineer could not in fairness be held responsible for the failure since his plans were not followed. It is a matter of common experience, however, that such extenuating circumstances are seldom understood by the general public. The inevitable result in such cases is that the engineer's reputation suffers in the same proportion as the failure appeals to the popular mind. In this particular instance the harm done in this way was doubtless small, owing to the lack of spectacular features in connection with the failure of the dam. We hope no injustice has been done this engineer whom we unreservedly commend for the broad-minded spirit he has exhibited in giving the profession a frank account of his experience relative to the design, failure and reconstruction of the dam. We desire merely to call attention to a common result of attempting to carry on engineering work with inadequate funds.

It is interesting to note this engineer's first conclusion based on his experience of five years with the varying misfortunes of this ill-fated structure. His conclusion is: "Any construction where the funds available are very inadequate, is sure to be unsatisfactory to everybody concerned—owner, engineer, contractor, the innocent public." This conclusion is absolutely sound and engineers will do well to keep it strictly in mind.

Aside from the main conclusion quoted, it is also interesting to note that in the case of this dam an expenditure of $500 would have safeguarded an investment of $4,000 and would have made unnecessary the subsequent expenditure of a second comparatively large sum, probably another $4,000. Had there been state control of the design and construction of engineering works in Tennessee, this failure would doubtless have been prevented, since the uncompleted dam could not then have been placed in service.

It is also interesting to note that the author concludes that his recommendations might have been more faithfully followed if he had charged more for his services. It usually happens that the opinion which is cheaply obtained is lightly regarded. That is, from the engineer's standpoint, the most provoking aspect of such enterprises as we are here discussing. In addition to being blamed for what he really is not responsible for, he usually finds himself very poorly paid for his honest effort to combine philanthropy and engineering. The former is best carried on as a side line.

BUILDINGS

Structural Features of a Reinforced Concrete Combined Storage Warehouse and Office Building in Seattle, Wash.

(Staff Article.)

The Port Commission of the Port of Seattle, Wash., has recently let the contract for the construction of a combined office, warm and cold storage warehouse which possesses a number of interesting features. The building, which is located on the Seattle water front west of Railroad Ave., is of reinforced concrete construction, the columns being supported on circular piers resting on pile foun- [illegible] ins., and a width of 80 ft. 0 in. The plan of the building is a rectangle from which a triangle, having legs of 60 ft. and 62 ft. 11 ins., has been cut from one corner. The structure is surrounded by a concrete platform, the width of which, on the east, south and west sides, is 6 ft., while on the north side the width is increased to 9 ft. The north portion of the building, covering an area of 80 ft. × 103 ft. 7½ ins., constitutes the cold storage warehouse. The remainder of the structure is used for warm storage and office purposes. Although the roofs of the two sections of the buildings are at the same general elevation, the cold storage section contains a basement and five upper floors, while the warm storage section contains four stories with no basement. The basement of the cold storage section is 6 ft. below the first floor of the warm storage section. The first three floors of the latter are used for storage purpose, while the outside portions of the fourth floor are used for offices and the inside area for a large assembly hall and storerooms. Until recently the site was covered with water to an average depth of about 33 ft. The area has already been partially filled in, and the fill is to be completed before the construction of the building. The filling is made inside a bulkhead, which runs along three sides of the site and consists of two lines of concrete piles between concrete stop logs, these lines being spaced 18 ft. 6 ins. apart. The lines are tied together at each pile with rods, and the space between the lines is filled with earth. For a considerable part of its length the bulkhead carries a railroad track, which serves the warehouse

Extending around three sides of the warehouse there is to be constructed later an elevated roadway. One end of the steel girders for this roadway rests on brackets on the exterior columns of the warehouse. Along the west side of the warehouse the roadway is at the elevation of the second floor of the warm storage section (elevation + 24.0); along the east side it is at the elevation of the third floor of the warm storage section (elevation + 40.0), for about half the length of the warehouse, and then slopes downward until at the north end of the structure it is at about elevation + 28; along the north side the roadway slopes to meet the grades at the east and west sides.

MATERIALS, EQUIPMENT AND REQUIREMENTS.

Concrete.—The concrete used in the piers and in the superstructure is a 1:2:4 mix. It is specified that the fine aggregate shall be of such quality that mortar composed of 1 part Portland cement and 3 parts fine aggregate shall at least be equal in strength to mortar of the same consistency made with the same cement and standard Ottawa sand. The minimum allowable size of the coarse aggregate is ¼ in., and it must be evenly graded from the smallest to the largest particles. It is required that the concrete be mixed in a machine batch mixer continuously for at least five minutes to increase the waterproofing qualities of the concrete. The concrete walls

Fig. 1. Foundation Plan of Reinforced Concrete Storage Warehouse in Seattle, Wash. Drawing Also Shows Bulkhead, Dyke and Contour Lines of Original Bottom of Bay Before Site Was Filled.

are to be poured continuous with the columns, and new and old concrete must be joined by first clearing and roughening and then slushing the joint with a 1:2 cement mortar.

The concrete pavements and floors which are laid on the fill and are reinforced with wire mesh are to be laid in two layers, the second layer being placed before the first has set.

The finish for the first, second and third floors of the warm storage section consists of a ¾-in. layer of 1:2 cement mortar, in which is incorporated a metallic hardening compound furnished by the commission. Before applying the finish to the concrete floor slab the latter is to be cleaned with acid and washed; it is then to be slushed with neat cement and the top coat immediately applied. The ¾-in. finish is in addition to the thickness of floor slab called for on the drawings.

Cinder Concrete.—The cinder concrete used at stairway and elevator enclosures, shipping space of cold storage warehouse, toilet rooms and roof (where necessary to obtain required grades) is to be mixed in the proportions of 1 part Portland cement, 3 parts fine aggregate

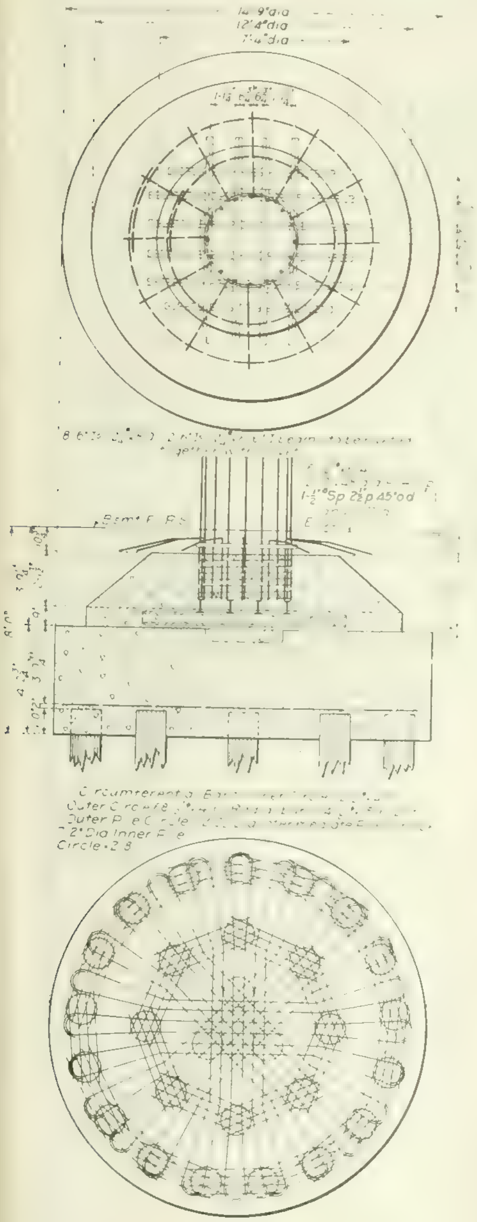


Fig. 3. Plans and Vertical Section of Typical 28-Pile Pier for Interior Column of Cold Storage Section of Warehouse.

and 6 parts cinders. In general, the thickness of the cinder concrete floor slabs is 5 ins.

Concrete Piles.—The reinforced concrete piles for the bulkhead walls (see Fig. 2) are to be reinforced as shown, the piles being thoroughly cured before driving. The concrete used in their construction is a 1:2:4 mix. The piles are to be driven with the aid of a water jet to the exact penetration necessary to bring their tops to the required elevation.

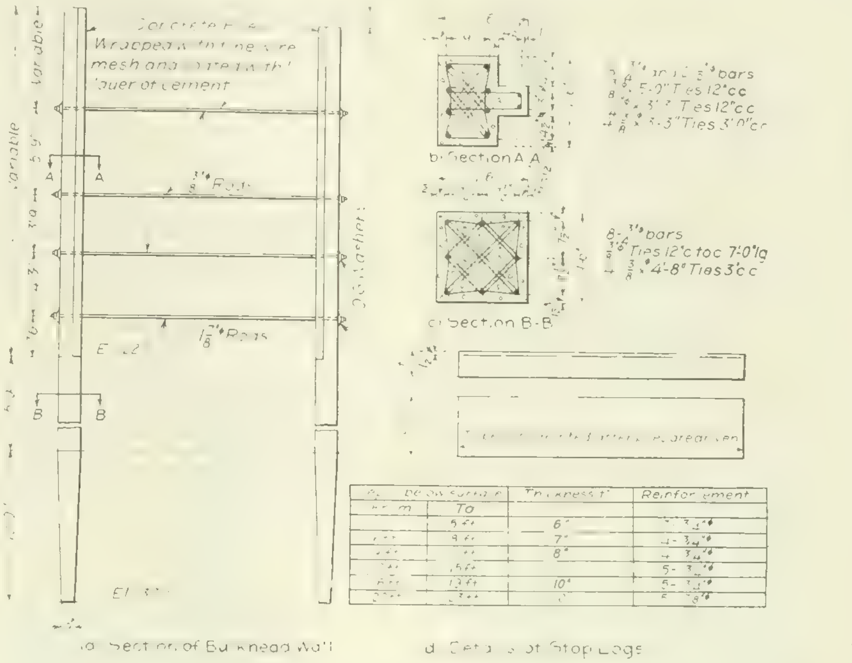


Fig. 2. Details of Reinforced Concrete Bulkhead Enclosing Site of Storage Warehouse in Seattle, Wash.

Wood Piles.—The wood piles used in the pier foundations are cut from sound, live Douglas fir trees of slow growth, having not less than six rings to the inch on any radius. The minimum diameter of any pile at the top shall be 9 ins. and at the butt 14 ins., after the bark is removed.

Tile Work.—Below elevation + 23 (which is about the elevation of the second floor of the warm storage section) the outside walls are concrete. Above this elevation (except cornices and fire walls) the outside walls are constructed of 8-in. scored "Denison" interlocking tiles. The enclosures around all stairways, elevators and machine rooms, also all partitions in corridors of cold storage warehouse and all partitions in first, second and third stories of warm storage warehouse, consist of 6-in. "Denison" interlocking tiles, except bearing partitions under stair landings, which are constructed of 8-in. tiles. The partitions around the toilet rooms on office, the floor and roof garden consist of 4-in. tiles.

In laying the tiles it is specified that the vertical joints shall be broken at every course and that the interlocking tiles shall be lapped vertically and horizontally in such a manner that there shall be no continuous mortar joints through the wall. The joints shall not be less than ⅝ in., the mortar for which shall be composed of 1 bbl. of lime, 2 sacks of Portland cement and ⅔ cu. yd. of clean bricklayers' sand.

Exterior Finish.—The finish for the exterior tile walls consists of a smooth plaster coat of 1:2 cement mortar, applied by means of a cement gun The thickness of the finish must be such as is required to bring the surface to a true plane, in no case less than ½ in. In order to insure an even surface the nozzleman must be followed up by two plasterers, who are to rod and float the work true and even. It is desired to avoid using hydrated lime in the gunite, but if a trial shows the engineer that it is necessary to do so, "Alca" hydrated lime may be added to the extent of 10 per cent of the volume of the cement, and, at his discretion, the engineer may permit the volume of sand to be increased to 3 parts.

The exterior concrete walls, except bell courses, are to have a "hammer finish," a six-point hammer being used.

Elevators.—The warm storage warehouse is equipped with three electric freight elevators, running from the first to the third floor, with a capacity of 15,000 lbs. each when operated at a speed of 50 ft. per minute; and one electric passenger elevator, running from the first floor to the roof garden, with a capacity of 2,700 lbs. when operated at a speed of 250 ft. per minute. The cold storage section is equipped with two electric freight elevators, running from the first to the fifth floor, with a capacity of 10,000 lbs. each when operated at a speed of 50 ft. per minute.

Scales.—There are to be installed nine railway depot scales, six 3-ton scales in the warm storage section, two 3-ton scales on the first

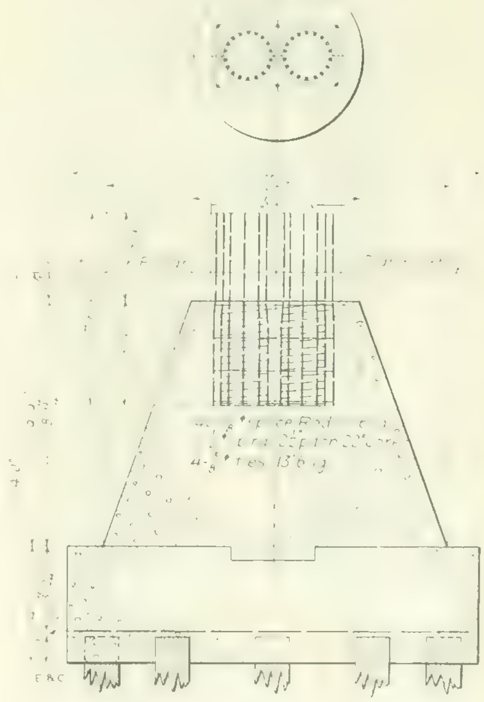


Fig. 4. Vertical Section and Top View of a 28-Pile Outside Pier for Warm Storage Section of Warehouse—Parts Not Shown Are Similar to Those Shown in Fig. 3.

floor of the cold storage section and one 6-ton scale on the third floor of this section.

Rolling Steel Doors.—The building is equipped with rolling steel doors as follows:

39 doors in the outside walls on the first floor of the cold and warm storage sections; 4 doors in the outside walls on the second

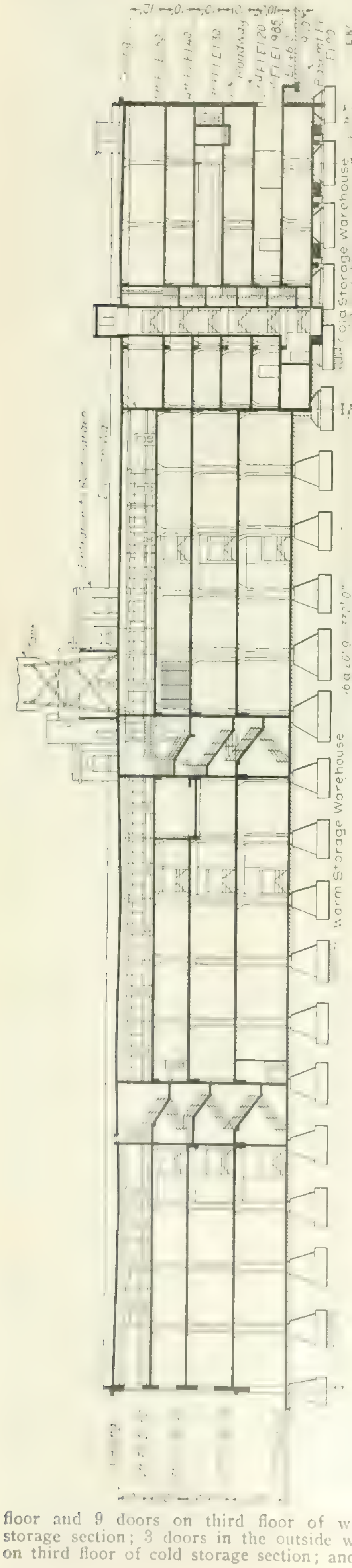


Fig. 5. Longitudinal Section of Warehouse Showing Story Heights of Warm and Cold Storage Sections and General Features.

floor and 9 doors on third floor of warm storage section; 3 doors in the outside walls on third floor of cold storage section; and 18 doors to the freight elevator enclosures in the warm storage section and 14 in the cold storage section. In general, the outside steel doors on the first floor are 12 ft. high by 14 ft. 9 ins. wide.

Heating System.—The building is heated by a low-pressure steam heating system, installed upon the overhead two-pipe basis of steam circulation. Oil is used as fuel and the oil-burning apparatus is electrically driven.

Cold Storage Plant.—The three ammonia compressors, the ammonia purifier and condenser, the brine coolers, the centrifugal pumps, etc., are located in the basement of the cold storage section of the warehouse. The five floors of this section are divided into 52 rooms of various sizes. These rooms are equipped with a sufficient number of coils to give the temperature desired, the range of temperature being from 10° F. to 30° F.

Lighting System.—In general, 100-watt "mazda" lights are installed throughout the cold and warm storage sections, although 40-watt lights are used at stairways, elevators, toilets, machinery rooms, boiler room, etc. The roof standards carry 150-watt lights.

Automatic Sprinkler System.—The building is equipped with an automatic sprinkler system, supplied from a 50,000-gal. steel tank carried on columns above the roof.

Local Products Favored.—It is stated in the specifications that preference shall be given to articles of local production or manufacture, quality and price being equal.

LOADS AND ALLOWABLE STRESSES.

Loads.—The various floor slabs and columns were designed for the live and dead loads given in Table I.

TABLE I.—LIVE AND DEAD LOADS USED IN DESIGNING WAREHOUSE.

Cold Storage Section.

Floor	Live load, lbs. per sq. ft	Dead load, lbs. per sq. ft.
Basement	600	135
First	250	110
Second	400	130
Third	250	110
Fourth	250	110
Fifth	250	110
Roof garden	125	100

Warm Storage Section.

Floor	Live load, lbs. per sq. ft.	Dead load, lbs. per sq. ft
First	On fill.	On fill.
Second	600	140
Third	500	135
Fourth	[illegible]	100
Roof garden	125	100

Allowable Stresses.—The floor slabs were designed under the [illegible] mushroom system, using the formulas developed by Dr. Eddy. The compression in the concrete was kept under 300 lbs. ber square inch and the tension in the reinforcing steel under 13,000 lbs. per square inch.

The columns were designed to conform with the building code of Seattle. The allowable compression in the concrete in the hooped columns was 500 lbs. per square inch, and in the columns having only vertical reinforcement, 350 lbs. per square inch. The permissible stress in all of the column reinforcement was 18,000 lbs. per square inch. All steel reinforcing bars were required to be plain, hard grade, conforming to the standard specifications of the American Society for Testing Materials, adopted June 1, 1912.

SUBSTRUCTURE DESIGN.

Bulkhead.—The bulkhead, which retains the deep fill at the site, bounds the property on three sides, as shown in Fig. 1. The site was originally covered with water, the original elevation of the bottom of the bay at this location being indicated by the contour lines in Fig. 1. The elevations refer to "city datum," which is 1 ft. below extreme high tide. The site has already been partially filled in, the elevation at the top of the fill at the outer face of the bulkhead being about —17.0. The fill is to be brought up to a sufficient height to enable the first floor of the warm storage section and the basement floor of the cold storage section to rest directly on it, the elevation

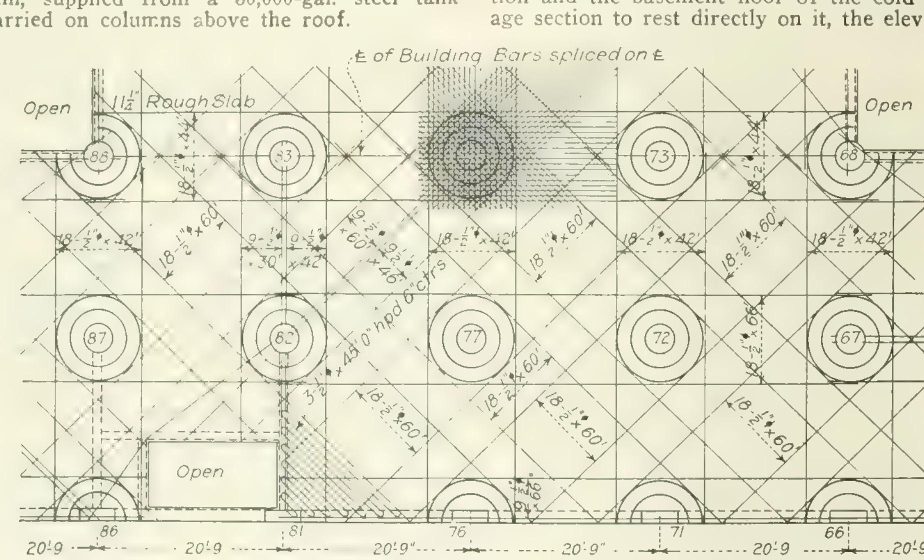


Fig. 6. Plan of Portion of Second Floor Slab of Warm Storage Section of Warehouse, Showing Size and Arrangement of Reinforcement.

of the top of the former being at + 6.0 and that of the latter, 0.0.

The bulkhead walls consist of reinforced concrete stop logs (extending from elevation —22.0 to the top of the bulkhead), between concrete pre-moulded piles, the spacing of the piles being 10 ft. 4½ ins. and the distance between the two lines of piles (out to out) 18 ft. 6 ins. Opposite piles in the two lines are tied together with four lines of rods extending through the piles, the bottom of the piles being driven to elevation — 37.0. The lowest tie rod is at elevation — 14.5 (or 7.5 ft. above the lowest line of stop logs), the next rod is at elevation — 10.25, the next at elevation — 6.5, and the top rod is at elevation — 0.75. The lowest rod has a diameter of 1⅞ in. and the others 1⅜ in. It is specified that these rods are to be wrapped with fine mesh wire and covered with a 1-in. layer of cement mortar. Figure 2 (a) shows a cross section of the bulkhead wall; Fig. 2 (b) shows a cross section of the upper part of a typical concrete pile, cut out to engage the end of the concrete stop logs; Fig. 2 (c) is a cross section of the lower part of a typical concrete pile, the section being taken below the stop logs; and Fig. 2 (d) gives the details of the stop logs.

The outside railroad track shown in Fig. 1 is already in place, being carried on piles. As shown in this drawing, a side track, to serve the storage warehouse, is to be constructed along the top of a portion of the bulkhead wall, the top of rail being at elevation + 2.17. As the main floors of the warm and cold storage sections are at elevations + 6.0 and + 9.83, respectively, and as the platform serving this side track is at elevation + 6.0, a retaining wall is constructed at the edge of the platform, its base being at the elevation of the inside line of piles of the bulkhead. The outside face of this wall is 6 ft. 6 ins. from the center line of the track and 4 ft. 6 ins. from the inner face of the inside line of bulkhead piles.

Foundation of Building.—After the bulkhead wall is completed and an earthen dyke is constructed along the easterly side of the site (see Fig. 1), it is the intention to pump the water from the enclosure and to construct the concrete piers for the columns in the dry. It is required that the fill must be brought up to elevation — 6.0 before the concrete foundation slab is poured over the piles.

The warehouse foundation consists of 104 circular concrete piers founded on green Douglas-fir piles (see Fig. 1). These piers vary in diameter from about 8 ft. to 17 ft., the number of piles in a pier varying from 9 to 38. It is specified that the minimum diameter of the piles at the top shall be 9 ins. and at the butt 14 ins. Their length must be sufficient to penetrate the fill and extend at least 15 ft. into the old bed of the bay, the original elevation of which is indicated by the contour lines in Fig. 1. It is proposed to drive the piles with the aid of a water jet to nearly the full depth, and then to seat them with the hammer until the recorded penetration at each blow indicates to the engineer that each pile will carry a safe load of 50,000 lbs.

The concrete piers are built in two sections—the footing and the pier shaft. To bond the two parts a hole 3 ft. square by 6 ins. deep is left in the footing, which engages a corresponding projection on the pier shaft. The piles project into the footings a distance of 1 ft., the footings being reinforced with rods, which lie in a plane 2 ins. above the tops of the piles. The bottoms of the pier shafts (except those having long conical shafts) are reinforced with I-beam grillages, the column reinforcement being extended to the top of these beams.

Figure 3 shows a vertical section and plans of a typical 28-pile pier for interior columns. The typical pier shown in this drawing is used for the interior columns under the cold storage section—Nos. 7 to 9, 12 to 14, 17 to 19 and 22 to 24 (see Fig. 1). The footings for these piers have a diameter of 14 ft. 9 ins. and a height of 4 ft. 2¾ ins.; while the pier shafts have a diameter at the bottom of 12 ft. 4 ins., a diameter at the top of 7 ft. 4 ins. and a height of 3 ft. 9¼ ins. The arrangement of the piles and the size and arrangement of the reinforcing bars and I-beams are clearly shown by the drawing.

The elevation of the bottom of the footings is the same for all piers. As the elevation of the first floor of the warm storage section (in which there is no basement) is 6 ft. above that of the basement of the cold storage section, the pier shafts under the former section are about 6 ft. longer than those under the latter. On this account no grillage beams are used in the pier shafts. Figure 4 shows a vertical section and a plan of the top of a 28-pile outside pier under the warm storage section, the pier being used under columns Nos. 46 and 51. The arrangement of the piles and of the reinforcement in the footing for piers Nos. 46 and 51 is similar to that shown in Fig. 3. The pier shafts of the interior piers under the warm storage section are similar to that shown in Fig. 4, although the reinforcement for the octagonal column forms a single circle.

The piers used for the remaining columns are similar to those shown in Figs. 3 and 4, although the size of the piers, the arrangement of the piles and of the reinforcement varies considerable, depending upon the number of piles required.

SUPERSTRUCTURE DESIGN.

General Features.—The warm and cold storage sections are separated by a concrete wall with cork insulation. The warm storage section has two stairways leading from the first to the fourth floor, these stairways being located along the south wall at about the third points of the building. The stairwells have plastered tile walls and are each 8 ft. × 20 ft. 2 ins. This section is served by three freight elevators having dimensions of 10 ft. by 8 ft. 6 ins., and one 6 × 6-ft. passenger elevator. The freight elevators are in the panels bordered by columns Nos. 88, 89, 94, 93; Nos. 68, 69, 64, 63; and Nos. 43, 44, 39, 38 (see Fig. 1), a corner of each elevator being adjacent to the first column given in each series. The passenger elevator is in the panel bordered by columns Nos. 52, 53, 58, 57, being adjacent to column No. 52. The cold storage section has two stairways with stairwells 12 ft. 10 ins. x 6 ft. 6 ins., one stairway being located in the northwest corner of the section and the other in the panel bordered by columns Nos. 16, 17, 22, 21, one corner of the first named stairwell being adjacent to column No. 5 and and a corner of the other adjacent to column No. 16. The cold storage section is served by two freight elevators, one having dimensions of 8 × 9 ft. and the other 6 ft. 9 ins. × 9 ft. These elevators are in the same panels as the stairways, the larger elevator being in the panel bounded by columns Nos. 16, 17, 22, 21.

Figure 5 shows a longitudinal section of the warehouse, the section being taken through the east bay. This drawing indicates the general construction feature and gives the story heights of both the warm and cold storage sections.

Floor Construction.—The flat slab, mushroom type of reinforced concrete floor is used, the only beams and girders used being those which frame the openings in the floors. Figure 6 shows a plan of a portion of the second floor of the warm storage section and gives the size and arrangement of the reinforcement in an interior panel and around openings. It will be noted that beams and girders are used only around openings in the floor. The typical columns have flaring octagonal heads, which extend on all sides 1 ft. beyond the face of the column.

The first floor of the warm storage section and the basement of the section used for cold storage rest directly upon the footings and the fill. These floors have a thickness of 12 ins., and are reinforced with No. 42 American Steel & Wire Co.'s triangular steel mesh, the edges of the reinforcements being lapped 4 ins. and the ends at least 12 ins.

The thicknesses of the upper floors of the warm storage section are: second floor, 11¼ ins.; third floor, 10¾ ins.; fourth floor, 7¾ ins.; and roof, variable to give the required slope. The thicknesses of the upper floor of the cold storage section are: first floor, 8¾ ins.; second floor, 10¼ ins.; third, fourth and fifth floors, 8¾ ins.; and roof, variable.

The arrangement of the reinforcement for the upper floors is indicated in Fig. 6. In placing the rods for the floor slabs, all those running from column to column directly on one side of a panel are placed first, those running at right angles to these are placed next, those in one diagonal belt next, and finally those in the other diagonal belt. Where a belt of slab rods runs parallel to a wall, a rod is first placed at the bottom on the forms, the normal and diagonal belts are next placed, and then the rods parallel to the wall are placed on top of the normal and diagonal belts. In

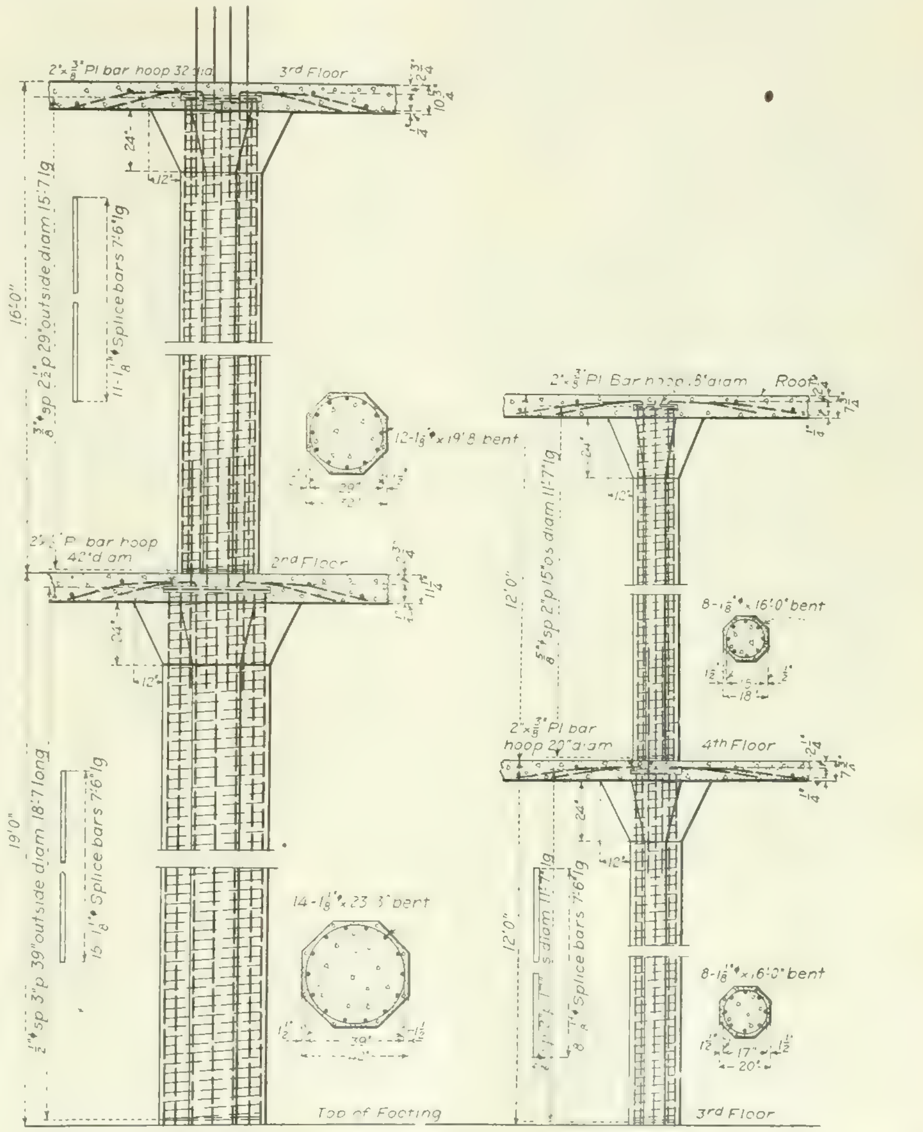


Fig. 7. Elevation and Cross Sections of Typical Interior Column in Warm Storage Section of Warehouse.

wiring the rods together (with No. 16 soft annealed wire) it is proposed to take a piece about 3 ft. long and first fasten an intersection, then to carry the wire diagonally to the next intersection (taking a half hitch) and then ins. The typical interior columns for the cold storage section have short diameter, as follows: basement story, 48 ins.; first story, 42 ins.; second story, 36 ins.; third story, 30 ins.; fourth story, 24 ins.; and fifth story, 18 ins.

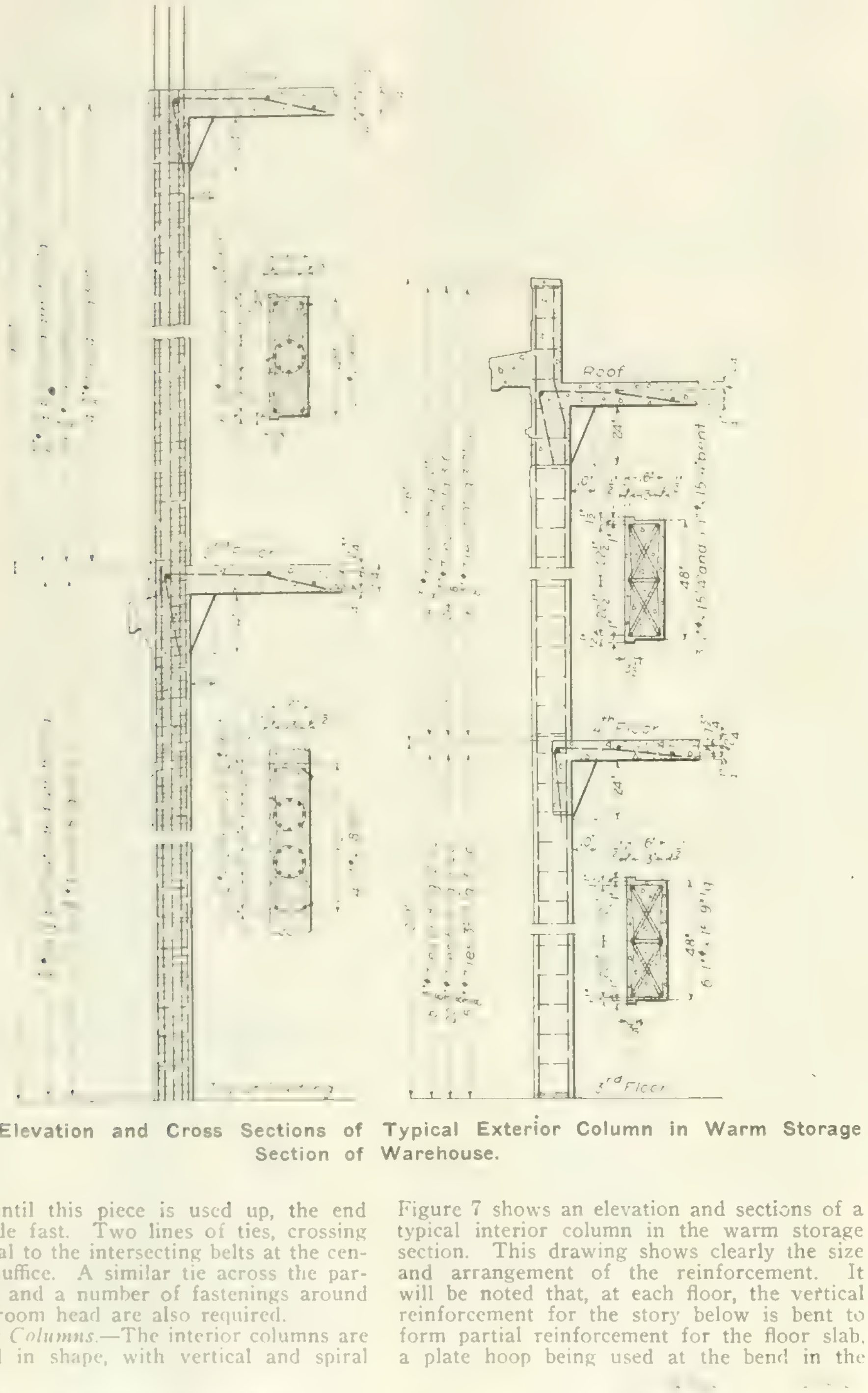


Fig. 8.—Elevation and Cross Sections of Typical Exterior Column in Warm Storage Section of Warehouse.

proceed until this piece is used up, the end being made fast. Two lines of ties, crossing and normal to the intersecting belts at the center, will suffice. A similar tie across the parallel belts and a number of fastenings around the mushroom head are also required.

Interior Columns.—The interior columns are hexagonal in shape, with vertical and spiral reinforcement, the latter being wound by machine. The typical interior columns for the warm storage section have short diameters, as follows: first story, 42 ins.; second story, 32 ins.; third story, 20 ins.; and fourth story, 18

Figure 7 shows an elevation and sections of a typical interior column in the warm storage section. This drawing shows clearly the size and arrangement of the reinforcement. It will be noted that, at each floor, the vertical reinforcement for the story below is bent to form partial reinforcement for the floor slab, a plate hoop being used at the bend in the bars. Columns Nos. 47 and 52, which support the legs of the water tower, have short diameters of 50 ins., from first floor to roof. The type of interior column used in the cold storage section is similar to that shown in Fig. 7.

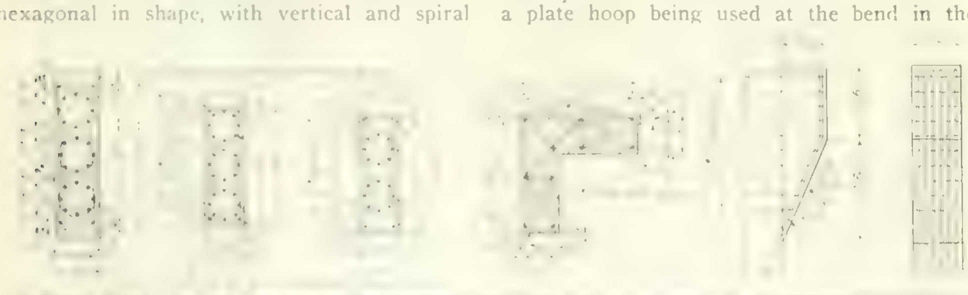


Fig. 9. Cross Sections of Typical Exterior Columns in Lower Stories of Cold Storage Section of Warehouse; [illegible] Section of Corner Columns and Details of Exterior Column Bracket.

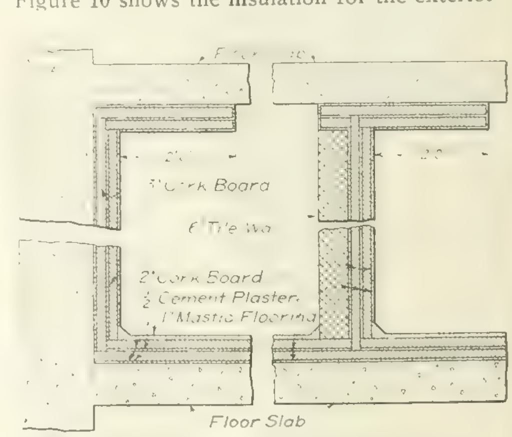
The columns which are in the 8-in. concrete division wall between the warm and cold storage sections are of similar construction, except that the half capitals at the floors of the two sections are generally at different elevations, while the first floor of the warm storage section is independent of these columns.

Exterior Columns.—The exterior columns of the warm storage section have the same thickness as the concrete wall in the first story, namely, 16 ins. These columns are rectangular in section and have a width of 60 ins. in the first story and 48 ins. in the upper stories. Figure 8 shows an elevation and cross sections of a typical exterior column in the warm storage warehouse. These drawings show clearly the type of construction and the size and arrangement of the reinforcement. It will be noted that the two lower stories have spiral cores.

The exterior columns in the basement story of the cold storage section have a thickness of 18 ins., while those in the upper stories have a thickness of 16 ins. Figure 9 (a) shows a cross section of a typical exterior column in the basement story, the column thickness being the same as that of the wall; and Figs. 9 (b) and (c) show cross sections of the column in the first and second stories, respectively. The column section above the third floor is the same as that shown for the third story column in Fig. 8. Figure 9 (d) shows a cross section of the L-shaped corner columns, and Fig. 9 (e) gives details of the bracket on the exterior columns for the elevated roadway girders.

Exterior Walls.—The exterior walls below the second floor of the warm storage warehouse are of concrete, 16 ins. thick; those above this floor are 12 ins. thick and are constructed of interlocking tiles, with a cement gun finish. In the cold storage section the exterior walls below elevation + 23.0 (3 ft. above the second floor) are of concrete, the thickness in the basement story being 18 ins. and for the remaining height 16 ins. Above elevation + 23.0 the walls are built of tiles and are 12 ins. thick.

In the cold storage section the outside concrete wall is insulated with a 3-in. and a 2-in. layer of cork board laid in ½-in. cement plaster. The tops of the floor slabs are insulated with two 2-in. layers of cork board laid in cement plaster, with a 1-in. mastic wearing surface. The bottoms of the floor slabs are also insulated, for a distance of 2 ft. 6½ ins. from the inside of the walls, with two 2-in. layers of cork board. The 6-in. tile partition walls between the numerous rooms of the cold storage section are also insulated. Figure 10 shows the insulation for the exterior



(a) Outside Concrete Wall (b) Interior Tile Wall

Fig. 10. Type of Insulation Used for Exterior Concrete and Interior Tile Walls of Cold Storage Section of Warehouse in Seattle, Wash.

walls, floors and interior walls. The outside tile walls are insulated in the same manner as the concrete walls.

The contract price for this warehouse was $299,000. The contract required that work be started June 15, 1914, and that the structure be completed in 245 days.

PERSONNEL.

The warehouse was designed by A. O. Powell, consulting engineer. The erection is under the direction of Paul P. Whitham, chief engineer of the Port of Seattle. The Pearson Construction Co. has the contract for the construction of the warehouse.

Some Data on the Proposed State Building Code for Illinois.

At the 47th General Assembly of the Illinois Legislature a resolution was adopted authorizing Governor Deneen to appoint a commission to prepare for submission to the Legislature a broad, comprehensive state building code. This commission, which consisted of N. Clifford Ricker and Ira O. Baker of Urbana, George J. Jobst of Peoria, and William S. Stahl, Richard E. Schmidt, William C. Armstrong and William H. Merrill of Chicago, immediately upon its appointment organized and proceeded to collect the necessary data to enable it to perform its work. The volume of work required, however, was found to be so great that the commission was unable to complete its work in time for submission to the last session of the Legislature. Although it was not possible to secure an appropriation to complete the work, the commission concluded to continue its labors and to make its final report at the next session of the Legislature in January. The following data on the commission's report, which is now in the hands of the printer, were taken from a paper by F. E. Davidson, presented before the Illinois State Convention of Licensed Architects.

In general, the administrative features of the proposed act are as follows:

A state "Department of Buildings" is to be created, to be under the control of a state "Building Commissioner" appointed by the Governor. The proposed state building code is to be enforced by the state building commissioner only where villages, towns and cities do not have a comprehensive building code, providing that none of the provisions of the building codes of the municipalities having such a department is in conflict with the broad provisions of the state code. The proposed law provides that applications to the local or state building commissioner for permits for the construction of buildings, or for changes or alterations of any building, shall be accompanied by a copy of the plans and specifications, and by a certificate signed by the architect who executed them, certifying that such plans and specifications are in accordance with the requirements of this act. The act also provides for a "Board of Arbitration," to be composed of three members, to pass upon the use of special devices, materials or types of construction not specifically approved by the act, one arbiter to be appointed by the state building commissioner, one by the architect, engineer, contractor or joint owner of the building, who may appeal from the ruling of the state building commissioner, and the two to select a third member. The act also provides that the expense of the arbitration shall be borne by the applicant, and that the decision of the arbiters shall be final and binding in regard to the question involved.

One of the important provisions of the administrative features of the act is that all rulings of the state building commissioner and all decisions of the boards of arbitration therein provided for shall be made a matter of record in the office of the commissioner, and that bulletins of the rulings of the state building commissioner on these questions, and likewise of the decisions of the boards of arbitration provided for therein, shall be placed on file in the office of the commissioner within 5 days after decision is made. The act also provides that these bulletins shall on request be mailed to architects, engineers, builders, contractors and other persons.

In a general way the building code is to be applicable, and is to be enforced, throughout the entire state of Illinois by the state building commissioner, except in villages, towns and cities where there is a building commissioner devoting his entire time to the duties of his office, and where such municipalities have in effect a comprehensive building code, in which event it is the duty of the officers of the municipality to enforce the provisions of the state code. The act specifically makes it the joint and several duties of the Attorney General, the State's Attorney and the City Attorney in towns, cities and villages having building laws and a building commissioner to commence and enforce prosecutions for violations of this act. The act provides a penalty of not less than $10 nor more than $200 for each offense of an architect, engineer, builder or contractor, or any superintendent of construction for an owner, architect, builder or contractor who shall design or make drawings or plans for the construction of any structure in violation of any of the requirements of this act, or anyone who shall employ or authorize another to design a building or other structure in violation of any of the provisions of this act, or who shall consent to any such violation.

The present "Architect's License Law" was carefully considered by the commission, and the proposed code does not conflict with any of the provisions of this law. The proposed code permits any person, mechanic or builder, to make plans and specifications and to supervise the construction, alteration or enlargement of any building that is to be constructed by himself or his employes, with the further provision, however, that the person, mechanic or builder making the plans and specifications shall furnish to the local or state building commissioner a certificate certifying the same to be in compliance with the requirements of the act.

The act is retroactive only as to doors, passageways, stairways, halls, corridors, exits or fire escapes of any school building, public hall, church, theater, department store, hotel, apartment house or other building or structure occupied or frequented by a large number of people; and the act provides that it shall be the duty of the state building commissioner on receipt of information that the doors, passageways, stairways, halls, corridors, exits or fire escapes of any school building, public hall, church, theater, department store, hotel, apartment house or other building or structure occupied or frequented by a large number of people are not intact, or properly located and maintained, and are not in accordance with this act, to cause an examination to be made of such building or buildings, and to require such changes or alterations to be made as are necessary to render said building or buildings safe for the public, and to afford sufficient means of exit and proper protection in case of fire or panic, and in case a notice is served upon the owner or occupant of any building deemed unsafe by the state building commissioner, authority is given under the act immediately to close said building.

The proposed code as prepared by the commissioner cannot be considered as a textbook on building construction, the commission confining itself more particularly to the broad principles of safe and sanitary construction rather than going into minute details of building construction itself. An examination of the proposed code shows that its requirements as to protected stairways, elevator shafts and other means of exit in all classes of buildings are more rigid than that of any code now in existence in the United States, and, while its adoption will not increase the cost of construction in Chicago or the larger cities, it will result in safer and more sanitary buildings throughout the state, and as a result it will increase somewhat the cost of certain classes of buildings outside of the larger cities.

WATER WORKS

Construction of Water Works Tunnels In the Metropolitan Water District of Massachusetts.

IV.

Design Features of Water Pipe Tunnel Under Chelsea Creek Between East Boston and Chelsea, Mass.

(Staff Article.)

The present article concludes the series pertaining to the construction of water tunnels in the Metropolitan Water District of Massachusetts. The first three articles, contributed by Mr. William E. Foss, assistant to the chief engineer of the Metropolitan Water and Sewerage Board, were published in our issues of July 22, Aug. 5, and Oct. 14, 1914, respectively. The former articles pertained largely to construction methods and costs, and illustrated certain features of design. This article relates largely to the design of a water pipe tunnel now under construction and gives a few notes on construction.

The sectional profile of the tunnel and its appurtenant structures is shown in Fig. 1. The bulkhead near the Chelsea shore is shown in a detailed section in Fig. 2. The design of the pile guards to protect the shafts is shown in Fig. 3.

The work is being done by the pneumatic process, under contract. The tunnel is similar to the one constructed under the same stream by day labor in 1910. The water pipe in this tunnel is 42 ins. in diameter, with special bell and spigot ends designed for outside and inside caulking with lead wool. The design of this joint is shown in Fig. 4.

This tunnel is designed to replace two lines of 24 in. flexible jointed submerged water pipes now laid in the bed of the creek. These pipes are to be removed to allow for deepening the ship channel. One of these pipe lines was laid in 1870 with a joint that is flexible in the vertical plane only. As this joint is not like any of the flexible joints described in Mr. Emil Kuichling's article on "Flexible Joints for Submerged Water Pipe Lines," published in ENGINEERING AND CONTRACTING of April 15, 1914, the details of its construction are here illustrated in Fig. 5.

Briefly, the work done under the entire contract consists of constructing by the pneumatic process under Chelsea Creek, between East Boston and Chelsea, a water pipe tunnel about 640 ft. in length, with a 42-in. cast-iron water pipe lining, a brick shell 8 ins. thick and about 8½ ft. in exterior diameter, with concrete masonry filling between the pipe and the shell; of removing two submerged lines of 24-in. cast-iron pipe for a distance of 770 ft.; of placing 7,000 cu. yds. of filling on State property on the Chelsea shore; of laying about 300 ft. of 36-in. and 24-in. pipe line to connect the tunnel with the existing pipe lines, and 240 ft. of 12-in. and 8-in. pipe line for blow-offs. The work also includes the furnishing and placing of the piles and timber required for constructing guards around the two shafts, and the bulkhead to support the filling on the Chelsea shore; for extending the existing fender guard on the East Boston side of the channel; for constructing the foundations for the 36-in. pipe lines; and for constructing a plank walk about 450 ft. in length along the existing fender guard and around the shaft on the East Boston side of the channel. While the work is going on the water supply to East Boston is maintained proper locations. Steel rods 1 in. in diameter, spaced 1½ ft. on centers, were built into the brick shells of the shafts to serve as emergency ladders in case of accident to the hoist. Steel rods ¾ in. in diameter were used for The 42-in. water pipes are thoroughly cleaned before laying, and the pipe interior is to be kept clean until the work is accepted. Pipe is laid accurately to line and grade and is held in position by suitable temporary

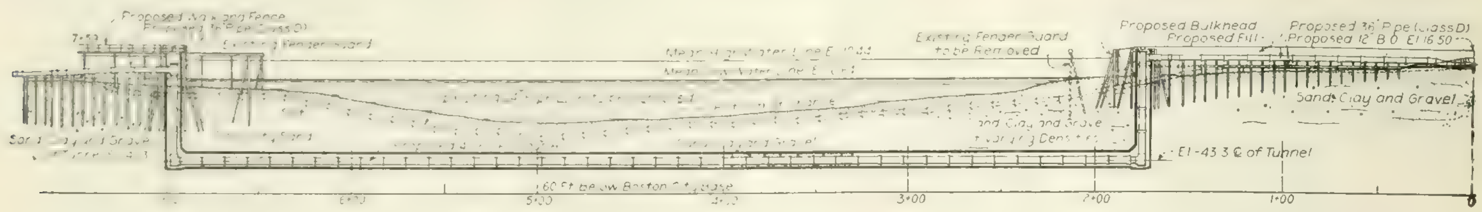


Fig. 1. Sectional Profile of Water Pipe Tunnel Under Chelsea Creek, Between East Boston and Chelsea, Mass.

through the easterly or upstream existing 24-in. pipe line.

Piling and Lumber.—Spruce piles were used for the pipe line foundation and for the timber guard for the tunnel shaft on the Chelsea shore of the creek. Oak piles were used for the bulkhead on the Chelsea shore of the creek, and for the pipe line foundation, the timber guard for the tunnel shaft, and for extending the existing fender guard, on the East Boston shore. The existing fender guard on the Chelsea shore was removed. All spur shores or batter piles were driven to the batter specified, and the inclination of the piles was obtained from driving and not from pulling the head of the pile. The spruce piles have a minimum diameter of 10 ins. at the butt after driven and cut off, and 6 ins. at the tip. The oak piles have a minimum diameter of 14 ins. outside the bark at a point 4 ft. from the butt, and 6 ins. inside the bark at the tip.

Long leaf yellow pine of "prime" quality was used for foundations for the 36-in. pipe line, for the bulkhead, for the shaft guards, for extending the fender, and for constructing the walks and fences on the East Boston shore. Tongued and grooved sheet piling was used in the bulkhead. This piling is fitted closely to the sea walls at the ends of the bulkheads to retain the filling placed back of it.

THE TUNNEL.

As stated previously the tunnel is being constructed by the pneumatic process. It is electrically lighted during construction. Telephone service is maintained between the tunnel and the field office above. A lock tender was on duty at each lock for the entire time the work was in progress under compressed air.

The upper portions of the shafts are constructed inside the steel casings previously mentioned, and illustrated in Fig. 6, and were bonding the brick masonry of the shaft below the bottom of the steel casing with the masonry above.

The excavation for the tunnel was sufficient in diameter to allow for building the 8-in. brick shell at least 7 ft. 2 ins. in inside diameter. The material removed was used for making the fill on the State property previously mentioned.

The brick masonry was laid up with the best quality of brick laid in 1:2 mortar. The brick shell is 8 ins. thick. When laid the masonry is plastered on the inside with a ½-in. coat of 1 to 1 mortar, which is later given a wash of neat cement grout. After the completion of the brick shell and before removing the air pressure from the tunnel, a second coat of mortar and a second wash is specified. All visible leaks are then to be caulked with lead wool. After the air pressure is removed the brick shell is to be kept substantially watertight until the concrete masonry is in place.

The concrete materials are mixed in the blocking and wedges until the joints have been made and the concrete placed. In the shafts, where the pipes are set in a vertical position, each spigot is supported on three small steel blocks 1 in. long by ¾ in. wide by ½ in. high placed in the socket of the pipe already laid, so as to insure a ½ in. joint opening.

The spigots are so adjusted in the sockets as to leave a joint opening of ½ in. between the ends of the pipes and the sockets, and a uniform space between the outside of the pipes and the inside of the sockets. The joints are made with lead wool and jute packing as shown in Fig. 4. The lead wool is manufactured from best grade of new desilverized lead, and has long fibres not exceeding .009 in. in diameter. It is furnished in rope form, wound upon substantial reels, with the layers separated by paper. Each reel contains approximately 150 ft. of ⅝-in. rope, and has printed instructions regarding the care and handling of the lead wool tacked upon it, and is protected with a waterproof covering. The joints are thoroughly caulked, both

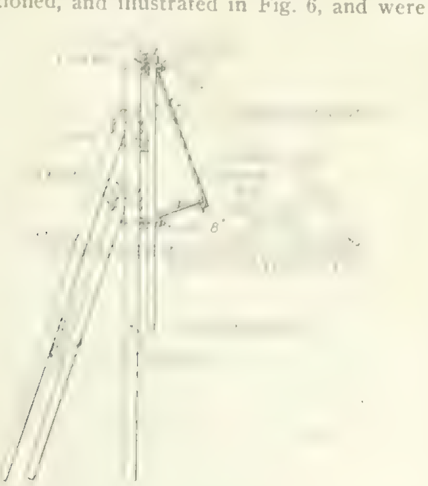


Fig. 2. Section of Bulkhead Near Chelsea End of Chelsea Creek Water Tunnel.

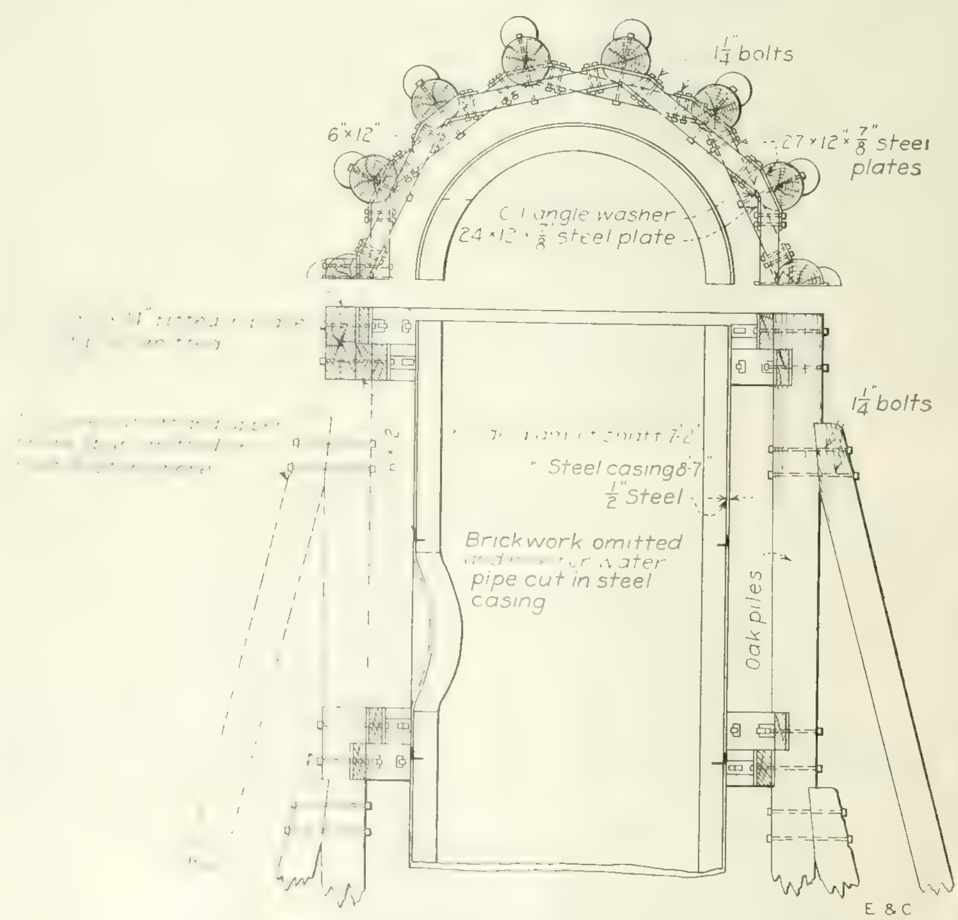


Fig. 3. Plan and Section of Pile Guards to Protect Shafts of Chelsea Creek Water Tunnel.

sunk into place within the shaft guards. Temporary weight was added when necessary to sink the tops of the shafts to the required elevation. Holes were cut in the steel casings for the 42 and 36-in. reducers in the proportion of 376 lbs. of the best American Portland cement, 9 cu. ft. of loosely compacted sand, and 15 cu. ft. of loosely compacted stone ranging from ¼ in. to 1½ ins. in greatest dimension.

on the inside and outside of the pipe, and made watertight. The final caulking of the joints on the inside of the pipe line will be done after the pipes are all laid and the concrete has been placed.

A hydrostatic test pressure of 200 lbs. per square inch is specified for the completed work. The other pipe lines in the contract are to be caulked with jute and lead wool also. Wooden insulating joints composed of a wooden ring in the socket of the pipe, to prevent metallic contact of the spigot end of the adjoining pipe, and wooden staves in place of the lead and jute of the regular joints, will be located at two points in the pipe line.

NOTES ON CONSTRUCTION.

The construction methods employed on this tunnel are substantially the same as on the former Chelsea Creek Tunnel, as described

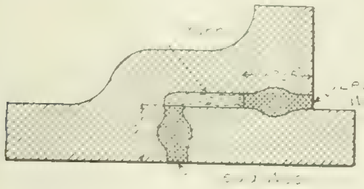


Fig. 4. Section of 42-in. Pipe Joint, Caulked With Lead Wool, in Chelsea Creek Water Tunnel.

in ENGINEERING AND CONTRACTING of Aug. 5, 1914, with the exception that the work is being done by contract instead of by day labor, and that the location of the shafts in water 12 to 15 ft. deep at high tide requires the use of a lighter for placing the upper portion of the shafts as shown in Fig. 6. Cast lead joints were used in the former tunnel instead of the lead wool joints now being employed.

The compressor plant installed has a capacity of about 1,700 cu. ft. of free air per minute, and includes one straight line and one duplex compressor, and one upright and one horizontal steam boiler.

The Chelsea shaft was sunk to grade using the small vertical air lock shown on top of the right hand cylinder in Fig. 6. A larger horizontal air lock was then set in the drift just beyond the shaft so that the loaded tunnel cars of muck could be passed through and hoisted directly to the surface from the shaft after the upper lock was removed.

Steel tunnel plates were used to support the roof of the drift, which was a clay hardpan, until the brick lining was built. The

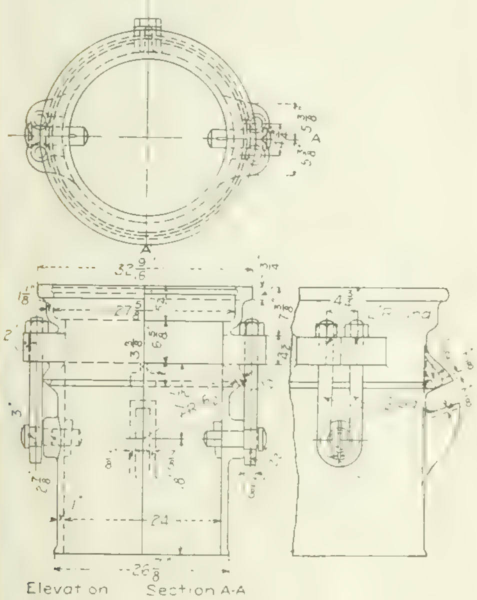


Fig. 5. Details of 24-in. Joint, Flexible in Vertical Plane Only, Used in Pipe Line Under Chelsea Creek in 1870.

work was carried on continuously with three shifts of eight hours each.

The air pressure carried was about 25 lbs. per square inch at high tide, or just sufficient to balance the static head of the tide. While sinking the shaft a spring of surface water was encountered in a gravel vein and it was necessary to carry the air pressure several pounds above the tide water pressure until the vein was clayed up and the brick lining completed at that place.

ACKNOWLEDGMENT.

Mr. Dexter Brackett is chief engineer of the works here described and Mr. William E. Foss is his first assistant. We are indebted to Mr. Foss for the information here given.

Fire Alarm Attachment for Recording Pressure Gage as Employed at Ripon, Wis.

The amount of the water pressure at the time of a fire is a matter often in dispute between fire and water interests. Fire department officials are prone, at times, to charge that poor water pressure accounts for their failure to check fires promptly. Very likely this is true in some instances, but that it is sometimes brought forward as a plausible sounding excuse for fire department failures is also true. This seems especially likely to happen where the interests of a fire department clash with those of a privately owned water works. So frequently has such a charge been made by fire department employes that in recent times water department officials have taken steps to safeguard themselves from these charges. Usually this is done by the use of pressure recording gages located at various points in the water works distribution systems. An interesting variation in this scheme of self-defense by water interests is here described, the information being taken from a paper by Mr. W. E. Haseltine in the Journal of the American Water Works Association for September, 1914.

Several years ago the Ripon Light and Water Co. got into a serious controversy with the officials of the city of Ripon owing to a claim of inadequate pressure at a fire. Fortunately, the company had in service a recording pressure gage by which it was able to prove to the satisfaction of experts engaged by the city to make an investigation, that the fault did not lie with the company's pressure but with the length of run of hose and the attachments made by the fire department.

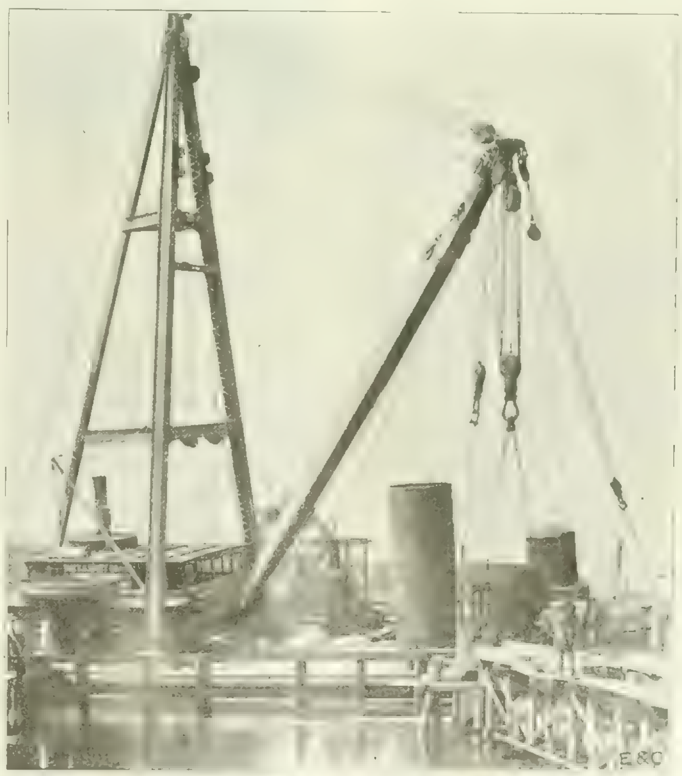
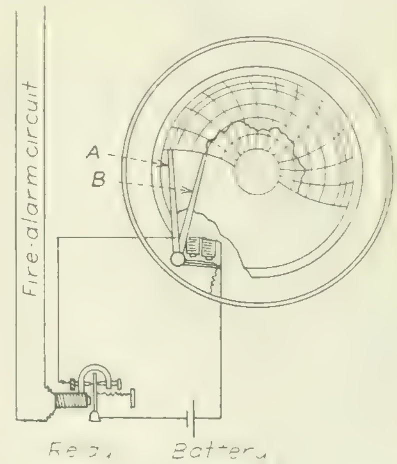


Fig. 6. View Showing the Placing of the Steel Encased Section of Chelsea Shaft for the Chelsea Creek Water Tunnel, June, 1914.

This decision created so much hard feeling among the city and fire department officials that the company was subjected to a series of petty persecutions in the way of so-called tests and complaints with regard to the length



Fire Alarm Attachment for Recording Pressure Gage.

A, Fire Alarm Hand; B. Pressure Gage Hand.

of time it took for the Company to furnish fire pressure after an alarm was turned in.

The Ripon Light and Water Co. furnishes domestic service by pumping into a standpipe, and fire service by shutting off the standpipe and pumping directly into the mains, and it seemed most desirable to have some method of proving beyond question how long it took to furnish this direct pressure after an alarm was turned in. For this reason the attachment here described was devised and put into service, and its operation as well as its effect upon the Company's relations with the city have been most satisfactory.

As will be seen in the cut, it consists merely of an auxiliary hand "A," turning on a [illegible] of the pressure gage and adjusted to mark coincidentally therewith. Under normal conditions this hand draws a perfect circle near the outer edge of the chart, but when an alarm of fire is turned in, a small electro magnet operating upon the armature attached to this hand jerks it to one side, making a mark at right angles to the regular curve, indicating indelibly upon the chart the exact instant at which this alarm was received. By observing the chart it is possible, without question, to determine what the pressure was at the moment the fire alarm was turned in and how long thereafter direct pressure was furnished.

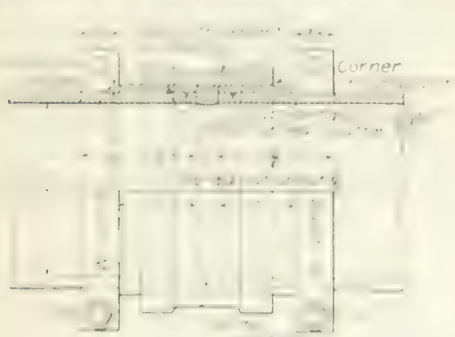
The connection with the fire alarm system is made through a regular telegraph relay, the magnet on the gage being operated by a single dry cell. The accompanying diagram indicates the position of the chart upon which the time of turning in the fire alarm is recorded, the pressure furnished, the exact pressure at the time the alarm was turned in, the time direct pressure is furnished and the amount of the direct pressure. The diagram also shows the necessary connections for the fire alarm hand.

This attachment for the gage was made by Mr. Haseltine in about a half day, using the magnet out of a small electric bell, the total cost of the material, including the relay, being about $2.

Type of Insulated Pipe Joint Successfully Employed at Providence, R. I.

During the past year three 16 in. wooden insulation pipe joints were set at river crossings of the high pressure fire service water mains in the city of Providence, R. I. The accompanying sectional elevation shows the details of design of these joints. This joint is made with the regular 16 in. sleeve, bored out 1-16 in., and white pine staves and ring. The thickness of staves is determined after the machine work is done.

Besides two other 16-in. double insulation joints in the high pressure fire service mains, the water works distribution system now has 15 6-in., 3 8-in., and 3 16-in. joints of this character in service, besides several small ones on lead service pipes. The first wooden insulation pipe joints were installed in 1907, and at the end of six years their efficiency [illegible]



Sectional Elevation of Insulated Pipe Joint for 16-Inch Water Pipe, Providence, R. I.

[illegible]

The illustration and the descriptive matter here given are taken from the latest annual [illegible]

[illegible]

Russia is feeling the scarcity of iron, espe- [illegible] and little material is left for town and village construction work

Motor Truck Attachment for Rapid Closing of Large Gate Valves, Successfully Employed at Boston, Mass.

(Staff Article.)

The water department of the city of Boston recently perfected a device which, attached to a White auto truck, is employed for the rapid closing of large gate valves in the distribution system. The device, which was invented by Mr. George H. Finneran, superintendent of the Boston distribution system, can close in 10 minutes gate valves 36 ins. in di-

View of Motor Truck Attachment for Rapid Closing of Large Gate Valves.

ameter which, in closing, require 307 turns of a gate wrench. Prior to the introduction of the device here illustrated and described the closing of valves of this size required the services of four men for 45 minutes. The importance of this saving of 35 minutes in the closing of such a valve is forcefully illustrated by considering the fact that the breaking of a 36-in. main on one of the important streets of Boston, where building is congested and property values are very high, would permit the escape of 50,000 gals. of water per minute. The chief advantages of the device are: It reduces property damage due to breaks in large mains, saves water, permits rapid regulation of water volume at fires, is independent of the physical endurance of workmen and facilitates testing of gates.

The truck responds to all fire alarms and other emergencies where water must be controlled to prevent loss or damage. The calls are frequently overlapping. Crews are on duty day and night. Boston is 15 miles in greatest length and 9 miles wide. The runs vary from one block to the end of the water distribution system. Under the old scheme, when several gates had to be closed, the few men available at night were almost exhausted before shutting the last gate. By its ability to work continuously the truck has relieved the fear of being unable to cope with any emergency.

The gate-closing device consists of a universal wrench socket with a worm gear enclosed in an aluminum housing and mounted on the running board of the truck, so that it can be easily brought into position immediately over a water-gate manhole. When the truck is in position a wrench is slipped through the socket. This wrench fits the nut on the gate gear below. The universal wrench socket, together with a universal joint on the end of the wrench, affords sufficient flexibility in case the truck is not on level ground or in case the wrench socket is not directly over the gate nut. It is an easy matter, however, for the driver to bring his truck into the exact position.

The worm gear is driven off the regular transmission of the truck. The device is operated by a lever placed upon the side of the truck and easily accessible to the driver. In closing gates the forward speeds of the transmission are used. In opening the reverse is used. All gears are made of chrome or nickel steel. All bearings are ball bearings. The aluminum housing is firmly bolted to the frame of the chassis and well braced to resist torque. The wrench is a hollow square steel tube terminating in a specially hardened steel socket with universal joints between socket and tube.

The gates are equipped with indicators showing the position of the valve and informing the operator when the valve is seated or entirely opened. Where indicators have not been attached to the gates a counter is used. This counter is placed on the end of the wrench, recording the number of its revolutions. This helps the operator to determine when the valve is entirely up or down. As a means of safety, in the event of the valve seating with force or before the operator expected, a pin of known strength, placed in the universal joint of the wrench, breaks off and breaks the line of force between the engine and the gate, thus preventing damage to either the gate or the gate operating device.

Lessons Taught by the Construction, Failure and Reconstruction of a Small Dam Near Tullahoma, Tenn.

The present article indicates certain lessons which may be learned from the construction, failure and reconstruction of a small dam, built for an institution having limited funds, near Tullahoma, Tenn. The article, in addition to discussing the design, construction and failure of this dam, also discusses the engineer's responsibility for works of his design, and also considers the ethics of cases wherein an engineer does not supervise the construction of works designed by him. The information given is taken from a paper by the designer of the dam, Mr. John Wilkes. The paper was presented before the Engineering Association of the South and was published in the proceedings of the society for July-September, 1914. In introducing the subject of the engineer's responsibility Mr. Wilkes points out that it is the custom, in such discussions, to assume that the engineer has full control of all the engineering features of a piece of work done under his direction and with which his name is associated. This may be true in many instances but Mr. Wilkes feels sure that there are many cases, such as the present one, where the authority of the engineer is very circumscribed indeed, and to hold him responsible for the failure of any construction under such conditions is an act of great injustice. The value of the article lies chiefly in its frank discussion of the business and ethical aspects of the case under consideration. Mr. Wilkes is authority for the statement that in the locality of Nashville engineering regulations are perhaps rather lightly esteemed both by those within the engineering profession and the general public outside the profession.

The institution for which the dam was built is the Pythian Home, Ovoca, about three miles from Tullahoma. On this 400-acre tract of land there is a waterfall 65 ft. high.

One of the features of Ovoca was to be an artificial lake made by damming the waters of the small creek which flowed through the grounds and which formed the waterfall menioned. The original purpose of the lake was wofold. It was intended to add to the beauty of the grounds, providing a place for recreaion, and also to serve the purpose of providng water power by utilizing the drop of over 0 ft. into the gorge. It was at first thought hat the power derived from this waterfall vould be sufficient to operate an electric railvay from Tullahoma to Ovoca. At the proosed site of the dam there was a rock bluff on ne side, against which the creek impinged diectly and, after making a circle around this luff, poured over the waterfall at a point a hort distance below.

It seemed an easy matter to throw a dam cross from this rock bluff to the hill opposite t, the valley here being, fortunately, quite narow, and the rock bluff itself would have ormed a sufficient spillway for the dam by simly removing a little of the natural earth surace. Such a natural spillway would have enailed no additional expense, and the dam could ave been carried up higher than the top of his rock bluff, say, 6 or 8 ft., if necessary. The nly disadvantage would have been the sacrice of the amount of space at this point which he rock bluff took up. There would ave been that much loss of ground pace, but it would have been much ess than an acre in all, and apparently round space was not very valuable just here. The bluff would have provided a spillway more han 150 ft. in length, and this seemed to be he natural solution of the problem. On preenting this to Mr. Fox, the superintendent of he home, however, he stated that "they," meanng presumably the board of directors, did not hink well of this plan.

The first plan for the dam was for an earth lam with a concrete core, the earth fill to be 16 t. wide across the top, as it was desired to arry a road across the dam from one part of he grounds to another. The height of the lam was to be 16 ft., length 175 ft., and the ore was intended originally more as a safety eature than as the means of dependence. The arth dam was decided on, on the score of heapness and its adaptability for carrying a oad across from one side of the lake to the other.

After the first plan was made, nothing was lone on the dam for a long while, except to et a bid from contractors as to the cost of uilding it. The writer estimated the material n the dam originally as 250 cu. yds. of conrete and 2,500 cu. yds. of earth fill, and placed he price of the concrete at $8 per cubic yard nd the price of the earth fill at 20 cts. per ubic yard, which latter was too low. This nade the estimated total cost of the dam 2,500. When it was built later, the actual cost vas nearer $3,500. It was stated in this preiminary estimate that something should be dded to cover the cost of making the spillvay and the railings for the road. In Decemer, 1909, I wrote Mr. Fox:

> Further consideration of the problem of a safe spillway for the dam you are building leads ne to conclude that you should have a notch or spillway in the dam for times of flood. I ave made a new drawing showing such a spillvay, blue print of which I enclose herewith. I elieve the dam will wash out if such provision s not made, which would be very dangerous, as vell as total loss of the money you have put in he dam. The spillway shown on the drawing s to be made by leaving a low place or notch in he dam 70 ft. wide and 3½ ft. high. This can e closed during ordinary times by plank, keepng the level of the water up, and opened when here is a flood. To cross this low place, some kind of bridge would be needed, and I have shown a timber trestle which will also serve as a support for the planks or flashboards. This spillway will add a few extra dollars to the cost of the dam, but I think the dam will be absoutely unsafe without this and liable to total detruction in time of high water.

On Jan. 21, 1910, I wrote in regard to the uggested reinforced concrete dam: "There is o be considered the fact that reinforced conrete is a new form of construction, and reuires to be more carefully done and under stricter specifications and inspection than the old time-honored plan of building an earth dyke, such dams having been built since the times of ancient Egypt, thousands of years ago."

In the spring of 1911 actual construction was begun, and the work was pushed to completion in May. When construction of the dam was begun, at Mr. Fox's request I visited Ovoca and inspected the trenches of the foundation to see if the foundation was suitable. The concrete core was built according to dimensions. When water was turned in, about 21 days were required to fill the lake. From the time that construction was begun until the dam was completed and the water turned in the writer did not see the dam, but supposed all along that it was being built according to the plan furnished. On July 4, 1911, being in that vicinity, the writer, on his own initiative and without request from the Ovoca authorities, went to see the dam, and was very much surprised to find that the spillway had been omitted entirely and the earth fill put in only on one side of the core wall, on the upper side, and even there not to the full dimensions called for in the plan. On the lower side, in order to strengthen up the dam, some timber struts had been placed, spaced about 12 ft. apart and being of timber about 6 ins. square, the top end resting against the face of the core wall about onethird of the way down from the top and the lower end against the natural bed of rock on which the dam was built. On inquiring the reason for the omission of the earth fill on the lower side, he was told that it had been omitted on account of lack of funds and because it did not seem to be necessary, as the concrete core wall "looked strong enough" by itself, but that the structure was not then complete and was not considered finished, and that as soon as more funds became available, the intention was to put in the fill on the other side of the core wall. The whole thing looked so dangerous that on his return to Nashville the writer sent Mr. Fox a letter from which the following is a quotation:

> Your dam at Ovoca is in a very precarious condition, being greatly overloaded—in fact, having about double the load on it that it should bear. I am quite uneasy about it. I do not wish it to go out, and I know you do not; but I fear that there is great danger of its doing so; and if the dam should fail it would be a loss of perhaps $3,000 to you, and maybe tremendous damage suits, and probably loss of life. I would respectfully recommend that you at once draw down the level of the lake 2 or 3 ft., and when the necessary strengthening has been done, the present level of the lake can be restored. As it is now, you are liable to have a disaster almost any time.

No reply was received to this letter; and being busy with other work, the matter almost passed out of the writer's mind, until the failure of the Austin dam in Pennsylvania reminded him of it. It was not, however, until January, 1912; when the pressure of other matters was somewhat relieved, that anything further was written on the subject. On January 12, 1912, the following was written to Mr. Fox:

> On July 7, after visiting Ovoca on July 4, I wrote you that your dam at Ovoca was, in my judgment, in a very dangerous condition, liable to be washed out by the first severe flood that came along, and suggested that you strengthen it. I never heard from you, and it may be that you did not get my letter. I write again, because I do not wish to let an accident occur like the one at Austin, Pa., which happened last fall, and which is but one of a series of failures that could have been prevented. If I have no pecuniary interest in this dam, I have at least a professional one, in that its failure would hurt my reputation. I am quite willing for you to take the advice of any other competent engineer as to whether it is safe or not, but I know it is not safe, and I hope that you have strengthened it.

In reply to this letter I received a letter which almost any one might consider as relieving me of responsibility. It read as follows:

> I have not strengthened the dam, as I have never regarded it as complete. It is our intention to complete it before again turning water into it. During the recent rains the basin filled, although the drain pipe was open, and the water ran over the top of the dam for a day or two. But as we had done all we could to prevent the basin from filling, we trusted to Providence and let the matter go. The dam stood without any indication of injury to it. I assure you that I consider that your letter was prompted partially by your interest in us and partially by your self-interest to protect your reputation. Permit me to say just here, that your mind may be at rest upon the matter, that we did not build this dam in accordance with your plans. Your plan provided for a wooden spillway. This we did not take into consideration. The core wall was in accord with your plan only as to its basic dimensions. Therefore by no manner of means could it be considered that you are responsible for this dam, and I write you this to exonerate you wholly from any adverse criticism should anything result from the incompletion of the dam. You are at liberty to say that the dam was not built by your plan or in accord with your instructions. This should relieve you of any responsibility whatever.

Notwithstanding this plain relief from responsibility, as far as it could be given by a letter, I did not feel satisfied, and wrote in reply:

> I am less concerned to be exonerated in case the dam should fail than I am to prevent the dam from failing, with the attendant loss to you and possibly others, and possibly a loss of human life which no amount of exonerating could make up for. I do not wish to meddle in your affairs, but I have a sincere desire to help you all I can to make this structure safe at the least expense. I did not recommend a wooden spillway, but suggested a wooden bridge across the concrete spillway, and that bridge only to carry a road which you said you wished to have across the dam.

In answer to this I received a letter saying: "I am confident that you are actuated by nothing more than a desire to serve us and humanity at large." In another letter of February 24 Mr. Fox wrote: "I shall observe your directions as soon as the weather will admit of our doing any concrete work." It is to be noted that this left the matter in a very much better shape than it had been previously, as there was a definite promise to make needed changes; so that I felt well repaid for having written in January.

Another letter on the same subject, being on February 26, was in regard to placing buttresses, spaced about 12 ft. apart, to make the concrete core wall capable of carrying the load.

I wrote again on March 14 as to some changes in this construction, and the same day that this letter was written the dam failed by being washed out by a flood during the night. Mr. Fox wrote me:

> Yours of the 14th came immediately after our disaster; and if you did not have some intuitive or telepathic conception of what was about to come to pass, I am very much mistaken. There is a movement on foot at Tullahoma to replace the dam, and I trust it will be our pleasure to have you with us in connection with this reconstruction of the dam.

After the dam had failed, I felt disappointed that we had not got it strengthened, and yet a satisfaction that I had been persistent in urging that it was unsafe, although it had required at the time considerable patience to enter upon a disagreeable task of urging its dangerous condition and one bringing in no pecuniary reward, so far as could be seen at the time. After the dam had failed, the writer, at Mr. Fox's request, made a trip to Tullahoma. It was thought then by Mr. Fox that the dam could be replaced by May, building a timber structure. In fact, the timber structure was decided upon; and if it had not been for the confusion of the foremen in laying out some of this timber work, it is probable that the dam would have been rebuilt in that way, although a calculation made afterwards showed that the stresses in the proposed structure were entirely too large for the strength of the timber.

The question of the best way to rebuild the dam was then taken up, and it was decided that patching up the old core wall was not a wise procedure, and that it would be best to build a new concrete dam in the gap that had been washed out, using buttresses and making the core wall practically a slab of reinforced concrete, reinforced from buttress to buttress, with steel rods in the concrete, which were also to take care of the temperature stresses. In August I wrote him to be sure to leave the spillway, as shown by the drawing, 72 ft. wide; and this, after some persuasion, he did. In the same letter I said: "Please note that the reinforcing rods are to be spaced 10 ins. apart." About August 26 I visited Ovoca and inspected the dam as it then stood. The buttresses were not sunk into the bed rock, as shown on the drawing, but built on top; and much of the reinforcing steel had been omitted, because, as Mr. Fox stated, it was "hard to get."

The following is a quotation from a letter of Mr. Fox dated August 27: "I have yours of the 26th, and have given your suggestions a careful reading; will endeavor to abide by them It will be in some cases impossible, but I shall do the very best I can. I am left to accomplish a great deal with practically nothing to do it with; and, therefore, it is not to be wondered at that there is a lack of carrying out of all the details necessary." From first to last this was the constant condition of affairs—namely, lack of funds all along. So the plans were continually modified and changed, and often with very bad results.

In November, 1912, I visited the dam again, and Mr. Fox told me that there was less reinforcing steel in the concrete than the plan called for. I wrote him upon my return to Nashville that I did not think the dam in that condition was safe. In reply to this letter he wrote: "I shall do, of course, all in my limited power to make this dam safe."

Mr. Fox's death from pneumonia occurred in December, 1912, when the dam was still uncompleted. Up to this time the building of this dam had been a series of disappointments to the writer. Things which it was thought had been decided on were changed and modified, some of them the most vital features.

On December 16 I wrote to Mr. H. W. Stratton, Mr. Fox's successor, as follows: "Referring to my letters of November 14 and 19, you will see that I wrote Mr. Fox that the dam in its condition then was very unsafe, and it is still unsafe, or was on December 1, when I last saw it; and as Mr. Fox was taken sick the day before that and was sick till he died, I presume it is in the same condition now as when I last saw it. The dam is in an unsafe condition, I think; and I hope you will see your way clear to taking steps to [illegible]

On January 3, 1913, I wrote: "I took the liberty of writing you on December 16 in regard to the dam which Mr. Fox at the time [illegible] your having been quite busy—I take the lib- [illegible] importance of having it put in proper condi- [illegible] terribly handicapped by the lack of funds, and I haven't at present a dollar available for that purpose."

During the summer of 1913, again on my own initiative, I made a visit to Ovoca, inspected the dam, and was again very much surprised, for I found that they had closed the spillway, which Mr. Fox, at my earnest solicitation, had left in building the dam, and had put solid concrete into the gaps, completely filling them. I asked Mr. Stratton if he had no fear for the safety of the dam, and he said, "None whatever," as he had seen water pass over the top of the dam its whole length and more than 1 ft. deep. I asked why the spillway had been closed up with solid concrete instead of with planks, as the design called for, and he said the planks were "impracticable."

On July 22 I wrote the last letter in the correspondence on the dam as follows: "Referring to the matter we discussed while I was at Ovoca last Sunday—namely, the dam there—I do not wish to annoy you, but I do wish to do my whole duty toward you and the public whose safety is affected by the security of the dam. In my judgment, the dam as it stands at present is not safe. What you told me Sunday of its having stood three severe floods of course eases my mind to some extent; but I still do not consider it a safe structure, and I believe other competent engineers investigating it thoroughly would come to the same conclusion." To this letter no reply was received. There the matter rested for the present. The second dam still stands.

Reviewing the history of the construction of this dam, the following points came up in which the actual construction was a disappointment to the engineer in plans not being carried out: 1. The fill was omitted on the lower side of the concrete core. 2. The fill put in on the upper side of the concrete core was not as wide as the plans called for. 3. The spillway was entirely omitted in the first dam. 4. The level of the lake was not drawn down when the dam was reported to be in an unsafe condition. 5. The owners stated that the dam was not built by my plans, when really it was, so far as it was built by any plan. 6. The second dam was planned for a timber dam, without any investigation by an engineer as to the strength of the proposed timber construction. 7. When it was decided to make a concrete instead of a timber dam, the breaking up of the pieces of the old dam for use in the new dam was abandoned, because it was said to be too much trouble to break the concrete up. 8. The reinforcing steel put in the second dam was less than the plan called for. 9. The buttresses of the second dam were not cut down into the solid rock, as the plan called for, to prevent sliding; and the reinforcing steel in these butresses was put in in an entirely different way from what the plan showed. 10. The fill on the upper side of the second dam was not made for a long time after the concrete was put in.

On these ten points, and others that could be mentioned, the construction of the dam was a great disappointment, and there was so much variation from the plans submitted and nominally accepted as to make the final structure entirely different from the structure planned

The concrete core wall of the first dam was not intended to be a dam in itself, although it was so used, so that no analysis was made of it by itself when the plan of the dam was prepared; but after it had stood as a dam by and of itself for nearly a year—or, to be more exact, for about ten months—I made an analysis of the section of this wall in the usual way, and from this it appeared that it was very remarkable that the dam stood at all. The second dam is insecure against sliding.

A professional man feels a delicacy, or should feel so, in the matter of offering his services where they are not called for. But in every instance, in connection with this busi- [illegible] struction of this dam; and, as stated above, [illegible] work several times; and if I had not voluntarily done this, the probabilities are that I would have known nothing about the matter until failure had occurred. Entirely too many failures of dams have occurred, but the popular idea that seems to be prevalent that engineers do not know how to build dams well enough to avoid failure makes it worth noting that only six dams of over 50 ft. in height have failed, and in two of these—the dams at Austin, Texas, and Austin, Pa.—the advice of the engineers was disregarded; so that the case is not as black for the engineers as it is sometimes supposed to be. Just here it may be stated, on the authority of a writer in the Engineering News of Sept. 4, 1913, that no arch dam has ever failed.

As there is very little that is admirable about this dam, and nothing that would be copied, the value of it as an object lesson must lie in a different direction—that is to say, no one would ever build such a structure designedly; and, therefore, the construction of this concrete core wall, in the first dam, and relying upon it to act as a dam of itself, may be looked upon as an experiment on a full-sized scale in the strength of a very thin concrete dam. A very costly experiment, it must be admitted, since its failure entailed a loss of about $3,000, but still an experiment from which something of value may be learned, due to the fact that it was submitted for nearly a year to the test of actual conditions, such as overflow by floods, passing through the low temperature of a severe winter, etc. No one could afford to designedly experiment with a full-sized dam under these conditions; and it is, therefore, all the more desirable that we learn as much as possible from the behavior of the dam. In this connection it may be said that while nearly everything else about the dam was actually constructed different from what the plan showed, this concrete core wall was strictly in accordance with the drawings. Of course this experiment loses some of its value, due to the fact that the conditions were not varied, one at a time, as they should be in a true experiment. Furthermore, there is some uncertainty or indeterminateness in the load applied to the structure, due to the fact that the earth fill was used behind the dam; so that the exact pressure cannot be calculated as it could be if the water pressure alone were the only factor involved. But the fact that the dam stood for so long without failing seems to the writer to indicate that an earth fill of this kind operates to decrease the pressure against the dam. According to Wegmann's treatise on dams (see page 89), the addition of an earth fill behind a masonry dam is a type of construction recommended by Professor Intze, of Aachen.

The next lesson that may be learned from this dam is in regard to the matter of spillway for an overflow dam. Neither Mr. Fox nor Mr. Stratton thought this was of any use. Mr. Fox thought that a 24-in. pipe was sufficient to take care of this feature, the 24-in. pipe to be placed at the top of the dam. A 30-in. pipe was actually put in for drainage at the lowest point in the dam; and, although this was wide open when the flood came, the water rapidly rose to the top of the dam, filling in a few hours the basin that in the normal flow of the stream required 21 days to fill. Considering this 30-in. pipe as an orifice, under a head of 16 ft., its discharging capacity would be about 150 cu. ft. per second; whereas the spillway 3½ ft. deep and 72 ft. long, as this dam had, in the original design, would have a discharging capacity of 1,500 cu. ft. per second, or ten times as much as the 30-in. pipe, and, of course, still more capacity than a 24-in. pipe under a 16-in. head, and still again very much more capacity than a 24-in. pipe under a head of 1 ft., say, as it would have if placed at the top of the dam. And even this spillway was insufficient. As stated above, the drainage area was considered to be 5½ square miles; for 6 square miles Fanning gives (see page 381 on his "Treatise on Hydraulics and Water Supply Engineering") a spillway 2.9 ft. deep and 54 ft. long. I increased his figures 50 per cent on the length of spillway and 16 per cent on the depth of spillway, and still did not get a spillway large enough. His figure for flood volume is 890 cu. ft. per second from 6 square miles, or 148 cu. ft. per second per square mile. In the American Civil Engineer's Pocketbook, second edition, published in 1912, page 904, is a table giving the maximum rate of discharge of streams. In this table a discharge of a stream draining 5 square miles—namely, Mad Brook, Sherburne, N. Y.—is given as 262 cu. ft. per second per 6 square miles, being the largest discharge given in that table. The average of all the streams having less than 10 square miles drainage area is 116 cu. ft. per second per square mile.

The next point to consider in the lessons to be learned from the failure of this dam is that of uplift, or pressure of the water, on the base of the dam, due to percolation and tending to overturn the dam. Some engineers are of the opinion that the full hydrostatic pressure of the water, due to the head of water behind the dam, should be considered as acting over the whole area of the base of the dam in determining its stability; while others think that two-thirds of the pressure of the water at the heel of the dam, decreasing to zero at the toe of the dam, is a sufficient allowance.

The last time I saw Mr. Stratton he stated that, in spite of all that he could do, there was a constant percolation of water under the dam—that is, between the base of the dam and its foundation; and this in spite of the earth fill at the back, which extended 32 ft., upstream at the base of the dam. This seems to show that water does get under the base of the dam, but it is to be noted that the material available near this dam was not very suitable for an earth-fill dam, being too gravelly and not having enough clay in it. Furthermore, that the width of this dam at the base was very small, and the observation as to leakage was made after the second dam was built, when part of the old dam was standing in a somewhat shattered condition.

In the treatment of the design of a masonry dam it is quite common with the textbooks to treat a section of the dam 1 ft. in length. With a gravity dam this is certainly correct; but the question of expansion and contraction over the whole length of the dam is likely to be lost sight of in such treatment, and for a dam exceeding 100 ft. in length the expansion and contraction due to temperature changes is quite appreciable and should certainly be given careful consideration. This is even more true in regard to a reinforced concrete structure, where the integrity of the structure depends on the reinforcing steel being unbroken. The amount of steel recommended for taking care of the thermal stresses alone is given by W. W. Colpitts in the Railway Age as 0.6 in. per square foot. A reinforced concrete dam is likely to be quite a light structure; and unless designed so as to be held down by the water, it is likely to fail by sliding. It is quite essential that such a dam have ample provision for thermal stresses; otherwise the concrete and reinforcing steel is likely to break halfway between the buttresses, where bending moment is maximum, and thereby reduce the strength of the slab to zero and cause a total failure of the construction. The amount that should be allowed for thermal stress could be calculated pretty closely if the average temperature of the concrete during a period of cold weather could be accurately determined. But this average temperature will depend upon the thickness of the concrete, its exposure, and the rate of penetration of the cold into the mass of the concrete, which will involve the duration of the cold spell. In this part of the country the variation of temperature is not so great as further north; but even here we often have zero weather, and the thermal stress is likely to be a serious matter; and I believe it is due to the thermal stresses that the first dam failed in the winter after having successfully stood with the lake full of water throughout the whole summer.

The type of dam adopted for the second dam was one with buttresses. These strengthen the dam very much, if the slab from buttress to buttress is made strong enough to carry the load to the buttresses. But it is worth pointing out, as shown by Prof. Ira O. Baker, in his "Treatise on Masonry," page 323, ninth edition, that the usual analysis for the pressure of the dam on the foundation does not apply to this type of construction, being only applicable in case the section considered is rectangular. Of course it can be assumed that all the load is transmitted by the slab to the buttresses and none of it from the slab to the foundation; and by assuming that the buttress acts in the same way as the base of a solid dam, some kind of an approximation can be made; but these assumptions are likely to be rather far from the truth in some cases.

In the light of all the foregoing descriptions and data on this dam, what should be said as to the responsibility for the failure of such a dam? When no material damage occurs, this question is not likely to arise, or is given very little consideration; but when the damage is heavy or there is loss of life, this point becomes very important; and when it is, what should be the answer to this question? The failure of such a dam is due partly to a bad design partly to bad construction, and chiefly to a great divergence between the design and the actual construction. I think the correspondence given above shows that the owners did not at first feel their responsibility very keenly, or, if they did, they did not mind varying from the nominal plan to a great extent. The institution for which this dam was built was of a philanthropic nature, and I wished to do all I could to help the undertaking along, and, therefore, felt very altruistic toward it; but I believe now, after an experience of five years, that better results to them, as well as to myself, would have resulted from charging them more for my services, for then perhaps they would have respected the plans more and varied less from them than they did. It may be stated that at any time only a few hundred dollars additional would have been required to make the dam entirely safe, possibly $500 or less; but that, or even $200 or $300, was lacking.

CONCLUSIONS.

To make a summary of this paper, the following conclusions may be stated:

1. Any construction where the funds available are very inadequate is sure to be unsatisfactory to everybody concerned—owner, engineer, contractor, the innocent public.
2. It is best for the engineer to have authority—that is, best for the owner.
3. It is also best for the engineer to have responsibility—that is, best for the engineer.
4. In a dam like this the drainage area should be carefully determined, by a special survey if need be.
5. The spillway should then be made amply large for this drainage area, some of the older formulas giving too small a spillway.
6. The material at the point of construction should be closely observed, and the dam designed accordingly.
7. If reinforced concrete is used, plenty of steel should be put in to take care of thermal stresses.
8. Where buttresses are used, there is uncertainty as to distribution of pressure in the base.

ROADS AND STREETS

Construction and Maintenance Details of Concrete Road Work in Wayne County, Michigan, in 1914.

The development of concrete road construction in Wayne County, Mich., is of interest in that there the first work of any great magnitude was accomplished. Details of construction and maintenance methods used during 1914 and given in a recent report of Board of County Road Commissioners of that county are presented here.

A survey of the road is made, plan and profile prepared, and these together with specifications and application for state reward are filed with the state highway department. Following the provisions of the county road law bids are asked for by advertisement for the construction of the road (a purely perfunctory proceeding, as no work has been contracted nor have any proposals been received for the construction of any road for several years past). When the bids are received, there is the option of accepting the lowest one and awarding the contract for the work to that bidder, or of rejecting all the bids, and doing the work ourselves under the day labor plan.

As soon as weather conditions will permit in the spring, and after the road has been staked out, the subgrade is prepared and shaped, the major portion of such work being done with scarifiers and graders,, the hauling power for which is furnished by steam tractors or rollers. Careful attention is given the grade to eliminate soft spongy places, and a 10 ton roller is used to roll it hard. To produce a good concrete road, thorough drainage is necessary in addition to a good subgrade. Both proper grade and drainage are difficult to cope with in Wayne County as the county for the most part is flat and situated in a valley not easily drained. The subsoil is largely of heavy, sticky clay with some loose, deep sand.

INDUSTRIAL RAILWAY.

A great problem has been in getting the materials on the subgrade, and various plans are followed on the different roads, due to varying conditions. On one road an industrial railway is used to transport all materials from the point of receipt to the point of construction. This outfit consists of a 7 ton, 30 HP. locomotive, 61,000-lb. double side V-shaped steel dump cars, each of 1½ cu. yds. capacity, and five 16 ft. flat cars, all of 2 ft. gage. The track is furnished in built up units, 15 ft. in length, consisting of steel rails fastened to steel ties. A turnout may be laid wherever needed by replacing a section of track by a switch or curve, as this is also furnished in 15 ft. lengths and of such radius that locomotive and cars will readily pass through. Two men can handle a section of track weighing 225 lbs. It is not necessary to turn the engine around as it pushes as well as pulls the load. The average train consists of 30 loaded cars, though as many as 42 cars have been hauled.

Materials for the work begin to arrive before actual concreting is in progress. These are immediately unloaded by a clamshell bucket and thrown into stock piles. After concreting has been in progress, the materials are unloaded directly from the railroad cars to the steel dump cars and transported to place. The unloading crew consists of 4 laborers, 1 engineer, and 1 team. The team hauls 7 loaded cars to the siding where trains of thirty cars are made up. Wherever shipments are delayed the stock pile is resorted to for materials. Work is started at the end farthest away from the railroad switch. Five miles per hour is the average scheduled speed, including time for coaling and watering. The actual running speed enroute is from 8 to 10 miles per hour.

There is no danger of dumped materials falling back upon the track, as the center of the pile is about 3 ft. from the edge of the nearest rail. Two men are able to tip a loaded car body. Materials are readily measured in the cars and distributed along the road at such [illegible] meet the require[illegible] the mixer. The aggregates are loaded into the dump cars, the cement in any available cars, and expansion plates, asphalt filler and other necessities on the flat cars. Coal for the mixer is likewise brought to the site over the railway. As concreting progresses the haul becomes shorter and the track is taken up. These rail sections are transported to the loading point on the return trips of the train.

The tracks for our industrial railway can be laid on any surface over which transportation

Fig. 1.—Type of Deep Roadside Ditch Used in Level Country.

of any kind is at all possible. Rainy weather and muddy roads do not impair the efficiency of rail haulage, nor is the load too heavy for the ordinary highway bridge. A factor of much importance is that very little hauling space is required. When necessary the track can be laid on the berm of the road, so that it is possible to haul material either in the same direction as the concreting is proceeding or else in the opposite direction. The railway, too, is practically independent of labor conditions, as one engineer and one brakeman make up the entire crew. Although the unit rail sections provide an easy method of crossing highways and steam and electric lines, and the clearing of these when necessary by removing the rail sections, it is not always possible to obtain permission to make such crossings.

STEAM HAULING OUTFIT.

Where the industrial railroad is not available a combination of team and engine hauling is

Traction outfits are also handicapped by bad roads, rainy weather and the limited speed which they are capable of attaining. Road rollers furnish the motive power largely for hauling wagons, each of which holds seven tons of material. Over earth roads one wagon is usually all that can be drawn. When, however, a concrete road is available for hauling, six wagons containing a total of 42 tons of material are made up into a train and hauled by one 10 ton roller. The crew of one of these trains consists of an engineer and a fireman. Our hauling operations involve the use of the industrial outfit, 8 traction outfits and from 100 to 150 teams.

PLACING OF MATERIALS.

One man on the grade has charge of the dumping of material and he is furnished with a copy of Table I in order that the material may be properly placed to minimize rehandling.

TABLE I.—MATERIALS TO BE PLACED PER 100 FT. STATIONS.

Width of road. ft.	Bbls. Cement.	Yards Gravel.	Yards Sand.	Wagon loads Cement.	Wagon loads Gravel.	Wagon loads Sand.
12	45	26	13	4	20	10
15	56	33	16	5	25	12
16	60	36	17	6	27	13
18	68	39	20	6	29	15

Cement—45 bbls. to 1 load. Gravel—1⅓ yds. to 1 load. Sand—1⅓ yd. to 1 load.

The foreman in charge of the yard is also furnished with a table of quantities showing amounts required on different widths of roads so as to avoid surplusage at a given point, Table II.

4 1-5 miles: at another point it ran up as high as 5 miles. On the West road the longest haul was over 5 miles.

Water.—The transportation of water of which large quantities are used in mixing and curing concrete, and supplying the mechanical equipment such as concrete mixers, traction engines, road rollers, etc., has often been a serious problem. This problem has been solved by laying 2 in. pipe along the road from the nearest source of supply, and pumping the water along the route either by gasoline engines or electric motors. On the Eureka road water was pumped a distance of over ten miles from the Huron River, the power being furnished in this case by an electric motor. On one section of Seven Mile road water was pumped by gasoline engines from a drive well to the mixer three miles away. On another section of the same road we secured our supply from a Detroit city water main and pumped it 2½ miles.

Cross Section.—The standard width for secondary roads of concrete is 15 ft., with a minimum width over all of 24 ft. The concrete is 6 ins. thick at the sides and 8 ins. thick in the center built on a flat subgrade. Six inch channels 12 ft. long are used instead of wooden forms along the side. As these forms are subsequently used to support the templet and bridge from which the finishers work, great care is exercised to get them true and rigid. Expansion joints consisting of two thicknesses of asphalted felt (about ¼ in.) inserted between two Baker armor plates, are placed in the road 25 ft. apart.

Mixing.—A concrete mixer is used which

TABLE II.—QUANTITIES OF MATERIALS FOR ONE MILE OF ROAD.

Width of road.	Pebbles Cu yds.	Pebbles Tons.	Pebbles Carloads.	Sand Cu yds.	Sand Tons.	Sand Carloads.	Cement Bbls.	Cement Tons.	Cement Carloads
12 ft.	1,369	1,916	55	684	855	20	2,396	455	13
15 ft.	1,711	2,395	69	855	1,069	24	2,995	569	16
16 ft.	1,825	2,555	73	912	1,140	26	3,194	607	17
18 ft.	2,053	2,874	82	1,026	1,383	31	3,594	682	20

Cu. yd.—	Tons.
Pebbles weighs	1.4
Sand weighs	1.25
Cement weighs	0.19
Carload—	**Tons.**
Pebbles weighs	35
Sand weighs	44
Cement weighs	36

Stock piles are also established in the fall at various unloading points so that breakdowns, car shortages, intermittent or irregular deliveries will not interfere with the work after getting under way in the spring.

travels under its own power, from which a 20-ft. boom projects which swings over a 180° arc. The dumping bucket is carried out on this boom under power and eliminates much hand labor. One batch consists of 3 sacks of Portland cement, 4½ ft. of sand and 9 ft. of gravel. The specification that the batch shall receive 16 complete revolutions and remain in the mixer for 1 min. is resulting in producing an excellent quality of concrete. Batches are discharged at intervals of about two minutes. On a 15 ft. road the crew of 32 men is capable of laying an average of approximately 450 lin. ft. of concrete during a 10

Fig. 2. Transferring Road Material from Railroad to Industrial Railway [illegible]

Fig. 3.—County Road Camp, Industrial Railway Loading Yards and Transfer Loader.

[illegible] weather [illegible] hardships, and [illegible] [illegible] [illegible]

Haul.—During the past year long hauls were [illegible] the farthest point of haul was [illegible] miles. On one portion [illegible]

hour day. As high as 525 ft. has been laid when conditions were at the best.

Before any concrete is placed in order to prevent absorption of the water from the [illegible] material, the subgrade is thoroughly wet down. The mix is fairly wet, and of such consistency that men working in the concrete sink four or five inches. Clean material is an absolute requisite to securing good concrete. Pebbles are washed and screened so as to be free from loam, clay, and other foreign substances. They range from ¼ to 1½ in. and are so graded as to reduce the voids to a minimum. Sand is bank sand, washed and screened, free from loam, clay, etc., and ranges in size from ¼ in. to dust with the coarser particles predominating. Wayne County has no stone hard enough, and all stone and sand used are shipped from outside points.

When the mixer is close to the stock piles, congestion is avoided by having 6 men load the barrows and 6 men wheel the materials to the skip. Wheelers and shovelers alternate in their work for each successive block of concrete, a scheme that has proved efficient because the variety of work tends to prevent its becoming monotonous. When the stock piles are a relatively great distance from the mixer, each of the twelve material men loads and wheels his own barrow so that a continuous stream of material is always enroute.

Finishing.—The concrete is brought to grade and shape by the use of a templet. This templet or strike board is made of two-inch plank, preferably in a single piece, the curvature of the edge being made to exactly conform to the finished surface of the concrete road which is crowned approximately one-fourth of an inch to the foot. On each side of each end is an iron handle for drawing the strike back and forth. The curved edge is shod with 1 in. angle irons bent to the curvature of the strike, giving it a metal wearing surface. The length of the strike exceeds the width of the road metal by 1 ft. A 12 in. plank is suitable for width of road metal up to 18 ft. If the road is wider than this, a built up form of strike is necessary, which is trussed to prevent a sag or flattening of the crown. The strikeboard is sawed back and forth on the side rails, and moved slightly forward at each stroke, giving the concrete its initial shape. After this operation no workman is permitted to disturb the concrete in any way either by stepping in it or by throwing anything upon it. This rule is imperative, as a violation of it means a road with waves and depressions by causing the neat cement and finer particles to rise to the top. We aim to have the stone take the wear as it is the hardest part of the aggregate.

The final finishing up of the road is done by two men who are termed "floaters," who work from a bridge which rests on the side rails, having no actual contact with the concrete. A wooden trowel of home manufacture is used for this purpose. The use of a trowel of this nature prevents the road from becoming slippery.

When the concrete will stand of its own weight, the side rails are removed and a 3 in. bevel is made at about 45° by cutting off the edge with a shovel, and allowing the surplus thus cut off to fall to the side. This prevents a sharp division line between the concrete and the shoulders. Each day's work is finished up to an expansion joint, and no more than 20 mins. is permitted to elapse between batches during the day. The day following the laying of the concrete it is covered with about 2 ins. of sand or loose soil such as is available, and is sprinkled during the day for 10 continuous days. This prevents the road from drying out and is an important factor in properly curing the concrete so that it will attain its maximum hardness and strength. Plenty of water is vitally essential in producing good concrete.

Roads are not opened for traffic until from 3 to 6 weeks have elapsed after the last batch of concrete is laid; the length of time depends upon the season of the year as concrete sets up much more slowly in cold weather than when it is hot and dry.

Shoulders of crushed stone or gravel, whichever is the more available, are built 3 to 7 ins. thick and 3 to 4 ft. wide on each side of the concrete; on many roads additional width of earth shoulders is built. This work is not started until after the road is at least three weeks old.

LABOR.

During the past year most of the work has been carried on well away from towns where suitable quarters could not be obtained for the men, so we provide sleeping and eating quarters in tents along the road. The men obtain food from the nearest town and cook their own meals, while the water supply for the roads serves as well for the men. The mixer crew's quarters are kept at sites a convenient walking distance from the work, and the men unloading and transporting materials live in tents pitched close to the railroad siding.

Three concreting crews were maintained in the field during the past summer and to each of these crews is attached a grading crew, a shouldering and ditching crew, an unloading outfit, and a culvert crew. An additional bridge and culvert crew was also kept busy building bridges and culverts over roads previously improved. All work is specialized and machinery is used wherever possible since man and horse labor are scarce and expensive in the country districts during the road building season.

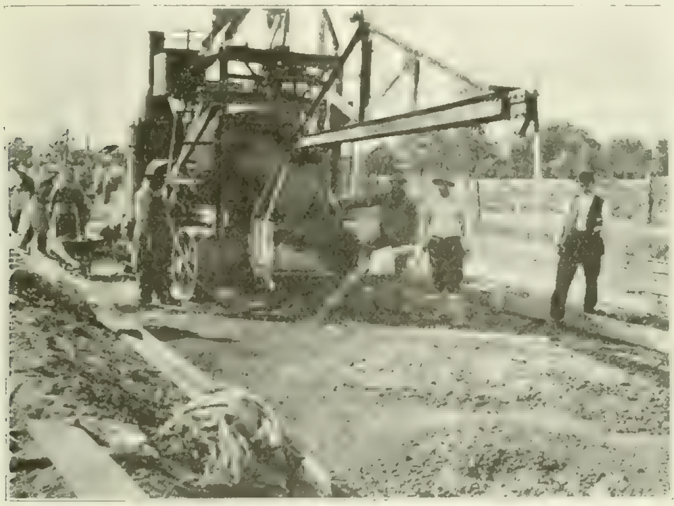


Fig. 4. Sprinkling Subgrade Preparatory to Placing Concrete. Note Side Forms in Place.

Fig. 5. Sprinkling Finished Road and Covering with Earth. Sprinkling Is Continued at Least 8 Days.

MAINTENANCE OF CONCRETE ROADS.

All ditches and culverts are cleaned out before freezing weather sets in and are also cleaned out in the spring. Bridges and guard rails are also painted and weeds and brush removed. The shoulders of narrow concrete roads need considerable attention and this is one of the reasons for altering construction to a width of not less than 15 ft.

Wherever cracks have developed or any spalling has taken place at the joints they are maintained by the use of tar and sand. A crew consisting of seven men and a team, provided with a tar kettle is utilized for this work. The foreman is paid $5.00 a day, the team and driver $5.00 a day, the "tar man" $3.00 a day, two laborers $2.50 each and two laborers $2.25 each. The tools used consist of two wire bristle brooms, a wheelbarrow, a couple of shovels and a tar bucket with a round spout.

A grade of Tarvia between A and X with a melting point of approximately 80° F. is now used exclusively. A lighter grade was first tried out, but did not give such permanent results as the heavier grade. Two men are utilized to sweep all cracks clean with the wire brooms, after which the man with the tar kettle fills the cracks with the tar which is heated to about 225° F. An excess of tar is poured in so that it extends an inch or so beyond the edges of the crack. It is then allowed to stand on the crack for a few minutes to prevent it from "bubbling" out in case the sand is wet. The sand, which is dry and coarse, is spread with a shovel over the crack and into the tar, and the whole is left for traffic to iron out. The excess tar and sand is worn away leaving a smooth, even surface, over which no jolt is apparent in passing either with a horse drawn or motor driven vehicle. This method of repair prevents the edges of the concrete from spalling and chipping, and no water can get through to the subgrade to freeze and heave in cold weather.

The work is preferably done on hot, dry days. It has been suggested to us that the better time to handle this work would be in the late fall when the cracks would be open the widest due to contraction, but the results we have secured in the summer months have been so satisfactory, that we have not tried out the latter plan. The small pit holes which are due simply to some foreign substance getting into the concrete like clay, wood or some fragment of inferior rock which might chance to be a part of the aggregate, are treated in the same manner.

Approximately two miles of concrete was previously covered with a tar and sand top. This mileage has also been taken care of, the same grade of Tarvia being used for other maintenance purposes. Instead of sand, washed and screened pebbles are used. These sections of road came through the winter in much better condition than ever before, and the repairs were correspondingly lighter. The method followed has been to sweep the spots to be treated with wire brooms after which the tar is poured. The pebbles are then spread over the tar from barrows or wagon, and rolled in with a 10 ton roller.

MAINTENANCE OF EQUIPMENT.

Each construction crew is furnished with a mechanic, whose duty it is to keep the equipment of that particular crew in the highest possible state of efficiency. Every piece of machinery acquired is constantly under the eye of one of these men from the hour of its delivery. Weaknesses are usually detected and the proper remedy applied before an actual breakdown occurs. Where a break does occur, no time is lost in replacing or repairing the broken part.

At the close of the season all machinery followed by the mechanics is assembled at the machine shops at Wayne. The process of overhauling then commences. During the winter months these men make a detailed examination; defective parts are removed and replacd by new, steel work is cleaned, woodwork is painted, etc. Every effort is made to check the inevitable advance of deterioration and decay, and when the equipment comes out for the resumption of work the following spring, it has been brought back to the standard of newness as far as skilled labor and up-to-date machinery can carry it. For repairs and renewals have been expended $11,809.05.

[illegible]

Culverts up to a span of 18 ft. are built according to a uniform plan. All are of reinforced concrete of the flat slab type, the slab supported by the abutments at the sides of the stream. The concrete mix in the footings is 1-3-6; in the wall 1-2½-5; and in the floor slab and copings 1-1½-3. Twisted bar reinforcement is used. The culverts are designed to carry a 15 ton live load with a factor of safety of four. Where a bridge has a span in excess of 18 ft., or where special or unusual conditions are encountered in the construction of a particular bridge or culvert, the plans are altered from the usual standard to meet the contingency.

CONCRETE TILE.

A tile plant is located in one of the buildings of the storage yard at Wayne. A number of men are kept constantly employed at the buildings for miscellaneous work, and whenever other work becomes slack, the men are kept busy making tile. In other words this industry is used as a filler. The tile are manufactured 8, 12, 18 and 24 ins. in size, and the [illegible] sharp sand.

New Course in Road Building, Drainage and Irrigation

[illegible] highway engineering of the Kansas State Agricultural College is planning on giving a short course in road building, irrigation and drainage during the coming winter. The course will be of ten weeks' duration and is intended to accommodate county surveyors and county engineers at present in practice who desire additional technical work along lines in which they are particularly interested.

Surveying, road construction, bridge and culvert construction, road laws and administration, irrigation and drainage engineering and specifications and contracts are among the subjects which will be treated in the cour[illegible]

Rock Asphalt Pavements in Salt Lake City, Utah.

The greatest difficulty encountered in the use of crushed rock asphalt as a paving material lies in securing a uniform product, i. e., a uniform distribution of bitumen in all portions of the [illegible]. The experience of Salt Lake City, Utah, with their material is interesting and indicates some of the limitations as well as the possibilities of rock asphalt.

9.3	Utah Rock Asphalt																
Extensions and Location	Per Cent Bitumen in Rock	Per Cent Refined Asphalt Added	Per Cent Bitumen after Heating	Temp Deg F of Heat	Asphaltic Cement Added: Kind	Asphaltic Cement Added: Penetration in Cms	200 Mesh	100 Mesh	80 Mesh	50 Mesh	40 Mesh	30 Mesh	20 Mesh	10 Mesh	4 Mesh	Thickness in Inches	Number of Tests
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]			18[illegible]	23[illegible]	1[illegible]9	[illegible]99	[illegible]5	[illegible]6	08	07	0	2	15
[illegible]	81	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]0	[illegible]	276	115	268	28	08	04	06	0[illegible]	2	35
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	78	185	266	108	29[illegible]	36	15	10	06	01	2½	28
[illegible]	78	15	105	[illegible]	[illegible]	85	[illegible]	235	128	225	21	08	07	04	0	2	45
Average	[illegible]	14	[illegible]	[illegible]		[illegible]	194	268	1.7	272	30	11	07	05	0	2	
[illegible]			[illegible]			4[illegible]									0	2	

Fig. 1. Average Analyses of Rock Asphalt.

Up to the beginning of the year 1913 there existed in Salt Lake City 31.03 miles of asphalt pavement, of which approximately 1½ miles were paved with rock asphalt, consisting of bituminous sandstone from California and bituminous limestone from Utah. The yardage of California stone was greatly in excess of the Utah limestone, as only two blocks or 1,320 lin. ft. were paved with the latter.

The first pavements of California bituminous sandstone were laid from 20 to 23 years ago. These pavements failed rapidly, and in some places the greater portions of certain blocks were resurfaced ten years later. As a result of the extensive patching annually on these streets, there is at the present time comparatively little of the original surface remaining.

The cost of maintaining the two blocks paved with Utah rock asphalt was very slight during the first ten years. Since that time annual repairs have been necessary, the maintenance averaging at the rate of ten cents for each square yard of original surface. This material [illegible] to withstand heavy traffic, and has, in a measure, been crowded from the mainly traveled portions of [illegible].

During 1912 the question of using Utah rock asphalt was agitated, and resulted in the letting of contracts for approximately 1.4 miles, or 29,000 sq. yds. of this type of paving.

Due to the inaccessibility of bituminous limestone in commercial quantities and to the abundance of a bituminous sandstone locally near Sunnyside, the latter has been used. Paving with this material was begun August, 1913. In the meantime additional contracts were let for 3.2 miles or 77,000 sq. yds. of rock asphalt. Samples first obtained of this material showed quite uniform bitumen content, but as development continued it became much more variable, until it has become necessary recently to test every hour at the crusher.

The following is an average analysis of 85 samples of raw material:

	Per cent.
Bitumen	9.5
Mineral aggregate passing a No. 200 sieve	18.9
Mineral aggregate passing a No. 80 sieve	39.8
Mineral aggregate passing a No. 40 sieve	29.2
Mineral aggregate passing a No. 10 sieve	2.6
	100.0

The range of the bitumen content has been from about 8 to 11.5 per cent, of which approximately 2 per cent is a light volatile oil. The mineral aggregate, after crushing, is, as will be noted, extremely fine. It is found that at least 11 per cent of bitumen is needed in this material to make it bind properly. Consequently for lack of sufficient native bitumen of suitable cementing value it is necessary to add refined asphalt before laying. In order to have the mineral aggregate properly graded the addition of coarser sand is also desirable.

Laid down in Salt Lake City, the cost of the raw material plus the crushing amounts to at least $6.50 per ton, or approximately 65 cts. per square yard for material at the plant, as compared with $3.40, or approximately 34 cts. per square yard for the raw materials required to lay an equal amount of artificial asphalt—namely, 1 ton of sand and filler at the plant and 240 pounds of refined asphalt. This excessive cost of the rock asphalt almost prohibits the use of a binder course, which is very essential to asphalt pavements.

The method of handling the rock asphalt this past season has been to crush the rock in a Williams crusher to pass a 10-mesh screen, and to heat it either in a Koehring hot blast mixer or in a stationary asphalt plant to a temperature of from 200° to 250° Fahrenheit, with the addition of refined asphalt when low in bitumen. After spreading on the street it is rolled with an 8-ton roller.

The results obtained thus far with the rock asphalt have not been altogether satisfactory. Whether heated by indirect heat or by the hot blast, the results have been practically the same. Though the greatest care has been exercised in raking and rolling, the finished surface is quite uneven and in places has ravelled and broken up shortly after laying. That this is not due to burning of the material is evidenced by the fact that it occurs on those portions where the indirect heat was used as

1913	Asphaltic Cement						Asphaltic Concrete Binder								Asphaltic Wearing Surface Mixtures											
Extensions and Location	Per Cent Bit. [illegible]	Pen [illegible]	Temp [illegible]	Pen [illegible]	Per Cent Volatile Matter	Ductility in Cms	Per Cent Bit. Sol. in CS_2	Per Cent Stone Passing 10 Mesh	Per Cent Stone Passing 1 Mesh	Per Cent Stone Retained 1 Mesh	Temp A C at Plant Deg F	Pen A C at Tank in Cms	Temp at Street in Deg F	Thickness in Inches	Per Cent Bit Sol in CS_2	Per Cent of Sand Passing Standard Screen: 200 Mesh	80 Mesh	40 Mesh	10 Mesh	4 Mesh	Temp Sand at Plant Deg F	Temp A C at Plant Deg F	Pen A C at Tank in Cms	Temp at Street in Deg F	Thickness in Inches	Number of Tests
[illegible]	[illegible]	[illegible]	[illegible]	38	83	100+	[illegible]	[illegible]4	[illegible]99	10	[illegible]35	70	230	15	128	146	253	234	170	20	330	260	72	300	2	12
[illegible]	99.0	[illegible]	[illegible]	[illegible]	85	10+	6.8	[illegible]	60.0	115	295	73	260	15	108	134	255	245	23.5	20	315	300	75	280	2	20
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	4	75	70	28[illegible]	[illegible]0	1[illegible]	240	60	225		123	145	274	228	20.2	20	300	275	67	275	2	5
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	76	[illegible]	[illegible]	257	718	4[illegible]	[illegible]30	72	240	[illegible]5	112	120	235	246	270	23	320	300	75	310	2	45
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]80	71	[illegible]	15	116	108	258	236	280	38	310	235	65	300	2	10
[illegible]				45											122	125	240	225	275	21	315	255	80	300		18
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		[illegible]	[illegible]	65	[illegible]	[illegible]	[illegible]	240	15	118	129	259	235	238	23	315	270	72	294	2	
[illegible]		[illegible]					5-8	[illegible]	[illegible]	0	[illegible]	[illegible]	[illegible]	15	[illegible]	10-15	18-36	20-50	8-25	0-10	375	325	70-80	250 to 325	2	

Fig. 2. Average Analyses of Standard Sheet Asphalt.

much as where the direct heat was in use, and, further, by the fact that repeated analyses of the material that has been broken up show almost no evidence whatever of any burning of the material, but do show that it lacks asphaltic cement. This results, apparently, from the variation which has prevailed in the bitumen content of the raw material, and the consequent difficulty of determining accurately the amount of refined asphalt needed.

How this pavement will stand heavy traffic and weather conditions is at present indeterminate. With a large yardage laid and a large amount additional contracted for, the results are, naturally, of much concern.

Fig. 3. Rock Asphalt Crushing Plant in Operation in Salt Lake, Utah.

The foregoing analysis of rock asphalt has recently been materially changed, permitting the addition of approximately 40 per cent of sand properly graded and sufficient asphaltic cement to bring the amount contained to approximately 12 per cent, and eliminating the use of any process where the flame comes in direct contact with the mixture.

The above alterations have been very beneficial in more respects than one. A very satisfactory sand grading has been secured—that is, the high percentage of fines naturally contained in the rock have been decreased, being replaced by stone dust or cement to the proper limit, and the percentages of 10 and 40 per cent mesh material have been increased in the resultant mixture. By the addition of asphaltic cement not only was the percentage in the mixture controlled, but the proper penetration that the traffic and other conditions on the street demanded was secured.

The cost of the pavement has been decidedly cheapened, in consequence of which it was possible to lay a 1-in. close binder that was omitted from the straight rock asphalt specifications. The surface of the pavements laid last year under the original specifications are extremely uneven, due to the size of the batch, which of necessity was small, with the type of heater used, and owing to the fact that it was impossible to place the roller until the material was at a temperature that did not permit of satisfactory compression, and resulted in noticeable and uneven joints. The pavements as laid this year are extremely smooth and even, conforming to the street grade and crown.

The use of Utah rock asphalt was accepted this time with qualifications. However, the pavement being laid this year seems justifiable.

Sylvester O. Cameron is city engineer, and from his report and information furnished us by F. N. Huddleston, engineer of streets, the above article has been prepared.

A large working model of the New York state barge canal system has been shipped to the Panama-Pacific exposition from the New York state engineer's office. The model was prepared under the personal direction of Mr. John A. Bensel, state engineer. It will occupy 2,600 sq. ft. of space in the Liberal Arts Palace.

Adaptability and Cost of Concrete and Macadam Pavement Bases in Oakland, California.

Modern pavements ordinarily consist of a distinct base and wearing surface, and these two parts of the complete pavement may profitably be considered separately. In a paper before the Department of Engineers, Councilmen and Street Superintendents of the League of California Municipalities W. H. Frickstad, assistant superintendent of streets, Oakland, Cal., discussed pavement bases in Oakland, and a portion of his paper is given here:

The usual foundation of pavements in general use consists of Portland cement concrete about 6 ins. thick. This gives a foundation that will bridge weak spots in the subgrade caused by backfilled trenches or poor soil. If laid with ordinary care it will outlast any type of wearing surface. Assuming a cost of 20 cts. per square foot for sheet asphalt complete, the foundation costs about 9 cts. in Oakland.

The only substitutes suggested are water-bound macadam and asphaltic concrete. Five or 6 ins. of water-bound macadam, measured after rolling, formerly cost us 7 to 8 cts. per square foot. For the sake of resolving all doubts in favor of the macadam, let us assume that as a foundation 6 ins. of macadam can be produced for 6½ cts. Asphaltic concrete 6 ins. thick would cost us about 14 cts. Therefore, the only choice seems to be between hydraulic concrete and water-bound macadam, the latter ordinarily costing about three-fourths of the former.

Water-bound macadam, thoroughly compacted by months or years of traffic, placed on a perfect subgrade, makes a strong foundation. But if compacted only by rollers and placed on the ordinary surface soil, ranging through all varieties of poorly drained sand, gravel, clay, loam, adobe and mixtures thereof, on a street with three or more main pipe trenches throughout its length, and three service trenches to each lot, it is a poor foundation for an expensive surface.

If the macadam base already exists, of course the financial estimate has a different complexion. Instead of costing 6½ cts., or any other amount, it may seem to cost less than nothing in that it need not be removed. But trouble begins with the construction. The surface of the macadam will be found irregular, with a strong tendency to have a crown too steep for an asphaltic surface. Then, about that time, if the gas company and water company are enterprising or the Council insistent, each service pipe will be inspected and removed. Perhaps a larger main will be laid by one or both companies. Perhaps the Council will be possessed of real forethought and order side sewers to run to every vacant lot. At any rate, when it comes to laying "hot stuff," the foundation is not the foundation that entered into the discussion prior to the adoption of the specifications.

Oakland has had experience with macadam bases covering twenty or more years. Most of the heaviest travel sections have long since worn out. Others are being gradually replaced. Still others have given fairly good service. Whenever we make a repair of the surface or replace the pavement over a trench we put in concrete base. This temporary expedient is increasing the cost of repairs, but it staves off the cost of reconstruction.

Of course there may be cases where it is proper to lay asphaltic surface on macadam base, as on country roads, or in cities that have an alley system, where there is small occasion to break into the pavement. But it must be remembered always that macadam depends for its strength essentially upon the character of the subgrade.

It has been proposed to decrease the thickness of the base to 5 ins. or even to 4. On the basis of 9 cts. for 6 ins. or concrete, 4 ins. would cost about 6½ cts. Four inches of macadam would cost about 4½ cts. on the basis of 6½ cts. for 6 ins. It is impossible to give any conclusive reasons for adopting any certain thickness of base. Pavements with thin bases have successfully withstood traffic for many years. On the other hand, wheel loads and speeds are constantly increasing, and some of the larger cities have already adopted 8 or 9 ins. as the proper thickness for concrete base. Thinking of the base as the element that gives strength to withstand heavy loads, essential to the very existence of the pavement, and if we expect it to outlast several renewals of the wearing surface, it seems wise to be conservative on this question. This is especially true if the surface is to be of an expensive type. It is surely unwise to economize on the base if the surface is to cost five or ten times the possible saving.

Fig. 4. Heating and Laying Rock Asphalt.

Inspection of State Aid Road Construction in Wisconsin.

Inspection of road construction accomplished under the direction of the State Highway Commission of Wisconsin is made difficult by the large number of small projects undertaken, making it uneconomical to place an inspector on each section of road under construction. The methods followed in overcoming this difficulty are outlined by J. T. Donaghey, chief inspector for the Wisconsin Highway Commission, in a paper before the Northwestern Road Congress, and his paper is given here in part.

ORGANIZATION FOR INSPECTION

It was impossible to place an inspector on each individual job, as some states do, for the reason that funds were limited. The work was widely scattered and the appropriations available would not warrant the expenditure. Neither could a sufficient number of competent inspectors be found, for during the season of 1913 no less than 200 jobs were under way at all times, and now it has reached 350 or more. Also the state aid law provided for the election by each county board of a county highway commissioner, who would have direct charge of the construction and who should eventually be competent to direct the greater part of the work in his county.

To give the necessary supervision at the lowest possible cost it was decided by the commission to divide the state into seven districts, each containing from nine to twelve counties. A division office was established at the principal railroad center of each division and a division engineer working under the direction of the chief engineer was placed in charge.

A chief inspector was employed, who, working from the main office at Madison, gives general inspection to the work under the direction of the chief engineer. It is necessary for him to be familiar with the conditions in each county as to materials available, type, quality, cost of work, etc. He is called upon to attend county board meetings, farmers' institutes, good roads and various other meetings, and must be sufficiently informed to talk intelligently upon each local road problem.

Day Labor Work.—The division engineer's duties are varied. Under his direction all surveys and plans are made, relocations and changes in the proposed state aid system are investigated, materials selected, etc. The greater portion of his time, however, is applied to road and bridge inspections. Mon- [illegible] office attending to the correspondence of the week, approving grades and looking after the details of his division. The balance of the week he drives from one job to another throughout the division, inspecting the work under construction, making a final inspection and report of each job as soon as completed.

He has authority to make necessary changes or corrections in the plans, as changes are frequently advisable and mistakes will sometimes occur. He stays on a job only such time as is necessary to determine whether or not the work is being carried on in a workmanlike manner and according to the plans and specifications.

If not, it is his duty to locate the trouble and remedy it before leaving. Also on force account work to investigate the unit costs and determine whether or not the foreman on the job is keeping his accounts correctly. He carries with him a plan of each piece of work and he checks it up from time to time. When weather conditions permit, the division engineers inspect from 15 to 30 separate jobs during the week, often driving 100 miles per day. The greater portion of the roads and a small number of bridges are built by force account, with the county highway commissioner's foreman in direct charge of the work.

Contract Work.—On the contract work a resident inspector is placed by the county highway commissioner, excepting on grading jobs. The inspector is furnished with a copy of the plans, cross sections, contract and specifications, it being his duty to see them properly carried out. He must give the contractor to understand at the outset that he will be compelled to do all that the contract calls for and in the manner specified, no more or less. The inspector who thoroughly understands the plans, contract and specifications and insists calmly but firmly on having them lived up to invariably gets along better with the contractor and secures the best results.

No inspector should allow work to start until the contractor has the necessary equipment and materials on the ground to enable him to carry on the work in a rapid and workmanlike manner. The job that drags along indefinitely for lack of labor, equipment or material is never as satisfactory as is the one carried on rapidly from start to finish, and this applies equally as well to force account work.

The division engineer inspects the contract jobs only often enough to determine whether or not the resident inspector is insisting upon proper results and at times to settle any controversy that may arise between the inspector and contractor. He also makes a final inspection when the job is completed.

General Features.—The state aid law provides for a wide distribution of state aid work. In Wisconsin there are 71 counties, several of which are larger than the state of Rhode Island, containing 1,200 towns, 294 villages and 90 cities under 5,000 inhabitants, in all 1,584 units of government that are eligible to receive state aid for highways. In 1914 there is under construction either roads or bridges in about 1,200 of these units. It would be impossible to secure a competent inspector on each of these jobs and it would be an unnecessary expenditure of money if we could.

Owing to the fact that 75 per cent of the road work is force account, under the direction of the county highway commissioner, there is no incentive to cut down the amount of material used or to slight the work, thereby eliminating the necessity of the close inspection necessary on contract work. The cost of inspection on the 1,306 miles of road was $8,900, and for the 320 bridges $3,576. or about $6.81 per mile for roads and about $11.18 for each bridge.

It is very difficult to estimate the true value of the inspections. However, the fact that the state road and bridge inspectors travel from one town to another and from one county to another, picking up from and disseminating among the various road and bridge crews the latest and best methods of selecting and handling the various kinds of material, road, bridge and culvert construction, also the necessity and use of up-to-date road-building machinery, alone has been worth to the state at large several times the total cost of inspections.

A few pointers for road inspectors: Inform yourself on the plans, contract and specifications before reaching the job, if possible. Look the job over carefully before making any comments. If criticism is necessary, direct it to the man highest in authority on the job, and do so privately. When a decision is asked for that is purely a matter of judgment, give it then and there. Your judgment never will be better than right at that minute. Do not "take sides" in any controversy that you are called on to settle. Investigate the matter thoroughly, then render your decision. Never correct a workman. He is not to blame for following the foreman's orders. Remember you have a decided advantage if able to control your temper when the other fellow loses his. Be firm, but absolutely fair. Remember it may be possible for you to learn something each day, and often from a source least expected. If you have made a mistake, candidly admit it. The men who never make mistakes never do much business.

GENERAL

A New Fuse for Eliminating Misfires in Blasting.

As stated by the author the object of this paper is to bring to the notice of engineers a safety detonating fuse by the use of which misfires in blasting may be eliminated and safety in blasting operations promoted. This new detonator is a French invention and is known as Cordeau detonant or detonating fuse and is sold under the name of Cordeau-Bickford. It consists of a lead tube 5 to 6 mm. in diameter, filled with trinitro-toluene. While applicable to all classes of mining it will appeal especially to those who have to do with deep-hole blasting in open-cut mines or quarries, or any operations where a large number of holes are to be shot at one time. An abstract of the paper which is published by Mr. Harrison Souder in Transactions American Institute of Mining Engineers for October, 1914, is as follows: To summarize, this fuse has three very important qualities (1) It is safe. (2) It is instantaneous. (3) It increases the efficiency of an explosive [illegible]

Safety.—There is no danger in the handling or storage of the fuse. It cannot be exploded by friction, fire, or ordinary shock. It requires the use of a strong blasting cap prop-

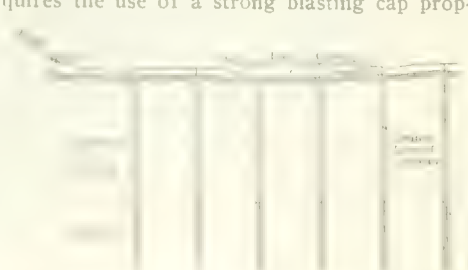


Fig. 1. Diagram Showing Method of Con- [illegible] Holes.

erly attached to explode it. In blasting charged holes, the cap or exploder can be applied outside the hole, thus avoiding the danger of burned powder caused by side spit from ordinary fuse; also any risk of accident while tamping and the risk from a portion of an unexploded charge accompanied by a cap remaining in the debris from a blast is entirely obviated.

Speed.—The average rate of speed of this fuse is estimated to be close to 17,000 ft. per second, so that when it is used the explosive charge is detonated instantly throughout its entire length, instead of at one point as is the case with the blasting cap or electric exploder.

Efficiency.—It is known that the speed of an explosive decreases as the explosive wave travels away from the detonator. That the powder in a hole has the strongest explosive effect around the exploder is evident from an examination of the face of the bank after a shot. This can be demonstrated also by placing sticks of dynamite on the ground end for end, about 6 ins. apart, with the cap in the first stick. The explosive force gradually lessens until it finally ceases to progress, leaving the farthest sticks unexploded.

By using this fuse the charge is detonated instantaneously throughout its entire length. This results in a saving of about 10 per cent of explosives as determined by results obtained. It is not affected by heat, cold, or moisture, and lasts indefinitely without deterioration. It is wound in continuous lengths on spools containing 100, 200, or 300 ft. each, and weighs about 7 lbs. per 100 ft. It is accepted by transportation companies without restrictions except that it shall not be packed with other high explosives.

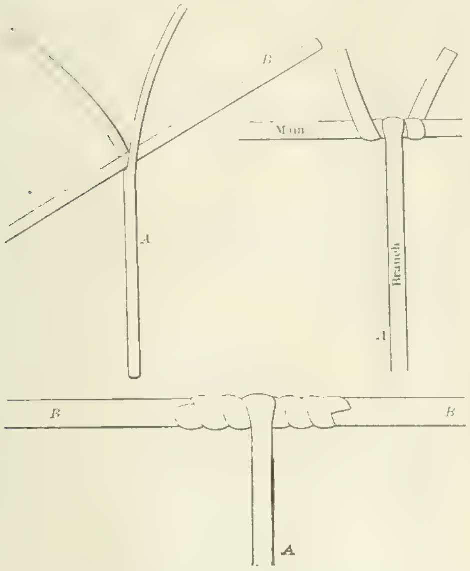


Fig. 2. Diagram Showing Method of Connecting Main and Branches.

The method of applying the fuse is shown clearly by the accompanying sketches. Figure I shows how the holes are connected by the detonating fuse. The method of making connections between main and branch lines is shown in Fig. 2, and that of attaching the detonator to the fuse in Fig. 3. Either ordinary fuse and cap, or an electric exploder, can be used to set off the detonating fuse.

How to Attach a Detonator to the Detonating Fuse.—First, be sure the end of the detonating fuse is freshly and squarely cut off. If ordinary fuse is used, attach a cap by crimping it to the fuse in the usual manner. Next attach a union to the detonating fuse by crimping the end opposite the slit. Insert the cap in the slit end of the union till it comes firmly against the detonating fuse and then slip the ring over it to hold it firmly in place. Be sure the detonating fuse is cut off squarely and that the cap is firmly seated against it. A space of ⅛ in. between cap and fuse may be sufficient to cause a misfire.

Branch line A is joined to trunk line B by twisting the split ends, one to the right, the other to the left of the intersection. These joints may be protected from danger of loosening or breaking by winding with electric tape. This also protects them from moisture. This precaution is not necessary except where many connections have to be made or wet work is encountered.

This operation is not difficult, but it is very important, and should be done carefully and in accordance with three simple rules. (1) Remember that it is the branch fuse that is to be slit and twisted about the main and never the opposite; (2) Be sure to make a firm joint; (3) At the point of intersection, be sure to lead the branch fuse away from the main at a right angle, at least for an inch or two. Beyond that curves and angles do no harm.

If the direction of the explosive wave is carefully borne in mind, there can be as many branches and sub-branches as are desired. To slit the end of the branch line an instrument called a "cordeau slitter" is provided. It can, however, be done easily with a sharp knife. After slitting it for a distance of 3 or 4 ins., separate the legs and, after placing the main snugly in the crotch, wind the legs about it, one to the right and the other to the left. If some of the trinitro-toluene is lost from the split ends, no harm is done provided the main is pushed firmly against the exposed material in the crotch. If many connections have to be made, it is possible that the handling and moving of the fuse may displace some of the connections first made before all can be completed. To guard against such an occurrence it is better to wind all joints with electric tape. This will hold them firmly in place. Before a shot is fired, the line should be thoroughly inspected. See that the branches leave the main at right angles. Be sure that they are all made firm.

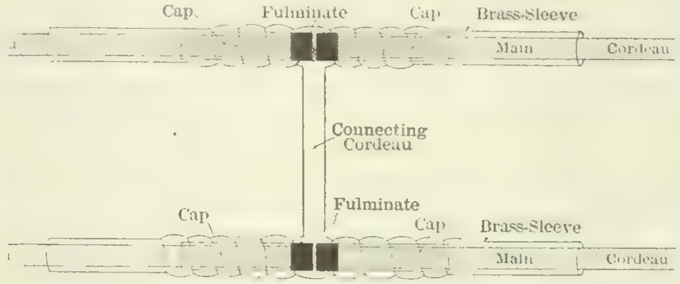


Fig. 4. Diagram Showing Method of Connecting Two Main Lines.

On very large and important work where more than one main is employed, these mains may be joined together as a measure of precaution. This is done in the manner shown in Fig. 4 by cutting the mains and putting caps on the ends and then uniting them again by means of a brass sleeve, the ends of which are firmly crimped to the detonating fuse. Around these brass sleeves the ends of the connecting line are wound in exactly the same manner as when attaching a branch to a main line.

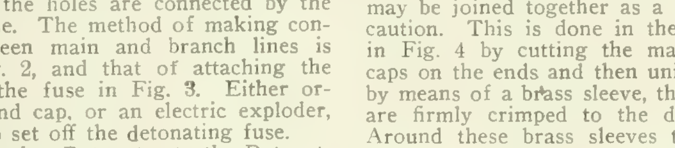


Fig. 3. Diagram Showing Method of Attaching Detonator.

Costs of Drilling and Method of Testing Drill Efficiency at North Star Mine, California.

A method of testing drills which is believed by the author to prove a satisfactory basis upon which to judge rock drills is described in a paper by R. H. Bedford and William Hague in the August Transactions of the American Institute of Mining Engineers.

The rock at this mine is either close-grained diabase in the upper levels, or tough granodiorite in the lower levels. The vein, which has an average dip of 23°, consists of about 5 ft. of "formation" lying between walls of unaltered country rock. Of the 5 ft., solid quartz and stringers make up 18 ins. of pay ore. The formation, which consisted of but slightly altered country rock, is almost as hard as the unaltered walls. From 4 to 4½ ft. is sent to the mill. One-third of the holes are drilled in the quartz; two-thirds in the formation. The holes are about 4½ ft. deep and break on the average of 1.15 tons per hole. The average number of holes per stope drill shift is at present 5.65. The number of drill shifts throughout the mine in 1913 was 18,679, and the cost of labor for drilling, power, supplies, upkeep of machines and air lines, tool sharpening, and distribution, amounted to 33 per cent of total mining expense. To do this work there are in commission 43 No. 12A, three No. 17V and eight No. 16V Waugh stopers, three Jackhamers, two Waugh pluggers, and 17 No. 8 and two No. 7 water Leyners.

For the past three years a record has been kept of the repairs on each machine and number of shifts operated by it. The figures given in Tables I and II for the cost of repairs per drill shift have been taken from this record. The underground air pressure is about 90 lbs. The air consumption per drill shift was measured by aerometers. The leakage of pipe lines between compressor and meter is not included. From the number of pieces of steel sharpened for each type of drill during the year the average used per drill shift has been ascertained. For other supplies the figures given represent a year's average.

The 16V and 17V Waugh drills cost 10 cts. less for maintenance supplies, but require 15 pieces of steel per shift instead of 10, which leaves the cost about equal.

The drilling at this mine is one-third of the cost of delivering ore at the mill. The importance of selecting a suitable drill for the work and keeping it in its highest state of efficiency is obvious. At first, time was wasted and trouble experienced in testing underground drills which were unsuitable. Drills which sounded all right when run in the repair shop failed to work satisfactorily underground. There was need for some reliable testing machine to overcome the difficulties. W. D. Paynter, who was responsible for keeping drills in repair, after experimenting for some time, perfected a testing machine, which he has patented. Briefly it consists of a device whereby the blow of the drill delivered against a plunger is measured by the distension of a diaphragm, oil being the medium of transmission. By means of a lever arm, the movement of the diaphragm is amplified. A pencil on the end of the lever, marking a piece of paper on a revolving drum, gives a graph of the work done by the drill over a given period of time.

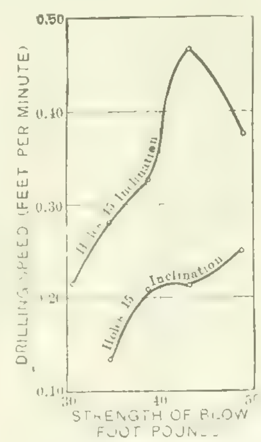


Fig. 1. Curves Showing Influence of Strength of Blow on Drilling Speed; Air Fed Stoper.

The general method of testing a drill consists in first obtaining graphs in the shop and then the drilling speed underground. Having passed the first test, the drill is used in the mine, a record being kept for several months of the footage drilled. Graphs taken from time to time show whether or not the drill is deteriorating, while the card record of repairs gives the cost of its upkeep. Drills sent to the shop for repairs are not returned to the mine until they show on the tester a satisfactory graph.

Table III compares (*a*) three drills striking

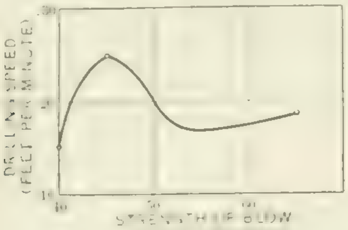


Fig. 2. Curve Showing Influence of Strength of Blow Upon Drilling Speed of Water-Leyner No. 8.

a hard blow, (*b*) two drills striking a medium blow. These were tested in very hard ground with holes at an inclination of 25°, 1¼-in. cross steel with 2¼ and 2-in. bits being used; conditions, however, were kept the same for each test, which lasted 5 minutes.

TABLE III.

	Foot pounds.	Blows per minute.	Drilling speed feet per minute.
(a)	53	[illegible]	0.126
	52	1,320	0.165
	5[illegible]	1,584	0.250
(b)	40	1,260	0.108
	42	2,352	0.195

These results indicate that drilling speed varies approximately with the blows per minute, the strength of the blow remaining constant.

Table IV compares drilling speed with varying strengths of blows. The conditions of test were Ground only medium hard; length of each test, 5 minutes; 1¼-in. cross steel with 2-in. bit; holes at an inclination of 45°; drill used, air-feed stoper.

TABLE IV.

	Blows per minute.	Foot-pounds per blow.	Feet drilled per minute.
(c)	1,272	48½	0.378
	1,222	43	0.447
	1,200	38½	0.308
	1,170	34½	0.250
	1,090	30½	0.188

A test under the same conditions as in (*c*), except that the inclination of the holes was 20°, gave the results shown in Table V.

A test (*e*) was made under the following conditions: Ground hard; length of each test,

TABLE V.

	Blows per minute.	Foot-pounds per blow.	Feet drilled per minute.
(d)	1,272	48½	0.251
	1,222	43	0.206
	1,200	38½	0.198
	1,170	34½	0.135

5 minutes; drill used, No. 8 water Leyner; 1⅛-in. hollow steel with 2¼-in. bit; holes nearly horizontal. In this test varying

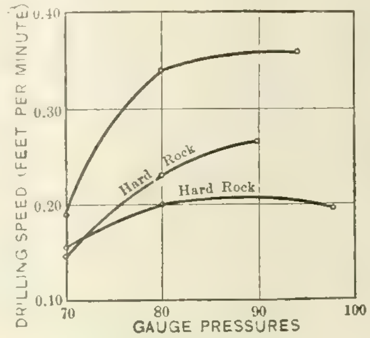


Fig. 3. Curve Showing Influence of Gage Pressure on Drilling Speed.

strengths of blow were obtained by means of stops of different lengths screwed into the ends of the valve chest.

As already stated, Table III indicates that for blows of equal strength the drilling speed is approximately proportional to the number of blows, even when these differ as much as 40 per cent. In constructing the following curves from Tables IV, V and VI, where the maximum variation in the number of blows is 15 per cent, the drilling speed has been arbitrarily adjusted according to the results obtained from Table III, so that the effects of

TABLE VI.

	Blows per minute.	Foot-pounds per blow.	Feet drilled per minute.
(e) (1)	1,368	40	0.162
	1,272	45	0.234
	1,260	52	0.119
	1,212	65	0.129
Repetition of test (1) for confirmation—			
(2)	1,368	40	0.135
	1,272	45	0.224
	1,260	52	0.183
	1,212	65	0.195

the strengths of blow may be comparable. These curves all show the same tendency to flatten at above a certain strength of blow, which in this ground happens to be about 45 ft.-lbs. It will be noticed that the curves for the Leyner and for the stoper at 45° indication are almost exactly similar. This is thought to be due to the fact that in each case the face of the hole is clear of cuttings. In the cases of the flat holes, which do not clear themselves, the drilling speed is not only lower, but does not show the same peak. This suggests that where holes can be cleared of cuttings a 45 ft.-lb. blow is sufficient to obtain the fastest drilling, whereas in holes where the deadening effect of cuttings exists the limit of effective blow is higher. This also emphasizes the desirability of a stoper with water feed to the face of the hole.

From these tests, and others giving similar results, it has been decided that for conditions existing at this mine, stoping drills should strike a minimum of 40 ft.-lbs. As none of these drills has yet been found that strikes over 55, it remains solely a question of maintenance to keep them at this point. The Leyner, with blows as high as 65 ft.-lbs., presents a different problem. In this type the valve was so adjusted, by means of standard plugs screwed into the ends of the valve chest, as to give at average gage pressure blows in the neighborhood of 45 ft.-lbs. It is hoped that this reduction in strength of blow will result in lessened breakage of steel, decreased repair costs, and maximum drilling speed. The adjustment has not been in use long enough to give any figures on the first two points.

The effect of setting the minimum strength of blow in the stoping drills at 40 ft.-lbs. has been to increase the breakage of steel by 1 lb. per drill shift, and the cost of repairs by 27

TABLE I.—COST PER DRILL SHIFT FOR WATER LEYNER NO. 8.

Item.	Description of unit.	No. of units used per drill shift.	Price per unit.	Cost for item per drill shift.
Labor of drilling	8-hr. shift	1	$3.25	$3.25
Maintenance—				
Labor		....	....	0.10
Supplies		....	....	0.62
Power	1,000 cu. ft. of free air compressed to 100 lbs.	15	0.0275	0.41
Supplies—				
Lubricants	Quarts of "Red Engine"	0.66	0.07	0.05
Hose	Feet of 1-in. 5-ply wire-wound	0.12	0.33	0.04
	½-in. 5-ply wire-wound	0.12	0.19	0.025
Drill steel	Pieces used	16.75	....	....
Labor sharpening and repairs		....	....	1.12
Labor distribution		....	....	0.35
Steel consumed	Pounds	2.66	0.125	0.33
Power for sharpener	1,000 cu. ft. of free air compressed to 100 lbs.	5.00	0.0275	0.15
Oil for forge	Bbl. of 42 gals	0.12	1.80	0.21
Upkeep of air pipe		....	....	0.09
Total		....	....	$6.75

TABLE II.—COST PER DRILL SHIFT FOR WAUGH 12-A.

Item	Description of unit.	Number of units used per drill shift.	Price per unit	Cost for item per drill shift
Labor of drilling	8-hr. shift	1	$3.00	$3.00
Maintenance—				
Labor		....	....	0.10
Supplies		....	....	0.37
Power	[illegible]	[illegible]	0.0275	0.47
Supplies—				
Lubricants	[illegible]	0.33	0.07	0.02
Hose	[illegible]	0.12	0.27	0.03
Drill steel	[illegible]	10	....	....
Labor sharpening and repairs		....	....	0.30
Labor distribution				0.21
Steel consumed	[illegible]	2.2	[illegible]	0.14
Power for sharpener	[illegible]	1.5	0.0275	0.04
Oil for sharpener	[illegible]	[illegible]	1.8[illegible]	0.06
Upkeep of air pipe			....	0.09
Total			..	$4.83

cts. per drill shift. The footage drilled, however, has been increased 15 per cent, reducing the cost per foot of hole drilled from 20.3 to 18½ cts. The output per drill shift has increased 10 per cent during the same period, but this figure is complicated by the width of stope broken, and is therefore not quite comparable.

In the case of the Leyner drill, the cost for sharpening, breakage, and repairs make a high cost per drill shift. Comparison with a type of machine in which these items are low is valuable, as it proves that the expense is justifiable. A 5 by 7-ft. drift in this mine requires 11 holes, aggregating 43 ft., to break a round. A 2¼-in. reciprocating drill, striking 40 to 50 ft.-lb. blows, 600 to 800 times a minute, used to require two drill shifts for drilling the round, at a cost of $4.45 per drill shift, or 20½ cts. per foot of hole. The Leyner does this in one drill shift for 15½ cts. per foot of hole.

The use of the tester in the shop has greatly facilitated repairs, as any abnormal action in the drill is disclosed at once. Experiments have demonstrated several points. Among these may be mentioned the effect of feed-barrel pressure. If the feed-barrel packing is slightly defective, the drill does not hold against the ground hard enough, causing as much as 30 per cent decrease in drilling speed. In the 16V stopers, wear in the barrel brushing after about 250 drill shifts causes cushioning due to leakage of air, with reduction in the strength of blow, amounting to as much as 20 per cent. In the 17V type, at pressures below 85 lbs. per square inch, the strength of blow drops very rapidly. Plugging some of the exhaust ports causes the strength of blow to decrease less rapidly. This makes the drill more effective at low pressures.

Examination of Table IV shows the decrease in drilling speed that may be expected with a stoper striking 30 ft.-lbs., as compared with 45 ft.-lbs. On this point it may be stated that many confirmatory tests have been made. Unless the drills are maintained at their best, the strength of blow drops rapidly (sometimes after 100 drill shifts), with resulting decrease in drilling speed. The cost per drill shift, however, remains practically the same.

Experience at Inglewood, Calif., With the City Manager Form of Government.

The commission form of government has spread with surprising rapidity through the United States, following its introduction in Galveston in 1901, and in over 300 cities of this country it is now in force. While this form of government is a distinct advance over the form it has supplanted there is still room for improvement. This improvement will be effected by the adoption of the commission-manager form of government. The present article, which is taken from a paper before the recent annual convention of the League of California Municipalities by Mr. Paul E. Kressley, City Manager of Inglewood, Calif., points out some of the defects of the commission form of government, shows the advantage of the commission-manager plan and gives the results of the application of this plan in his city.

In some cities the commission form has proved very successful, while in others it has been a complete failure. This must not be regarded as a defect in the form of government, but is due rather to the inefficiency of the members composing the commission. In some cities the commisison government has superceded administrations which were not thoroughly inefficient, and where there was not quite such a broad opportunity for improvement as in some other localities. It is in such cities that the weaknesses of commission government are beginning to appear in a very striking way. Take for example Wichita, Kansas, where as elsewhere the commission plan was hailed five years ago as a "business plan" of city government which would settle all the difficulties under which the town had been laboring. Recently the commission in that city authorized the issuing of $88,000 in five year bonds to cover current deficiencies. After four years of the new rule, that does not look like good business. A little searching into the local situation reveals certain things which are directly or indirectly responsible for this condition. One of the members of the commission is an ex-street laborer. This of course, is an honest calling, but can one be sure that it gives a man quite the preparation for managing one of the departments of a city as important as Wichita? There was doubtless excellent reason for electing him, for the labor people had an unquestionable right to select one of their men to represent them in the city's governing body. The man in question had a reputation for integrity and honesty, but even commission government has not made him a good business manager.

Here then, is one trouble with the commission plan which the Wichita incident brought out in a graphic way. It is only by accident that a city can secure five men who will be at the same time perfect representatives of a recognized divison of the people and properly equipped for administrative service.

If these are illustrations of the weaknesses of the commission form of government, what then, is the substantial and permanent contribution of commission government to the city problem? The one great thing it has done is to demonstrate the wisdom of giving over to a single elective governing body all the municipal powers. It has given the people a workable instrument, so far as general regulation is concerned, but it has failed to give the governing body the means with which to translate the general will of the people into detailed acts of government in the most effective way. A five-headed government carries with it the possibility of friction, which leads to inefficiency, which in turn leads to waste and even corruption. Furthermore we can not secure municipal experts by the process of popular election.

Commission government in the old sense, then, is a five-headed affair. It is not always intended to be such, but it is so regarded by the average citizen, and so it works out in practice.

The commission-manager plan does away with this five-headedness of municipal administration and substitutes a single head, not with advisory powers merely but, with certain safeguards, with the powers of administrative "life and death" through actual control of appointments and removals.

The chief executive or city manager is not an elective officer, but is appointed by the council or commission; he therefore does not divide responsibility with the commission, but is subordinate to it. He need not be, at the time of his appointment, a resident of the city but may be chosen from anywhere in the country; he is not chosen for a definite term but holds office so long as he gives satisfaction to his superiors. This plan filters everything through a group. It reduces the personal equation. Without loss of administrative unity, it abolishes one-man power.

Mr. Kressly has been city manager of Inglewood for the past seven months. The section of the city ordinance which defines his powers and duties follows:

1. To see that the laws and ordinances of the city are enforced.
2. To exercise control over all the departments of the city and direct the work of all appointive officers.
3. To employ and dismiss all city employees.
4. To superintend the construction of all public work within the said city.
5. To approve or disapprove the requisition for the purchase of any article or articles for the said city by any department or officer before the purchase is made.
6. To attend all meetings of the Board of Trustees and to recommend to the said board for adoption such measures as he may deem necessary or expedient.
7. To keep the Board of Trustees fully advised as to the financial condition and needs of the city; and
8. To perform such other duties as may be prescribed by ordinance or resolution of the Board of Trustees. Provided, however, that any and all acts of the said city manager under this ordinance shall be subject to the approval and control of the Board of Trustees of the City of Inglewood, and they shall have full power to correct and set aside any action taken by him under this ordinance whenever they shall deem it proper to so do; and provided further, that the legal department of said city and the City Attorney are hereby expressly excepted from the operation of this ordinance.

The duties and powers being thus defined the question arose: What plans are to be adopted and what methods are to be followed to insure the success of the administration?

The plan of selection outlined was predicated upon two ideals, efficiency and economy. These were the basis upon which the superstructure of the organization was erected. We quote from the paper to show Mr. Kressly's procedure:

I determined that the requisites of all employes should be efficiency, economy and loyalty, and that all employes who possess these qualifications should be retained regardless of their political faith. With the requisites thus set forth, I formulated plans and analyzed the problems that confronted me, to find the best way, not by judgment alone, but by careful, thorough and painstaking investigations of all the details of the various departments, thereby familarizing myself in a general way with the work and enabling myself at once to detect any lack of efficiency and to correct the same by setting up tentative standards of performance, by better methods of performance, by inducing the employes to attain these standards, or by equipping them with a clear, complete and exact knowledge of the best and most expeditious way of doing the work with the least possible expenditure of material, capital and labor.

On account of having served as city engineer for several years, I was familiar in a general way with the weaknesses of the various departments and was therefore in a position to devise measures which would eliminate these ideas and practices, by requiring all employes strictly to observe office hours, to get permission before leaving their regular duties, to see that no supplies, materials, machinery, tools, etc., were purchased without an order signed by the city manager, and by requiring all employes to fill out daily time cards, showing the number of hours worked, together with a description of such work and the correctness thereof verified by the O. K. of the head of the department, and turned in daily to the city clerk. These rules are strictly enforced and it is impressed upon the minds of all employes that their indefinite retention in office depends solely upon the efficiency and skill with which their work is performed.

Formerly there was a great deal of waste in the purchasing of supplies, due to a lack of a proper system. Employes would purchase supplies regardless of cost, and have it charged to the city, and when the bills were presented, frequently no one remembered the purchase. This was a very lax method and gave the city clerk no opportunity to check the amounts and charge them to the proper account. Furthermore supplies and materials were purchased whether they were needed or not, and the various departments continued incurring expenses by purchasing supplies, machinery, etc., until the city treasury was drained.

This condition was controlled by the adoption of a budget, thereby limiting the expenditures of all departments within the actual revenues of the city, and requiring departments to submit at the end of every month a detailed report which enables me to follow closely the activities of all departments. An accounting system was installed which provides balance sheets of all accounts, thus enabling the city clerk to furnish me with a complete report of all departments in a very short time. In the departments where it was practical a unit cost system was installed for the purpose of comparison, and fixing standard methods of performance.

I wish to call attention to another important change introduced in the matter of handling complaints. Formerly, when a citizen objected to something done by a city officer, he would go to one of the trustees and complain, frequently with the injunction that his name was not to be mentioned; then the trustee would bring the matter up at a board meeting, and the official, not knowing who made the charge, would be in a very bad position when it came to defending himself. I made a ruling that all complaints of any nature must be made in writing, containing the signature of the complainant, so that the employe will have full knowledge of all details of the charges and can therefore defend himself properly. This procedure has reduced the number of complaints to a minimum, and it makes the employes feel that they are receiving a square deal and [illegible] them to take a greater interest in their work than when continually subjected to unjust criticism.

In order to develop harmony and departmental co-operation I arranged for regular conferences to be held with the heads of the various departments, where all important matters are considered and the programs and activities of each department are discussed and planned.

These conferences have aroused the interest and enthusiasm of every employe, and today all are striving to place the business affairs of our city on a higher plane. To give an idea of our success, I will state the percentage of reduction in the expenditures of the various departments for the past six months compared with the expenditures for the same period during the past three years. It is as follows:

Source of expenditure.	Per cent saving.
Printing and supplies	31
City hall maintenance	[illegible]
Fire Department	[illegible]
Police Department	21
Street cleaning	[illegible]
Recorder's Department	30
Street maintenance and repairs	28

The revenues of the city have been increased considerably by enforcing ordinances and by greater vigilance with peddlers, etc., some of the most important being:

Source of revenue.	Per cent increase
Peddling licenses	12
Dog licenses	[illegible]
Fines for speeding	[illegible]

Receipts from renting out road machinery have averaged $90 per month. The building, plumbing and electrical inspector formerly received all fees collected. This official has been placed on a salary and the receipts of this department now add a substantial amount to the revenue of the city.

With the commission-manager plan of municipal government many things are possible, because the manager can keep operating departments running in harmony, thus cutting out waste of time and effort. The various city departments can be made to assist each other in numerous ways. Take for instance the offices of street superintendent and city engineer; where heretofore these two departmnts were separate they are now consolidated into one. The engineer who designs the work can now also supervise its construction and appoint his own inspectors. The result is better work in every respect, and the engineer can see that his designs are properly executed. This proceedure is especially effective in small cities where the salary of the street superintendent is usually not sufficient to obtain the services of a man who is qualified to supervise the construction of new improvements, but who is sufficiently well qualified to supervise the cleaning, maintenance and repairs of streets, etc., under the direction of the engineering department.

I have frequently been asked the question: From what profession or calling should the important position of city manager be filled? My answer is that city managers should be civil engineers, preferably with municipal engineering experience. A city manager should also possess the additional qualifications of the economist, the fiancier and the executive. In answering in this manner I feel that I am fully justified in favoring the members of my profesion because they are recognized as an important factor in municipal administration. What other profession has men who are better qualified as municipal experts than the engineering profession? Are not the members of the engineering profession made peculiarly qualified for executive duty, in the course of their education, field work, and research work?

Engineering is perhaps as much a matter of common sense as of education, and the engineer to succeed must have both. Therefore a capacity to get at the facts in any case, supplemented by good judgment, are predominating traits, characterizing this profession more prominently as essentials than is the case with any other profession or calling.

Mr. Kressly wonders whether even those American cities which have progressed as far as the commission-manager plan, have yet got far enough away from past traditions and practices to keep any one city manager long enough in office to give either him or the plan a fair test. He expresses the hope that some of the cities which have adopted the plan will give it a fair trial.

Rapid Excavation of the Slide of October 15 at Culebra Cut, Panama.

The dredges at Culebra in six days' operations removed 119,600 cu. yds. from the slide of Oct. 15 which closed the cut to navigation and imprisoned a number of vessels in the canal. The "Canal Record" gives the following account of the work:

A large slide on the east bank of Culebra Cut, directly north of Gold Hill, moving between 5 p. m., Wednesday, Oct. 14, and 9 a. m., Thursday, Oct. 15, carried about 725,000 cu. yds. of earth and rock into the Canal prism, and blocked the channel for 1,000 ft. to the passage of vessels larger than towboats, causing the suspension of traffic. This occurred two months, to the day, after the official opening of the Canal to commercial traffic. The big dredges were working at Cucaracha slide, south of Gold Hill, at the time the slide started, and with commendable foresight on the part of Messrs. Comber and Macfarlane steps were taken to move them northward; but for this action the results secured could not have been accomplished, as the condition of the channel was such that loaded barges could not be passed through the slide. The dredges were worked under triple shift. The 15-yard dipper dredges *Gamboa* and *Paraiso*, and the 5-yard dipper dredge *Mindi*, began excavation at the north side of the slide during the night of Oct. 14-15, while the mass was still in motion. The dipper dredge *Cardenas* was added to the fleet on Oct. 15, and during the 16th an extension of discharge pipe was laid from a point opposite Cucaracha slide, which allowed the pipeline suction dredge No. 86 to take part in the work.

The channel was cleared sufficiently by noon of Tuesday, Oct. 20, to allow the passage during the afternoon of seven vessels, which had been delayed at the south end of the Canal. The vessels had been taken to Pedro Miguel Lock and moored alongside the approach piers and within the lock. The actual passage of the seven through the cut occupied about four hours; they followed each other at intervals of about half an hour, propelled by their own power, but passing the slide under the control of a tug fore and aft, to hold them to the course. Thirteen vessels were awaiting passage from the Atlantic entrance; nine of them were transferred to the anchorage basin in Gatun Lake, south of Gatun Locks, on Oct. 20, and are being handled through the cut today.

The slide which blocked the Canal is 2,100 ft. long, and broke back about 1,000 ft. from the center line of the channel. It occurred in a part of the bank formerly involved in the east Culebra slide, and is designated as the "New Culebra" slide. The old slid appeared to be dead, and the channel opposite it had been cleared to almost full width and depth. At the present time, no further movement is indicated, but practically all of the 725,000 cu. yds. involved in the movement of Oct. 14-15 will have to be dredged from the prism, which may cause further motion. The material is masses of rock and earth, in about equal proportions.

Following the preliminary survey made on the morning of Oct. 15, after the slide had come to rest, it was estimated that, unless further movements occurred, the channel could be opened in one week. A still later estimate set Oct. 22 as the first day on which vessels could pass the slide. The performance of the dredging forces in clearing the channel in two days less time than was believed to be necessary indicates the fitness of the equipment and the loyal persistence of the personnel.

The excavation accomplished by the dredges at the base of the slide, in periods ending at 8 a. m. of each day named, was:

Dredge	15th	16th	17th	18th	19th	20th	Total.
[illegible]	3,935	5,160	6,175	6,910	7,800	5,775	35,755
[illegible]	[illegible]	[illegible]	7,625	7,600	6,720	4,975	37,970
[illegible]	[illegible]	1,000	2,403	2,709	1,640	2,137	11,734
[illegible]		[illegible]	3,251	2,031	3,210	2,052	13,571
No. 86		..	3,825	5,850	6,250	4,650	20,575
Total	[illegible]	16,087	23,279	25,100	25,620	19,589	119,605

The maximum day's output was 25,620 cu. yds., in the period ending at 8 a. m., Oct. 19, or Sunday, when the dredge and towboat crews appeared for duty, without orders or instructions.

BOOK REVIEWS

Steel Construction. By Henry Jackson Burt American Technical Society, Chicago, Ill [illegible]

[illegible] and formulas needed in designing the structural steel framework for building. The application of the formulas is explained by the solution of problems illustrating current practice in steel-frame building construction. The [illegible] and as a reference for designers, although it should best serve the former purpose. A feature of the book is [illegible] and detailed explanations in connection with [illegible] steel. The procedure followed and the data given on this design should prove of value both to students and to practicing engineers interested in steel skeleton construction. It is assumed in the discussion that the reader has mastered such subjects as "Strength of Materials," "Structural Drafting," "Statics," and "Roof Trusses."

The author devotes 15 pages to the procedure followed in manufacturing steel from iron ore; 20 pages to the properties of steel sections and to general information concerning them; 11 pages to a discussion of the composition and properties of structural steel; and about 20 pages to data on rivets and bolts and to information on rivet spacing and driving. Under the heading "Beams," 45 pages are given to a discussion of beam design, calculation of load effects and of resistance, practical applications and details of construction. To a discussion of riveted girders the author devotes 39 pages, treating the theory of design, design of plate girders and of other forms of riveted girders, practical applications and details of construction. In the 60 pages of discussion and data on steel and cast iron columns the principal items considered are strength, column sections and details of construction. Tension members (hangers) are very briefly treated. The discussion of wind bracing comprises 30 pages, the subjects considered being: general conditions, systems of framework, design of wind bracing girders, combined wind and gravity stresses, and effect of wind stresses on columns. The design of a 16-story fireproof hotel covers 74 pages of text, the subdivisions considered being: fireproof specifications, loads, type of floor construction, framing specifications, design of steel members, column pedestals, wind bracing, miscellaneous features, and dimensioning drawings. The protection of steel from rust and fire is also discussed briefly. The text closes with some general instructions covering the writing of specifications and a set of specifications offered as a guide.

Plane Surveying.—By William G. Raymond American Book Co., New York. Flexible leather, 5x7 ins., 590 pp., illustrated. $3.

This book, now in its second edition, is in pocketbook form and has been almost entirely rewritten and rearranged. There are 16 chapters and 8 appendices. The chapter headings are as follows: Measurement of Level and Horizontal Lines; Vernier and Level Bubble; Measuring Differences of Altitude, or Leveling; Determination of Direction and Measurement of Angles; Land Survey Computations; Some Surveying Problems; Some Special Office Instruments; Stadia Measurements; Meridian, Latitude, and Time; General Surveying Methods; Curves; Topographic Surveying; Earthwork Computations; City Surveying; Hydrographic Surveying and Mine Surveying.

The titles of the eight appendices are: Examples and Problems in Surveying; The Judicial Functions of Surveyors; The Ownership of Surveys; Geographical Positions of Base Lines and Principal Meridians Governing the Public Surveys; United States Public Land Surveys; Three-Point Problems With the Plane Table; Field Method for Determining Refraction; and Tables. There are 19 tables.

The book was written for use in the classroom and in the field. While well suited to both uses, it is particularly valuable as a textbook for engineering students. The matter it contains is logically arranged and clearly and concisely stated. A feature worthy of note is the large number of problems stated in various parts of the book. These seem well selected for the purpose of illustrating the character of computations made in plane surveying, and should form an excellent basis for note-book work in connection with classroom instruction.

The book is well illustrated by cuts of field and office instruments taken from manufacturers' catalogues.

The Microscopy of Drinking Water.—By George C. Whipple. John Wiley & Sons, New York. Cloth; 6x9 ins.; 450 pp.; illustrated. $4.

This work, which is now in its third edition, is intended primarily for superintendents of water works who are in charge of storage reservoirs, water works engineers and students of water analysis. The first part of the book has been entirely rewritten, new material has been inserted in nearly every chapter and several new chapters have been added. Among the new chapters the most important relate to the copper sulphate treatment of stored waters, the stripping of reservoir sites, the purification of water containing much algae, and the use of microscope and photomicrography. The last named chapter was prepared by Dr. John Bunker of Harvard University. Dr. Bunker also prepared the plates which show in colors the common organisms found in water supplies. The book is divided into two parts. The first part contains 18 chapters and discusses sanitary problems. The second part contains 12 chapters and is devoted to a technical classification of the microscopical organisms found in water supplies. The book is fully illustrated. This work should be in every water works engineer's office. It has been greatly improved in its revision and takes rank among engineering classics.

Concrete Roads and Pavements.—Revised edition. Cloth, 5x7½ ins., 338 pp. By E. S. Hanson. The Cement Era Publishing Co., Chicago, publishers. $1.50.

The revised edition of this useful compilation and discussion of recent literature on concrete road construction includes several chapters on subjects not treated in the former editions and more than 100 new pages have been added to the book. Other subjects briefly treated in the first edition have been expanded into full chapters. The advantages of concrete as a road material are fully discussed and a new chapter included on the promotion of concrete roadways. The methods and costs of concrete road construction are fully treated, the various steps being described and complete abstracts given of important descriptive articles that have appeared in technical publications. Concrete culverts and bridges are also discussed. The specifications and appendix have been revised to conform with present standards.

Report of the State Commission of Highways of New York.—1913. Two volumes, cloth, 6x9 ins., 602 and 219 pp., illustrated.

The Annual Report of the New York Highway Commission for 1913 is contained in two volumes. The first volume consists of a report on the work of the commission in the various countries of the state, expenditures and recommendations for the ensuing year. The second volume is of unusual interest giving in a readable and well illustrated form recent road practice and experience in Europe presented by the late W. De H. Washington, delegate to the Third International Road Congress.

Strength of Materials.—By H. E. Murdock. John Wiley & Sons, Inc., New York. Cloth; 4⅞x7¼ ins.; 341 pp.; illustrated. $2.00.

In this, the second edition of Mr. Murdock's book on the "Strength of Materials," the author has revised and enlarged the text. The present edition follows the same simple treatment of the subject as did the first—without the formal use of the calculus. The book is designed to give a fairly complete course in the subject to students who have not had the calculus or to those who prefer graphical representations. Although the aim has been to emphasize the elementary principles and to develop independent reasoning in the student, the ground covered is that usually given in college courses for engineering students. The book meets a need for a simple treatment of this important subject.

Specifications for Vitrified Brick Street Pavements and Vitrified Brick Highways.—Paper, 6x9 ins., 18 pp., illustrated. Distributed free by the National Paving Brick Manufacturers' Association, Engineers' Bldg., Cleveland, Ohio.

This pamphlet is a revision of the Directions for Laying Vitrified Brick Pavements issued in 1911 by the National Paving Brick Manufacturer's Association that has had a wide circulation among those interested in street pavements.

The new pamphlet presents in a readable and attractively illustrated booklet the recommendations of a committee of the Brick Manufacturers' Association with regard to the laying of brick on city streets and country roads.

The following subjects are treated: Grading, Drainage, Stone Curbing, Concrete Foundation, Cushion, Expansion Joints, Brick, Brick Laying and Inspection, Rolling, Cement, Grout Filler, Cement Filled Brick Pavement Without Curb, Crushed Stone Foundation, Old Gravel or Macadam Foundation, Paving Brick Foundation, and Sand Filler.

Reports and Bulletins of Interest to Engineers.

Prevention of Accidents From Explosives In Metal Mining.—Miners' Circular 19, Bureau of Mines, Washington, D. C. By Edwin Higgins. Pamphlet, 6x9 ins.; 16 pp.; illustrated. Free on application.

Some facts about dynamite, storage of dynamite, handling of explosives, blasting caps, dynamite, making the primer, charging and firing, misfires, thawing dynamite, blasting by electricity, electric machine, electric detonators, operation of electric blasting machine, black powder.

Production of Explosives in the United States.—Technical Paper 85, Bureau of Mines, Washington, D. C. By Albert H. Fay. Paper, 6x9 ins.; 16 pp. Free on application.

Classification of explosives. Tables: amount of explosives manufactured, amount of black blasting powder used, amount of high explosives used, amount of permissible explosives used, and mine fatalities due to explosives, short flame explosives used in coal mining, amount of coal produced per pound of explosive, permissible explosives used in different coal fields.

Statistics of Gas and Electric Companies.—Report of Public Service Commission, First District, New York. Cloth, 6x9 ins.; 600 pp. Free at discretion.

Gives the returns filed by gas and electrical corporations (including the so-called electrical conduit companies) for the year ended Dec. 31, 1912. An abstract of each return appears in Part III in its alphabetical order and contains the essential items relating to the operations of the year and the condition of the property at the close of the year. Financial data are presented in the abstracts in comparative fullness, while operating statistics are presented more fully in the tabular summaries of Part II. An interpretation of the results of tabulation and compilation is presented in the analysis of Part I, where their significance is brought out by means of comparisons with previous years.

Stream Gaging and Spirit Leveling Records, New York.—Report of Bureau of Hydraulics, Barge Canal Department, Office of State Engineers, Albany, N. Y. Cloth, 6x9 ins.; pp. 458; illustrated. Free at discretion.

Describes methods and tabulates results of stream gagings for 1913, 281 pp. Gives results of spirit leveling from 1906 to 1911, pp. 287-458.

Chicago Traction.—Report of Board of Supervising Engineers, Chicago. Cloth, 6x9 ins.; pp. 259; illustrated. Free at discretion.

Summary of financial and operating statistics and report of chief engineer on construction, renewal and improvement of track, rolling stock, buildings, tunnels, power plants, etc.

Origin of Coal.—Bulletin 38, Bureau of Mines, Washington, D. C. By David White and R. Theissen. Paper, 6x9 ins.; 390 pp.; illustrated. Free on application.

Covers geologic relations of coal, physiographic conditions attending the formation of coal, rate of disposition of coal, regional metamorphism of coal, origin and formation of peat, microscopic study of coal.

Weights and Measures.—Definitions and Tables of Equivalents, Kansas City Testing Laboratory, Kansas City, Mo. Paper, 6x9 ins.; 16 pp. Free at discretion.

Purposes to show in concise form all ordinary relations and one common relation of all units of measurement usually encountered in the different branches of applied scientific work.

Illinois River.—Physical Relations and the Removal of the Navigation Dams, with Supplement on the Waterway Relations of the Sanitary and Ship Canal of Chicago. Sanitary District of Chicago, Chicago, Ill. By Lyman E. Cooley. Paper, 6x9 ins.; 121 pp.; illustrated. Free on application.

Sets forth the pertinent facts relating to the removal of the navigation dams in the lower Illinois River, and increasing the capacity of the stream bed between Utica and the Mississippi River.

Progress of Stream Measurements in Canada, 1912.—Department of the Interior, Ottawa, Ontario. By P. M. Sauder. Paper, 6½x9½ ins.; 460 pp.; illustrated. Free at discretion.

Describes scope of organization for and methods of conducting stream measurements followed by tabulations of records of measurements conducted in 1912.

Report on Irrigation.—Department of the Interior, Ottawa, Canada. Paper, 6½x9½ ins.; 170 pp.; illustrated. Free at discretion.

Gives annual reports of commissioner and superintendent of irrigation, reports of engineers, inspectors and hydrographers, statistics, etc.

Condensed Catalogs of Mechanical Equipment.—A Collection of Catalog Data Concerning the Products of Manufacturers of Mechanical Equipment. The American Society of Mechanical Engineers, New York. Cloth, 6x9 ins.; 307 pp.

This is the fourth issue of this publication. The purpose is to present in condensed form the principal facts and details concerning various classes of mechanical equipment (1) as a convenient index to individual catalogs issued by manufacturers, (2) as a means of comparison between various makes of the same line of goods.

Relative Resistance of Various Conifers to Impregnation With Creosote.—Bulletin 101, U. S. Department of Agriculture, Washington, D. C. By C. H. Teesdale. Paper, 6x9 ins.; 43 pp.; illustrated. Free on application.

Structure of conifers, experimental methods, materials used, method of application, penetration as affected by wood structure, grouping of species, theory of penetrance, records, conclusions.

Thermal Properties of Steam.—Bulletin 75, Engineering Experiment Station, University of Illinois, Urbana, Ill. By G. A. Goodenough. Paper, 6x9 ins.; 69 pp.; illustrated. Price, 35 cts.

Thermodynamic relations, review of earlier investigations, development of general theory of the properties of steam, resume of formulas, conclusions.

Illuminating Power of Kerosene.—Bulletin 37, Engineering Experiment Station, Iowa State College, Ames, Ia. By William Kunerth. Paper, 6x9 ins.; 31 pp.; illustrated. Free at discretion.

Introduction: historical review, object, scope and reliability of tests, quantity of oil obtained, apparatus used—illuminating power: how determined—table of results—relation of cost of illuminating power—fogging of chimney—illuminating power and charring of wick—odor and illuminating power—density and its relation to illuminating power—flash point and its relation to illuminating power—burn point and its relation to illuminating power—viscosity and its relation to illuminating power—surface tension and its relation to illuminating power—optical proprties: red oil, index of refraction, effect of light on the illuminating power—color of kerosene flame—burning quality—illuminating power and draft—effect of air on the illuminating power of an oil—B. T. U.'s—the kerosene oil lamp as an illuminant—the kerosene oil lamp as an illuminant—summary.

Diaphragm Method for the Measurement of Water in Open Channels of Uniform Cross-Section.—Bulletin, Engineering Experiment Station, University of Wisconsin. By C. R. Weidner. Paper, 6x9 ins.; 72 pp.; illustrated. Price, 25 cts.

Descriptions of apparatus and methods of measuring at three European installations—Swiss Bureau of Hydrography experiments to determine accuracy—conclusions.

CONSTRUCTION PLANT

MACHINES DEVICES MATERIALS

The Application of a Power Scraper to Gravel Pit Excavation.

An economical method of excavating packed gravel without the use of expensive plant has long been sought. The layout of a successful gravel excavating, crushing and screening plant is clearly shown in Fig. 1. This plant is owned and operated by Patrick Clark at Ottawa, Illinois, and consists of a ½-cu. yd. Sauerman power scraper operated by a 50-

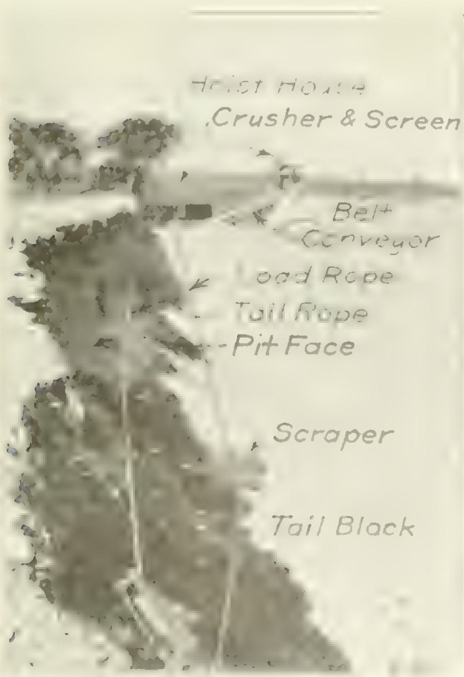


Fig. 1. Power Scraper Operating Along Pit Runway.

HP. Fairbanks-Morse electric hoist, a belt conveyor delivering to the crusher, a crushing screening plant and storing bins delivering into railroad cars.

[illegible] delivering it to the belt conveyor, illustrated [illegible] of the hoist tightens the tail or pull back rope drawing the scraper into the pit. The load rope is tightened by the other drum and the scraper travels toward the conveyor hopper digging its load. At the conveyor the material gathered by the scraper is dropped on the belt, the scraper being of the bottomless type. The tail rope is then tightened and the process repeated. The scraper is simply pulled back and forth over the material to be excavated. The position of the guide block through which the tail rope operates is shifted slightly when a change in the location of the scraper runway is desired.

Ample power is desirable in operating the digging outfit. With a ½-cu. yd. scraper, a 50-HP. hoist as described, or a 9 x 10 in. double cylinder standard double drum steam engine with ample boiler capacity, are about the right sizes to use.

The Sauerman bottomless power scraper is built with two heavy side plates and a back plate riveted to these side plates. A renewable cutter edge is fastened on a runner frame which is pivotally and adjustably connected to two hinge plates which are bolted to the back end and lower side of the side plates of the scraper. The cutter edge, being pivotally and adjustably connected, can be readily adjusted for digging the different kinds of material and soil. The scraper is substantially reinforced throughout. The sides of the bucket are equipped with renewable wearing strips. These wearing strips can be easily taken off and put on by means of bolts. In case of hard digging the cutter edge can be equipped with teeth. The scraper is built in five standard sizes: ½, ¾, 1, 1½ and 2-cu. yds. capacities and is sold by Sauerman Bros., 322 S. Dearborn St., Chicago.

A New Method of Tin Lining or Coating.

(Contributed.)

Epicassit is a German material which serves to protect metal against corrosion. It consists of pure tin powder reduced to dust and a liquid deoxidizer. These are mixed to the consistency of a thick paint. The metal surfaces to be coated should be carefully cleaned, painted and then heated by any suitable means, as over a charcoal fire, or by a powerful blow torch, etc., to the [illegible] The coating will run no more than does paint, thus making possible the tinning of vertical or even of inverted surfaces. Articles of any size, weight or wall thickness whether steel, wrought iron, cast iron, copper or

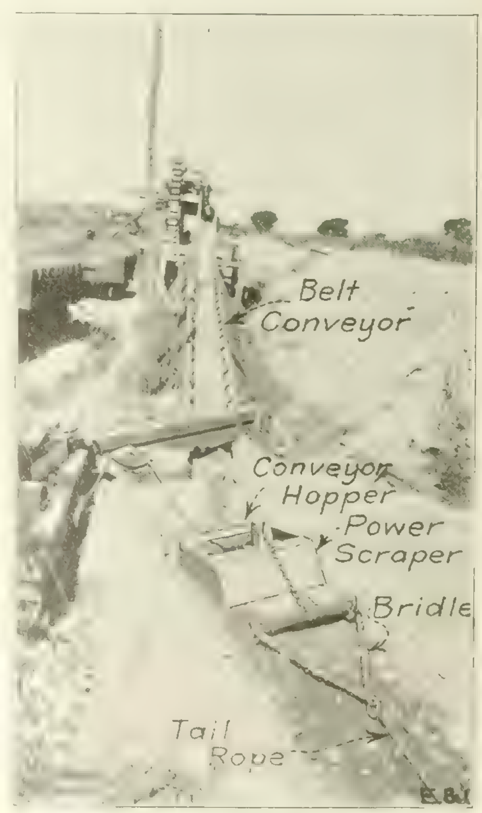


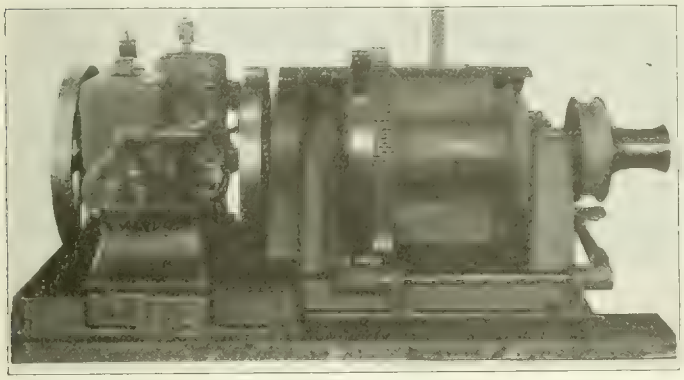
Fig. 2. Power Scraper Dumping at the Conveyor.

brass, etc., may be tinned without dismantling or moving. The usual loss of material attending the ordinary methods of applying metal coatings is avoided as there is no oxidation or drip waste. A coat may be applied to one side of an article only, at a saving of one-half the metal in dip coating and in addition to this coatings which have been locally damaged can be promptly and easily repaired with Epicassit. Epicassit is not a new material, though new to this country. As stated above it was compounded in Germany and has been employed for many years by makers of machines and apparatus for preparing food, and in hospitals, bakeries, hotels, restaurants, creameries, dairies, breweries, distilleries, military barracks, etc., etc. For articles coming into contact with food Epicassit brand "A" is used as it contains pure tin without any alloy of lead or zinc. Brands "B" to "E" for other uses contain tin, lead and zinc. This material is now made available here in the United States in five brands to suit various uses by Hess & Son, 1033 Chestnut St., Philadelphia, Pa.

An Improved Large Size Gasoline Hoist.

(Contributed.)

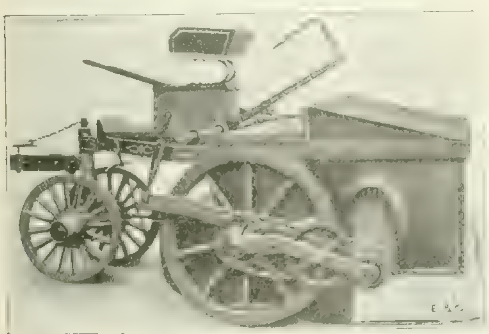
On many smaller contracts, gasoline hoists have been preferred because of their light weight, the ease with which their fuel could



Gasoline Hoist for Contractors' Use.

be transported to them and their low water consumption. In the larger hoists, from 6 to 10 hp., however, the power produced by the ordinary gasoline engine is irregular and the weight is still great.

The latest type of hoist to be placed on the market, endeavors to remedy these troubles. It is equipped with a two-cylinder opposed gasoline engine, which gives a steady, even power. The engine is hopper cooled and requires only a couple of buckets of water a day to keep it from becoming overheated. This type of engine runs practically without vibration; consequently more of the fuel is used up in useful work and less fuel is required to do the work. This type of engine is built by the Heer Engine Co., of Portsmouth, Ohio, in six sizes, equipped with 6, 8, 10, 12, 14 and 16 hp. engines.



Operating Mechanism of the Baker Pick-Up Street Sweeper.

The power of the engine is transmitted in either direction by the use of strong clutches and a large brake controlled by a ratchet lever, which holds the load when the clutches are disengaged. Any load which has been lifted by the cable drum may be lowered rapidly by using a foot brake. Contractors generally use the outfit in connection with double platform elevators. A winding drum is provided to carry the elevating or hauling cable and a sheave for carrying the cable to operate the elevator. This is generally fixed so that one elevator goes up while the other comes down, so that power is required to hoist only the actual load. Thus one platform is at the bottom being loaded while the other is at the top being unloaded. As shown in the specifications, the hoist may be equipped in any of these gears, 15 to 1, 12 to 1, and 9 to 1. After the engine is started, no further attention is required. With the oil cups, gasoline tank and hoppers filled, all the operator has to do is to throw the lever, controlling the clutches of the hoist. When a heavy load is thrown on, the throttle governor with which the engine is equipped opens up and allows more fuel to enter the cylinders.

A Simple Pick-Up Street Sweeper.

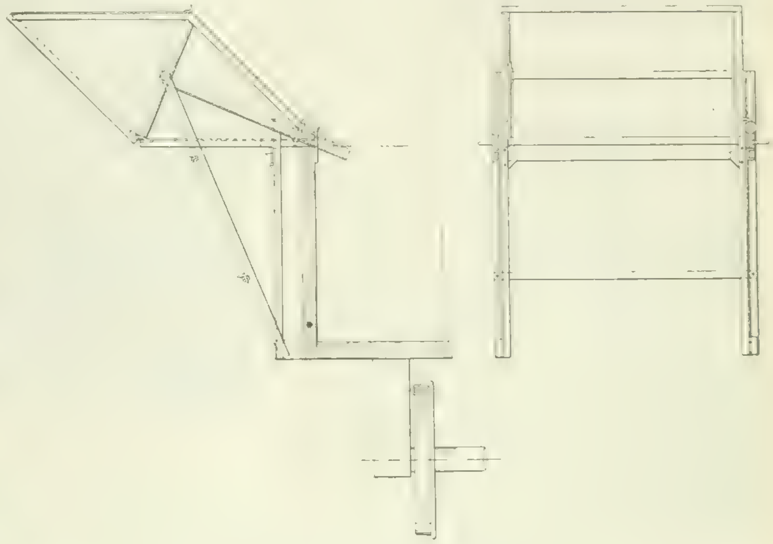
Many city street cleaning departments—especially those in small cities—have frequent use for a light street sweeper that picks up and carries with it to a convenient dumping point the street refuse it encounters. The sweeper illustrated is designed to meet these requirements, being light in weight, about 2,000 lbs., and operated by two horses and a driver. The machine is designed to operate close to the street curb, a special guide wheel (not shown) being provided to prevent damage to the curb.

The main frame of the sweeper is made of steel bars 3x⅝ ins., passing completely around the broom case and uniting in front with the truck casting. The driving wheel is 32 ins. high, with 2x3-in. rim and 3x½-in. tire. Broom is 7 ft. long and when new 24 ins. in diameter. Brooms are rope wound and made of any fiber desired, for general use a combination of brass and bamboo being recommended. Front wheels are 24 ins. high, with 16 large spokes and extra heavy sarven hubs. The rims are 2x3 ins. The end plates in front of broom case and above dust pan are cast iron 1 in. thick, thus giving great strength and rigidity. The dust pan and broom case are made of 10-gage sheet steel, reinforced by angle steel. The gutter wheel is solid cast iron 1 in. thick, keyed to the axle. Compensating gear wheels allow this wheel to drive the broom when the other wheel is standing still. Thus the machine sweeps on a curve as well as in a straight line. All gears have long bearings and are well equipped for lubrication.

The sweeper is manufactured and sold by the Baker Mfg. Co., 506 Stanford Ave., Springfield, Ill.

A Device for Loading Wagons from Cars.

The rapid wagon loader illustrated is a device for the rapid and economical loading of sand, gravel, crushed stone, coal, coke, cinders and other material into motor trucks, traction drawn wagons, dump wagons and other vehicles, from railroad cars, gravel pits, cinder piles, coal yards and other sources where the material to be hauled can be shoveled by hand or elevated by mechanical means into the loader. It is so designed that it tips easily and returns to position automatically by gravity. It can be placed so that it will clear any standard vehicle, when it is used to load wagons from railroad cars. It is in two parts and is light enough so that two men can handle it readily. It requires only the handling of about 300 cu. yds. of crushed stone or gravel out of railroad cars to pay for one loader by the saving which it will make in team time over the old style methods of holding the team while shoveling into the wagon. It also saves time shoveling over any other method or device, because the

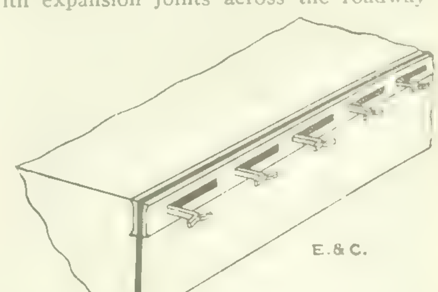


Rapid Wagon Loader for Loading from Cars.

shovelers can always see where they are pitching their shovelful; and do not waste time or "soldier" by looking over the side of the car to see if they are pitching into the wagon. This feature alone also prevents a large percentage of the waste on the ground and consequent reshoveling and cleaning up. Four men shoveling into the rapid loader will handle as much material as six men will handle otherwise. It can be easily used for the quick loading of motor trucks and traction-drawn wagons from an ordinary railroad gondola or coal car. Made entirely of steel, it is practically indestructible, while at the same time it is as light as consistent with proper strength and durability. Being made in two parts, which are separable, it is easily and quickly put in place or taken down and moved to another car. There are no hinges, latches or doors on it to get bent and out of shape. The loader is built in one, one and one-quarter and one and one-half yard sizes. The Bonney Supply Co., 377 St. Paul St., Rochester, N. Y.

The Kahn Armor Plate for Concrete Roads.

Concrete pavements are preferably provided with expansion joints across the roadway to

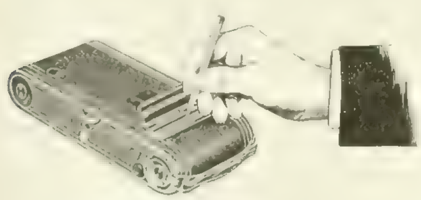


The Kahn Armor Plate for Concrete Roads.

allow for the expansion and contraction of the concrete and to prevent the formation of irregular cracks from changes in temperature and variation in moisture. These joints are ordinarily protected by steel plates so that the concrete cannot be chipped off at the exposed edges and form a weak spot in the pavement. There are two general types of these plates, the type illustrated with beveled edges and a type without beveled edges. The square corner of concrete back of the plate in the latter type is more apt to chip out than where a beveled corner is used, hence the development of the former type. The split prong ends of the joint may be bent in for shipment. When bent out in place they provide a firm bond with the concrete. The plates are made of open-hearth steel of the right composition and hardness to wear down with the pavement. The plates are 2½ ins. deep and are crowned at the factory as desired. The plastic asphaltum felt between the plates extends through the entire thickness of the pavement. The joint is manufactured and sold by the Trus-Con Steel Co., Detroit, Mich.

Camera Equipped for Recording Date and Title on Negatives When Made.

A variation in the ordinary "kodak" camera, which permits each negative when made to be permanently titled and dated, should be found useful by engineers in photographic recording of construction work. The Autographic kodak has a spring door in the back, which covers a narrow slot through which the titling is done (with a pencil or with the stylus provided for the purpose) on the red paper protecting the film. This slot is so located as to bring the title into the margin between the exposures. Such titling would appear on the bottom of an upright or at the left-hand end of a horizontal negative. The autographic film cartridge differs from the regular N. C. film cartridge in this respect: A thin red, instead of the familiar red and black (duplex), paper is used. This red paper, in itself, is not fully light-proof,



Autographic Kodak Camera.

but between it and the film is a thin strip of black displacing tissue. This tissue serves the double purpose of light-proofing the cartridge and of permitting the recording, by light, of writing upon the film. When the data has been written on the red paper and printed (by exposing with the door open to the sky for from two to five seconds) the image is photographically impressed on the film and appears when the film strip is developed. It is not a part of the autographic plan to have the title appear on the print itself, but simply to preserve it as a permanent record on the negative. It is obvious, however, that the title may readily be shown upon the print itself, the letters appearing in white upon a black ground. The process of development of the autographic film cartridge is exactly the same as with the kodak N. C. film cartridge. Autographic kodaks may be used with the regular kodak N. C. film cartridges. The other models of kodaks may be used with autographic cartridges, but to get autographic results one must use an autographic kodak and autographic film. Eastman Kodak Co., 300 State St., Rochester, N. Y.

A Telescoping Concrete Hoisting Tower.

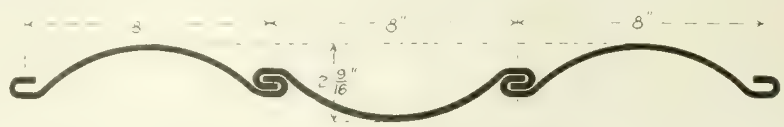
The accompanying illustration shows a new form of concrete hoisting tower, the principal feature of which is that the sections telescope into each other, thus permitting the entire tower to be loaded on a wagon. The tower is built to any desired height in sections, the sections being built up of steel angles, rigidly braced. At each corner of the bottom section there is fastened a worm and gears by means of which the tower is raised or lowered by four men. The discharge hopper and the operating mechanism are shown in the illustration, which shows the tower in its "lowered" position. The tower can be taken down and put up in a new position in a few hours without dismantling any part of it. The new tower is manufactured and sold by the Lorentz Iron & Machine Works, with sales offices at 95 Liberty St., New York, and works at Jersey City, N. J.

Concrete Hoisting Tower With Telescopic Sections, for Easy Transportation.

A Cold Rolled Steel Plate Sheet Pile

The steel plate sheet piling shown by sketch is designed for use pricipally (1) where economy demands a very low weight and cost per square yard of wall, (2) where the sheeting will not be driven in very long lengths or in hard material, (3) where transverse strength is secondary due to material remaining in position on both sides, (4) where high resistance against the passage of water through the joints is essential. These conditions are found in cut-off walls under levees, in core walls for dams and similar work and it is for such work that the pile was especially designed. The sketch shows clearly the shape of the pile and of the interlocking joint. The principal characteristics are: Weight per square foot of wall: 11.50 lbs.; weight per lineal foot of piling section, 7.66 lbs.; modulus of single section: [illegible] single section: 0.0925; moment of resistance in inch-pounds with factor of safety=4, or fibre stress taken at 16,000 lbs. per sq. in., 11,840; least radius of gyration of single section: 0.536.

This piling is manufactured by the Lackawanna Steel Co., Lackawanna, Erie County, N. Y.

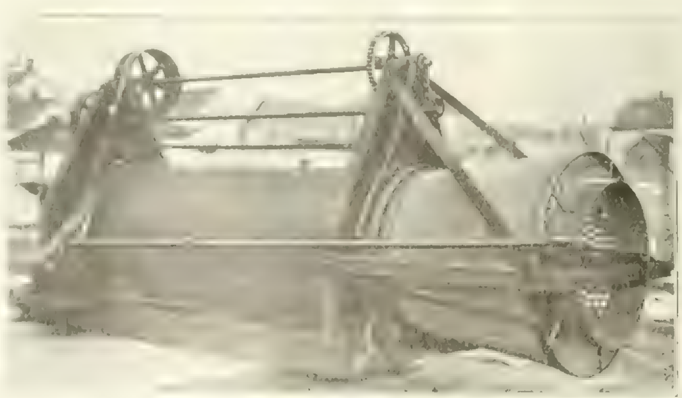


Cold Rolled Steel Plate Sheet Piling.

A Portable Gravel Washing Machine.

The washer illustrated is built in various large sizes for permanent gravel plants, but the illustration is of a small portable machine for concrete workers.

The washer is all metal, the end frames are cast iron, the revolving drum is of No. 16 sheet steel. On the inside of the drum are riveted angle iron bars, to which steel strips are riveted, forming an elevator carrying the material to the upper part of the drum, whence it drops into steel chutes, which are stationary and at fixed angle. From one chute the gravel falls into the water at the bottom of the drum, is taken up by the elevator and dropped into the next chute and so on until the washed material finally is discharged from the last chute. The clean water enters at the discharge end of the drum, passing through the drum in opposite direction and against the material and discharges over a flaring flange into a trough. The drum is 8 ft. long and 30 ins. in diameter. The base of the frame is of angle iron 4x6 ins. No. 1 is a machine to be used in concrete block factories, gravel pits, or for washing sand or gravel along river or creek banks where bridge building, pier and abutment work is done. The capacity of this machine is from 50 to 60 yards per day of 10 hours with a water supply of 10 to 15 gals. per minute. A 2½ hp. gasoline engine will run it. Floor space 3½ ft. by 10 ft.; weight of washer, 1,700 lbs. This machine can be furnished with a rotary pump attached. The machine is made by the Stocker Concrete Material Washer Co., Highland, Ill.



Small Portable Gravel Washing Machine.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLII. CHICAGO, ILL., NOVEMBER 18, 1914. Number 21.

Road Construction Plant and the Contractor.

Upon undertaking road construction it is required to be determined what type of construction plant is best suited to the work in hand, how much plant is required to complete the work with the greatest economy and profit, and what additional plant is required to expedite construction so that time limitations will not be exceeded.

Every construction project has certain fixed limitations within which the contractor must keep in planning work to be accomplished. These limits may be due to varied and uncontrollable causes, such as the type and number of laborers available, the location of the project with reference to bases of supplies, the length of profitable working season, topographic conditions with reference to ease of hauling and as affecting the installation and use of plant, climatic conditions during the working season—a most important factor—and facilities for quickly and profitably disposing of or storing plant after the completion of the work. Moreover, on contract work specifications invariably define other limits, such as the time of completion, the type and quality of material to be used, involving possible delay in securing materials at the proper time and consequent disorganization and loss to the contractor, and the possibility of important changes in the plan and extent of the work while in progress.

The foregoing are the broad general conditions confronting the contractor upon undertaking road construction work. In attacking this equipment problem the three important questions to answer are: What plant is best suited to the work, how shall it be acquired, and what will be its value after the completion of the work? In answering these questions sound experience is undoubtedly of greatest value in the proper correlation and nice weighing of the conditions affecting the proposed work.

There are, however, a few general rules of thumb in vogue among contractors that are worthy of mention. Plant is a substitute for labor, and its use is economical only when it will yield a good return on the investment over and above the labor cost without the use of plant. The economic ratio between plant and labor is, in a measure, a fixed quantity for each construction project. Whether or not any plant at all is needed and what is the least amount with which the work may be acomplished are fundamental questions. Finally, second-hand plant is to be avoided if the contractor expects to continue in the same line of contracting; if he expects to undertake a different type of work on the next project it may be a profitable investment.

Many contractors do not figure on plant expense when bidding on work. The first job undertaken is frequently figured with no profit other than the plant purchased. On succeeding jobs plant cost is not included, unless new plant is required. In short, the job pays for the plant. In figuring on the disposal of the plant at the conclusion of the work, as a rule, heavy machinery if sold will not bring to exceed 25 per cent of the first cost. Light machinery and small tools are usually worn out on the job and at its termination must be disposed of as scrap. Second-hand machinery ordinarily sells at about 50 per cent of its first cost after overhauling by machinery dealers. The salvage value of second-hand machinery but slightly used is larger than that of slightly used new machinery. In other words, the ratio of the value of third-hand to second-hand machinery is greater than second-hand to first-hand machinery.

Construction plant is the great problem of the road contractor. The cost of plant used frequently exceeds 33 per cent of the total cost of the work accomplished and seldom falls below 20 per cent. Unless plant is keenly judged, shrewdly acquired and kept constantly at work the profits of the road contractor may be continually feeding the mill in the merry-go-round of machinery purchasing.

Inefficiency in Hiring and Discharging Men.

Although it is generally recognized that the present methods used by managers of shops and factories in hiring and discharging workmen are inefficient and costly, few have given serious thought to a betterment of methods or even to a correct understanding of actual conditions. Generally, it has been taken as a matter of course that a large percentage of those employed would prove inefficient and unsuited to the work, yet the actual percentage seemed to be a matter of small concern. The economic loss caused by the indiscriminate hiring and training of workmen has not generally been realized. If an employer desired to increase his permanent force, say from 1,000 to 1,200 in a period of one year, under ideal conditions he would need to employ only 200 men, but conditions are not ideal; some die, some fall sick and, failing to return within a reasonable time, are discharged, while a very large number prove unfitted for the work at hand. The employer would be fortunate if double or treble the original force were not given a trial before the permanent organization was increased the desired 20 per cent. We believe the data submitted by M. W. Alexander in a paper presented before the National Machine Tool Builders' Association will surprise even those who have considered themselves acquainted with existing conditions and will show the need of a better understanding of this subject. Although the data apply specifically to factories, they are, in a general way, applicable to other industries as well.

A certain group of factories were selected by Mr. Alexander for investigation, the total number of workmen of various grades employed in this group being about 42,000. From Jan. 1 to Dec. 31 it was found that the number of men at work had been increased to 48,697, an actual increase in the permanent force of 6,697 men. To gain this increase 42,571 men were actually hired and set to work. It is thus seen that at the end of one year only about 16 per cent of the men employed were retained. It was concluded by the investigator that the hiring of at least 22,200 of these men could have been avoided, this number having been reached by assuming that 1 per cent of the employees die, that 5 per cent leave on account of sickness, that 10 per cent either withdraw or are discharged, and that the efficiency of the employment department is 75 per cent. The economic loss resulting from the indiscriminate hiring and discharging of workmen was classified in the original paper under the following heads:

(a) The clerical work of hiring.

(b) The instruction of new employees by the foremen and assistants.

(c) Increased wear and tear of and damage to machinery and tools by new men.

(d) Decreased rate of production during the early period of productivity of the new men.

(e) Increased amount of spoiled work by the new employees.

By using the preceding classification and percentages as a means of analyzing the direct loss to the employer and by dividing the workmen into various classes, Mr. Alexander reached the conclusion that the 22,200 men who should never have been employed caused an actual loss of over $35 per man, or a total loss of about $777,000 in one year. To decrease this loss the following suggestions are offered:

(a) A careful study of current employment statistics should be made with an analysis of the reasons for the discharges.

(b) We need a far higher grade of men in charge of the hiring and firing of men than we have had heretofore in our employment clerks.

(c) While it is important to exercise proper care, thought and study in the hiring of employees, it seems vastly more important to apply the proper methods in initiating new employees in the work, and to treat them properly.

(d) We ought to have effective systems of apprenticeship, factory schools, special training courses, or whatever name you may call them, so that we may not be dependent only on the grown-up men as they float around the country, but that we may effectively take hold of the youths of the country and train them in the ways of our industry, and in loyalty and intelligence.

(e) So far as it can be done, we ought to be able to regulate a little more the commercial requirements as they come to the factory.

An investigation of several factories in England and Germany indicates that the conditions existing in these countries are similar to those found in the United States, which tends to prove that the problem is of a somewhat international character. There is little doubt, however, that a systematic study of the problem will do much to lessen the losses occasioned by our present systems of employment. In our endeavor to make a machine do the work of many men in our shops and factories we have over-emphasized the value of the former and have not given sufficient consideration to the man problem. Mr. Alexander deserves great credit both for his investigations and for his analytic treatment of this important subject.

What Part Should the Engineer Take in Present Day Road Legislation.

Broadly stated road economics is divided into two distinct parts, namely, that which has to do with the welfare of society as a whole, and that which involves the economics of the methods used in attaining desired betterment. The former, under the American system of government, is a function of the people as a whole to be exercised by elected representatives, the latter involves the consideration of economic factors that are based upon detailed technical knowledge. The one is legislative in nature, the other executive.

Social conditions in the different sections of the state or nation are known to the citizens: the need for road improvement, the advisability of expending money for such improvement, the amounts that may be expended by the state or nation without embarassment to other necessary features of good government, as declared necessary by the assembled representatives of the various smaller political units and the devising of the necessary machinery to safeguard the expenditure of money are functions of the legislative body. But the initiation of plans for improvement involving the methods and materials to be used and the determining of the expediency of the extent and type of improvements at various points are functions of the executive branch of government and are in reality products resulting from the operation of machinery devised by the legislative branch. Legislation may bring into being a road commission created for the purpose of securing economy in the execution of roads, but when legislation specifies more than the results that are to be attained by that commission the need for the commission is vitiated; a circle is formed, responsibility going back to the people. Machinery is created but responsibility is not fixed.

What part should the engineer take in road legislation? Unquestionably his duty as a citizen demands that he take some part in the legislative planning of road work. His training in efficiency and methods and costs demands that he take a forceful part. The amount of work demanded of the individual by duty varies with the fitness of the individual to perform the work. If the training of lawyers in the principles of law peculiarly fits them for devising means of securing improvements in the general welfare and safeguarding the interests of the people, the training of engineers in the methods and economy of executing work more certainly prepares them for the analysis of problems involving the economic expenditure of money.

Fortunately, recent road legislation has been influenced to some degree by engineers. To no other influence can be ascribed the marked improvement in the types of road construction of the present day over those in vogue several years ago. And yet many improvements are yet to be made. Old evils that still hang on and new economic evils that have crept in which are freely discussed and their danger generally understood among engineers must be eradicated. It is a regrettable fact, however, that the influence exerted has been by individual officials rather than the concerted opinion of a good profession. This fact has frequently led to the downfall of the official that has had the moral courage to stand and fight for economical work and efficiency of subordinate employes, knowing that defeat means undoing and victory means future complication. The work of these men, however, has not been in vain. Engineers are realizing more strongly than ever before the value of their influence as a profession and their duty in exercising it wisely and forcefully in affairs [illegible]

An Opportunity to Secure the Appointment of an Engineer as a Public Service Commissioner.

The election of a Republican governor in New York state makes it probable that at least two new public service commissioners will be appointed by him early next year. Not only should engineering societies in New York state urge upon the governor the appointment of engineers to these positions, but national engineering societies might well join in the appeal.

More and more is it apparent that appraisal and rate problems are largely engineering problems. Business economics play a part in decisions as to individual rates in a composite rate schedule, but engineers are fully as capable [illegible] business economics as are lawyers, for example. Engineers, as a consequence of a scientific training, are ardent seekers of facts, and certainly they are excellent judges of the significant facts that relate to costs of constructing and operating public utilities. We have often called attention to complete misinterpretation both of theories and of facts by men not trained as engineers.

Engineers of the best sort are ordinarily *not* office seekers. Hence the necessity of united action by engineering societies. Individual members of the societies will rarely attempt to secure political appointments. This absence of political ambition is itself a strong recommendation for position on a public service commission. In one state the public service commission has completely changed three times in six years. In California the present chairman of the commission has just been elected lieutenant governor. The bane of commission regulation in several states has been the changing of commissioners, either as a result of political action beyond their control or as a result of the political ambitions of commissioners. It is especialy noteworthy that where engineers hold positions by appointment they generally last through several changes of political administration. Let it once be generally realized that the work of public service commissioners is mainly of an engineering nature. Let high class engineers be appointed as commissioners so as to form a majority of each commission, and it may be confidently predicted that rate regulation will be taken out of politics.

The Significance of the Voting Down of a Full-Crew Bill in Missouri.

For the first time a popular vote has been cast upon a "full-crew bill." Several state legislatures have passed such bills at the behest of labor unions, but hitherto the public that must ultimately foot the bills resulting from such "bills" has not been heard from. Now, by an overwhelming referendum vote, the people of Missouri have reversed the decision of their legislators and governor who had passed a law requiring all trains to carry a "full train crew." This vote is, we think, highly significant.

The public was for several years fed on sensational magazine articles about the alleged extortionate profits wrung from the people by railways and other public service corporations. The millions made by a few "railway kings" formed the theme of many a magazine writer. The millions lost in railway enterprises were not mentioned, nor was attention called to the millions of dollars of land values created in America largely by the railways, to the benefit of countless individuals. For obvious reasons the spotlight was kept on a few "swollen fortunes" and on instances of corrupt practices of which some railway men were guilty. But no equivalent light was turned upon the vastly more significant development of a great continent through the agency of the iron horse and his wheeled legs. The muck-raking writer posed his model not with a view of securing a correct perspective, but placed the model's feet as near to and his head as far from the camera as possible. Then pointing to the photograph, he said: "Note the anatomy of this monstrous creature. I call your attention especially to these huge feet that he uses to trample upon your rights, and ask you to observe with equal care the almost microscopic head with which he directs his activities. As for his heart, you will search in vain to find it in this life-like reproduction that I place before you."

For a long time it was, and to a limited extent it still is, profitable to publish distorted caricatures of men and organizations. It was not particularly profitable—or was not perceived to be—to publish true pictures, for sensationalism always attracts an audience when rationalism can scarcely hold attention. A murder trial will fill a court room, but the trial of a public utility rate case speedily empties the same room. Yet from the one the audience departs as poor in useful knowledge as it entered, while from the other a hearer may gain not only a knowledge of every day economics that concern his own pocketbook, but he may come to see far into the broad principles of industrial and political economy.

The public is shallow as to its reasoning, but deep as to its sense of justice. At first misled by specious articles as to the general character of corporations and their managers, the public has slowly been developing an intuitive perception of facts. Appraisal after appraisal of public utility property has disclosed an absence of over-capitalization in the great majority of cases, and with only the rarest exceptions have the true net earnings exceeded 8 per cent on the cost of reproducing the physical property. Slowly these facts have filtered into the public press, and have begun to have their effect. Intuition has led the public [illegible] of exorbitant profit have as yet been found by public service or railway commissioners there is every likelihood that few can be found. Commissions will naturally pick out the "shining marks" among public utility companies as their first subjects of investigation. Hence the relatively few proven cases of excessive profits indicate a still smaller proportion when *all* utilities shall have been investigated. The public needs no profundity of knowledge to see the signficance of the abortive efforts to find "robber corporations." But beyond this self-evident failure to find an instance in real life to match the fictions of the magazines, there have been other reasons for a popular revulsion of feeling toward railways and other utility corporations.

Every justice-loving person must have been shocked by certain features of the recent hearings of the "5 per cent rate increase case" before the Interstate Commerce Commission. First, there was the appointment of Louis Brandeis as "special counsel" to the Commission. The word counsel means, to the ordinary mind, advisor. The Commission—a semi-judicial body—selects as advisor an attorney notoriously opposed to the railways, a man who had been retained previously by shippers to fight against rate increases. This also was the man who had urged that by applying "scientific management" the railways could reduce expenses to such a degree that existing rates would be adequate. ENGINEERING AND CONTRACTING has repeatedly contended that "scientific management" has not been fully appreciated, not only by the managers of railways but by managers of every other industry. It does not follow, however, that they can be forced to apply "scientific management" against their wills, nor does it follow that even if they did adopt all its tenets they could secure great economies for a considerable time. The first spectacular advertising of "scientific management" resulting from the assertion of Brandeis that a million dollars a day could be saved by the railways has, perhaps, done much to retard the spread of its principles, particularly among railway managers.

But if it was a poor sort of advisor that the I. C. C. selected in person of Brandeis, it was a worse one that they had thrust upon them in the person of Clifford Thorne. The latter is one of the railway commissioners of Iowa, yet he has come before the I. C. C. in the capacity of a prosecuting attorney! Could any act be more repugnant to a sense of justice than the temporary conversion of a state railway commissioner into an avowed prosecutor of the railways, Bear in mind, also, that Thorne's own state was not directly involved in the "5 per cent rate case." What would be thought of a judge who, while still holding a position as a judge, should voluntarily appear before the Supreme Court as a prosecuting attorney? Yet this is precisely the sort of thing that Clifford Thorne did. Is it to be wondered that the public's stomach is beginning to turn when it receives this sort of "justice"?

But this is not all. Decision after decision has been rendered by state public service commissions couched in language so abusive of corporations, so extravagant in its denunciation that every well-balanced man who has read even a small fraction of such decisions must have been impressed with the absence of fairness and judge-mindedness in those who have written many of the decisions. The galleries could not more obviously have been played to by a melodramatic actor.

Finally it has been clear that the fall in the prices of railway securities and the steady decline in railway construction continues mainly because of alarm among investors. It was idle to tell the most discerning class of investors that railway commissions had made railway investments more secure than ever in the face of the fact that rates were being cut. Commissions were all too evidently seeking the "fat" and ignoring the "lean" parts of railway income. This process could end only in skeletonizing the entire railway system. At first railway managers were afraid to give voice to their fears, for doing so would make it still more difficult to finance the refunding of bonds, and the like. Ultimately a few of the bolder railway presidents decided to make public their belief that a continuation of such commission regulation as they were experiencing would end either in government ownership or in financial disaster. The public has been listening to both sides of the controversy, and, although the public is not a good judge of facts, its intuition both of what is fair and what is "gallery playing" is sound. The first vigorous expression of this intuition comes from the people of Missouri. Can it be doubted that other states will be heard from before long in a similar manner?

SEWERAGE

Converting Old Septic Tank and Contact Beds Into Two-Story Tank and Sprinkling Filters at Moorestown, N. J.—Operating Results.

There are a number of sewage disposal plants in this country of comparatively recent construction which no longer meet the demands placed upon them by the rapidly growing community which they serve and the more and more exacting demands of sanitation. Many of these plants were built at a time when the engineering profession had few data to guide them as to the large quantities of ground water which, in certain localities, are likely to enter the sewers and sewage disposal plant. Many of the plants are greatly overloaded and badly in need of extension. To extend them to meet the increased sewage flow due, either to the rapid growth of the community or to the large quantities of ground water reaching the plant, but not provided for in the original design, runs up into large sums of money which many of the communties cannot afford to spend. Such extensions can often be very economically made by completely remodeling such plants to adapt them to the growing needs of the municipality. This article describes in some detail how Moorestown, N. J., has solved the problem of extending its sewage disposal plant by completely remodeling it. The information given is from a paper by Mr. Alexander Potter, consulting engineer, New York City, before the recent annual convention of the American Society of Municipal Improvements.

ORIGINAL PLANT

Septic Tank.—The sewerage system and sewage disposal plant of Moorestown, N. J., were designed and built by Mr. Potter in 1901. As originally constructed and operated, the disposal plant consisted of a grit chamber 10 ft. square and 10 ft. deep. It was expected that this chamber would be cleaned out frequently, but it was actually cleaned out about once a year. From the grit chamber the sewage entered an open septic tank which consisted of an uncovered brick chamber 75 ft. long, 25 ft. 6 ins. wide and with an average depth of 7 ft. Two 20-in. brick walls divided it into four equal compartments arranged so that two, three or four of the chambers could be used at any one time. The last compartment was used also as a dosing chamber, the supernatent liquid in the upper 4 ft. of this chamber being automatically discharged by two 8 in. siphons onto the contact bed.

Contact Bed.—The contact bed was divided into four units, each 85 ft. by 42 ft. in area. Each unit was embanked with 16-in. brick walls. The natural soil, of a clayey nature, formed the bottom of the bed. Three of the beds were filled to a depth of 4 ft. with slag ranging in size from ¾ in. to 1½ ins.; the fourth bed was filled with cinders. The distribution onto the contact bed of the effluent from the septic tank was accomplished by means of a system of open-joint sewer pipe ranging in size from 10 ins. to 8 ins. and imbedded up to its horizontal diameter in the filter material. Each unit was underdrained by a system of vitrified sewer pipe laid with open joints and ranging in size from 6 ins. to 10 ins. Four 10-in. pipes conveyed the effluent from the four units to a common valve chamber, about 45 ft. away. In this chamber were located the four effluent control valves. A 15-in vitrified pipe conveyed the effluent from this chamber to the edge of Pensauken Creek, a small stream tributary to the Delaware River.

Early Operating Conditions.—For a period of several years after its construction the plant was virtually allowed to take care of itself. This is not an unusual condition in small towns; in fact, the indifference of municipalities, large and small, as to the proper care of such improvements costing large sums of money is most surprising and is one of the worst evils the consulting engineer has to fight. This condition was permitted to exist in Moorestown notwithstanding the fact that the chairman of the Township Committee was a physician of more than local reputation.

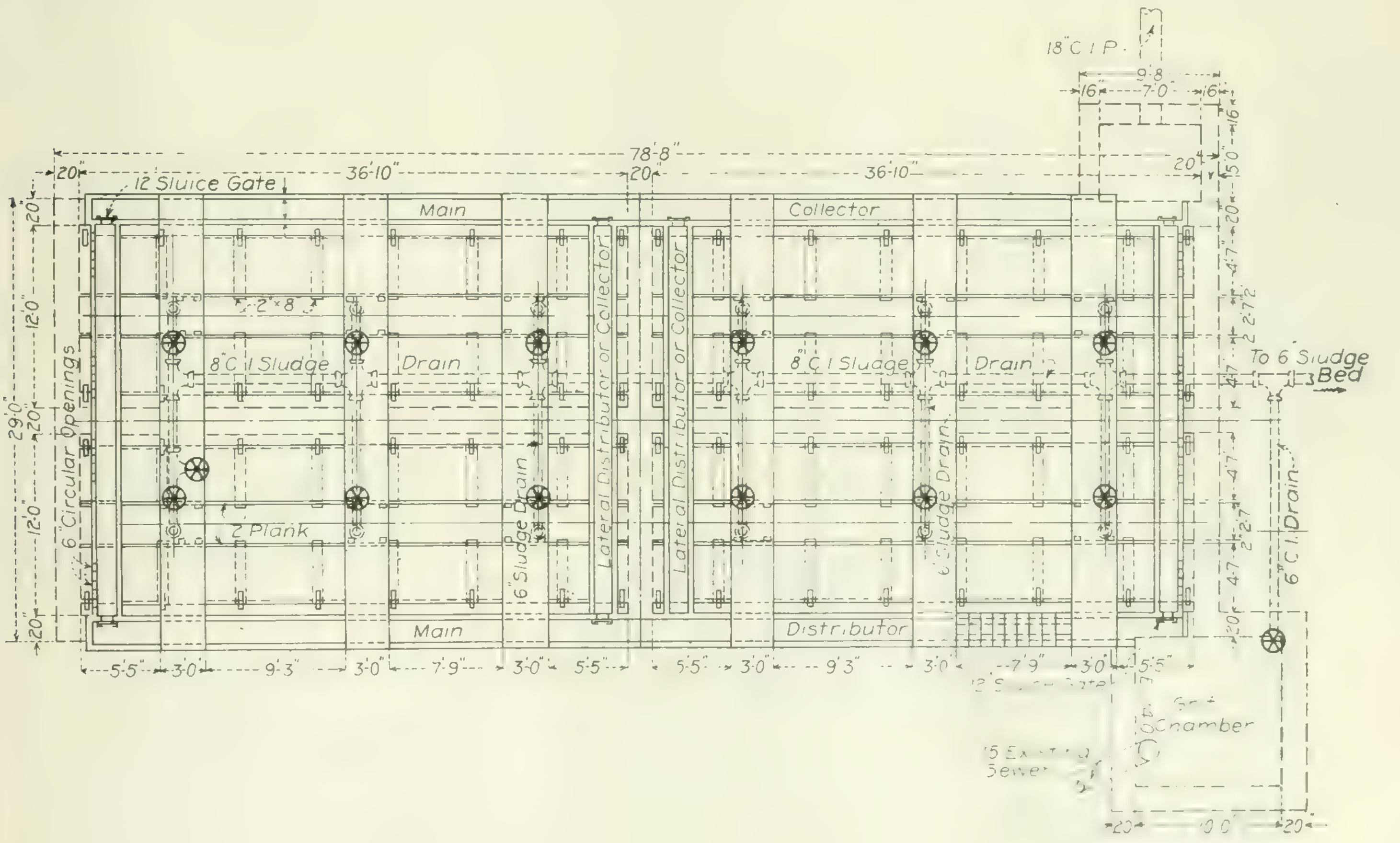


Fig. 1. Plan of Septic Tank at Moorestown, N. J., as Converted Into a Modern Two-story Tank.

ORIGINAL PLANT OVERTAXED.

The plant as built in 1901 was designed to treat from 200,000 to 250,000 gals. daily based upon a retention period of eight hours in the septic tank and an average rate of 600,000 gals. per acre per day for the contact bed. Since its construction in 1901, the sewer district grew very rapidly and the capacity of the plant became overtaxed.

In March of 1911 the State Board of Health made an inspection of the plant and found it to be treating a flow of about 500,-

000 gals. daily. On this basis the total calculated storage in the septic tank was four hours. As the fourth unit of the septic tank was at that time used as a dosing chamber, and as the tanks were also partially filled with solids, the actual storage was but a fraction of this amount as ascertained by tests made by the State Board of Health. Uranine placed in the sewage at the inlet to the plant appeared in the effluent of the filters in just one hour. The net rate on the filters, including the periods of rest, was at this time 1,250,000 gals. per acre per day. The flow was so great that the automatic siphons for dosing the contact beds intermittently were unable to handle it during the day and did not break except at the time of light flow during the night. In spite of this tremendous overload, it is the remarkable fact that analyses made on two separate days showed considerable oxidation, and a series of hourly samples taken on the 7th of March were found to be nonputrescible up to the sample taken at 4:30 p. m.

In view of the overloaded condition of the plant, the State Department of Health ordered the enlargement of the plant in 1912. The following summer the people of Moorestown voted $30,000, both for extensions to the sewerage system and for the enlargement of the sewage disposal plant. This amount was determined upon before the services of an engineer were secured and plans of any sort prepared. Of the total amount voted, $15,000 was to be devoted to building sewers, leaving only $15,000 available for sewage disposal purposes, irrespective of what the true needs of the case might require.

With an unlimited appropriation, design in the majority of cases is not difficult. With an inadequate appropriation, it is often necessary to resort to expedients which at times develop interesting results.

[illegible]

With a view of increasing the capacity of the plant, two alternate plans were considered, the one suggested by the State Board of Health to double both the septic tank capacity and to increase the area of the contact bed, and the other to remodel the existing septic tank into a modern two-story settling tank and to change the existing contact bed over into sprinkling filters. Further investigation showed that the existing contact bed was more or less clogged and needed overhauling. The disposal of the sludge from the existing settling tank also had be- [illegible] pointed to the advisability of remodeling the plant. Therefore, the latter plan, contemplating the remodeling of the plant, was adopted as being not only the more sanitary, but the more economic one in the end.

[illegible]

The plant as remodeled is of more than usual interest to the profession in that it shows that a disposal plant consisting of a septic tank and contact beds—a combination used extensively in this country a decade or more ago—can be readily remodeled into a modern two-story settling tank and sprinkling filter.

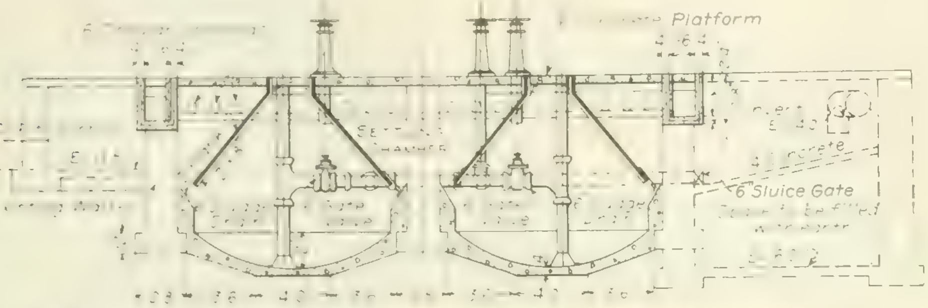


Fig. 2. Transverse Section of Remodeled Sewage Tank at Moorestown, N. J.

Settling Tank.—The remodeled tank is shown in Figs. 1, 2 and 3, the new work in solid lines and the old work in dotted lines. Figure 1 is a general plan; Fig. 2, a transverse section; and Fig. 3, a longitudinal section of the remodeled tank. The work of remodeling consisted in removing the existing bottom and excavating to an additional depth of 2 ft. 2 ins. As the new bottom is carried below the foundation walls of the old tank it was built in the form of an arch to give the necessary stability to the walls and to assist in the removal of the sludge. In the old septic tank the level of the liquid was maintained at an elevation of 15 ft. above sea-level. In the remodeled tank this was raised to elevation 15.5, necessitating the raising of the outside walls a height of 6 ins. These changes increased the depth of the tank from 7 ft. to 9.5 ft.

The remodeled tank consists of four units each of which is again divided into three compartments by a false bottom constructed of 2-in. creosoted yellow pine. This false bottom makes a slope of 40° with the vertical. The upper two compartments are settling compartments and the lower and larger compartment is the sludge digestion compartment. The total settling capacity amounts to a retention of 1.35 hours when the plant is operated at the rate of 500,000 gals. daily. The total capacity of the sludge digestion chamber figured below the level of the walls is 5,470 cu. ft., about 50 per cent. larger than required by the established practice. The general arrangement is clearly shown in Fig. 2.

Three 6-in. sludge pipes extend down into the sludge digestion chamber of each unit. All the sludge pipes are connected to a common 8-in. cast iron sludge drain, the general arrangement being that shown in Figs. 1 and 3.

The system of distributing the sewage to the various units is of interest, as it possesses not only flexibility of operation, but also is very efficient in preventing disturbances from currents and vortex motion in the settling compartments.

After leaving the remodeled grit chamber, the sewage enters the main distributing trough, a concrete rectangular channel built directly upon the existing wall of the settling tank. This channel extends the entire length of the basin. Extending across the tanks are four lateral rectangular troughs, each 15 ins. wide and 2 ft. deep. These lateral troughs can be used either as distributors or collectors. For this purpose each of the lateral troughs is provided with shear gates at either end so as to regulate the direction of the flow.

Attention is called to the method used of leading the sewage into the settling compartment. The liquid enters the basin in the direction opposite to that it must take in traversing the basin. This method has proven to be very effective in arresting vortex motion by impinging the inflowing current against the wall of the basin. The collecting trough acts as a scum board and tends to prevent floating matter from being carried out with the effluent. In each unit the direction of flow can be reversed by the simple process of closing two shear gates and opening two others. Reversing the flow at periods of about a month tends to the more uniform distribution of the sludge in the sludge digestion chamber.

Six transverse concrete walks, each 3 ft. wide, give the operator ready access to all parts of the tank. Except for these walks, the top of the tank is entirely open.

Wooden covers are provided for the main distributing channel and the main collecting channel so that during the winter months these channels can be covered to prevent the freezing of the sewage at the dead ends.

Sludge Disposal.—An 8-in. cast iron pipe conveys the sludge from the tank to the sludge bed the surface of which is 5.5 ft. below the water level in the settling tank. Unfortunately, this sludge bed had to be located about 200 ft. away from the settling tank and a closed conduit used to convey the sludge from the tank to the bed. There are several objections to a closed conduit for conveying sludge from an Imhoff tank to the sludge bed, the principal one of which is the difficulty of controlling the character of the sludge discharged and the large quantity of water required to flush out the sludge piping. To prevent the clogging of the sludge drain, provision is made so that the sludge pipe can be flushed out with settled sewage after every use.

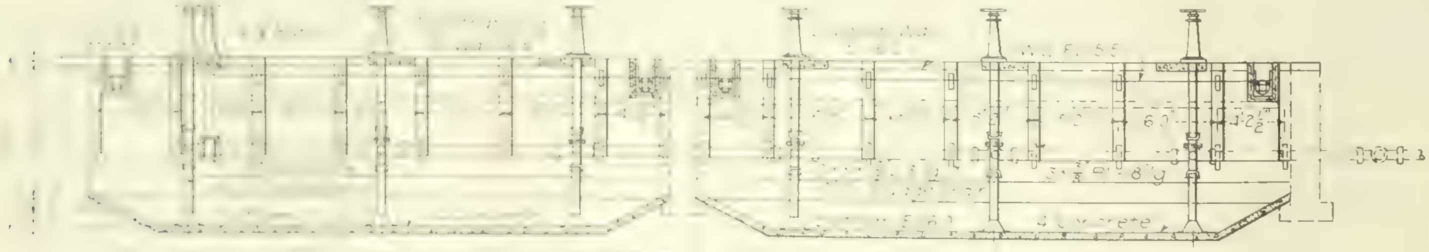


Fig. 3. Longitudinal Section of Remodeled Sewage Tank at Moorestown, N. J.

CONTACT BEDS CHANGED TO SPRINKLING FILTERS.

Instead of increasing the area of the contact beds to take care of the additional flow and to put them in proper condition, it was found more economical to remodel the beds into sprinkling filters. The original beds were only 4 ft. deep, a depth which is not sufficient for an efficient sprinkling filter. It was, therefore, decided to excavate the bottom of the beds from 6 to 12 ins. below the bottom of the old contact beds and to bring the finished surface of the sprinkling filter 6 ins. above the level of the old bed. This gave a depth of filtering material ranging from 5.5 ft. over the main drain to 5 ft. at either side.

It was found that the maximum head available to operate the nozzles was about 4 ft. 9 ins. A circular type of nozzle was used which was spaced 10 ft. to 4 ins. on centers in rows spaced 9¾ ins. apart. To reduce the frictional losses in the siphon and piping system to the minimum, the siphon was made 24 ins. in size—somewhat larger than customary. The main distributor and all piping were made of ample size so as to keep the

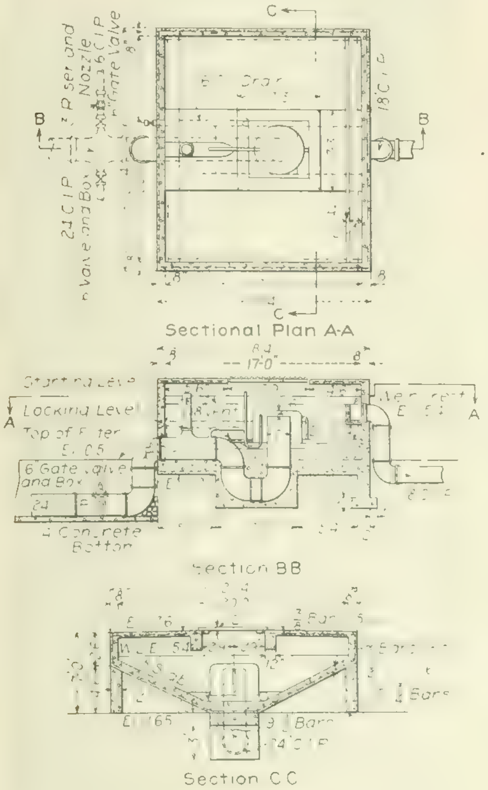


Fig. 4. Details of Dosing Chamber at Moorestown, N. J. Sewage Treatment Plant.

frictional losses, even when the plant is extended in the future, within 6 ins.

The nozzles were furnished by the Pacific Flush Tank Co., and are provided with adjustable orifices. Adjustable orifices, although not generally used except for experimental purposes, are of value in such a plant in that the operator can curtail the discharge from those nozzles over any portion of the bed where a tendency to pool and clog exists. This adjustable orifice has proved of value for when the plant had been in operation several months, an area about 10 ft. by 25 ft. in extent, over which the teams traveled in placing the filter material and which had been compacted by traffic, commenced to pool. This trouble was quickly remedied by throttling the nozzles which discharged over this area, thereby doing away with the tendency to pool.

Because of the limited funds available, and in view of the clayey nature of the subsoil, it was decided to place the filtering material directly upon the subsoil. The underdrain system of each unit consits of a central concrete channel 15 ins. in diameter. Extending at right-angles from this channel and spaced 24 ins. on centers, are lateral drains consisting of 6-in. vitrified bell and spigot pipe laid with open joints. The central drain is ventilated at either end.

COST OF REMODELING THE PLANT.

The cost of remodeling the plant and enlarging it 50 per cent to adopt it to the present flow was as follows:

Remodeling septic tank into two-story tank	$ 3,100
Construction of dosing tank, siphon and piping	1,800
Sprinkling filter, one-quarter of an acre..	8,300
Sludge bed and piping	300
Additional piping around plant	1,300
Final settling basin	500
Total	$15,300

The cost of remodeling the contact beds into a sprinkling filter was somewhat more than was expected, as it was found that less than 10 per cent of the filter medium of the original beds could be used in the new filter.

OPERATION OF THE PLANT.

Trouble with Fungus.—Almost from the very beginning when the plant was put in operation, the operator experienced considerable trouble with sewage fungi of the Leptomitus type. Masses of sewage fungus reached the plant continuously in such quantity as to completely cover, within 24 hours, the surface of the grit chamber 100 sq. ft. in extent with a solid floating mat varying in depth from a few inches to 12 ins. This fungus, because of its tendency to float, attached to the wetted perimeter of the pipe, the outlet sewer was found to be quite clean because of the high velocity of the sewage, amounting to from 2 to 3 ft. per second. This condition was not found to be the case in the laterals and the upper end of the outfall sewer where the velocity was about 1½ ft. per second and where the sewers were more or less unclean. All along the trunk sewer and in the laterals the growth of the fungus was so luxuriant that in many cases it completely covered the wetted perimeter of the sewer, not only where the flow was less than 2 ft. per second, but also in the lower portions of the outfall sewer where it reached from 3 to 4 ft. per second. Large masses of this sewage fungus were constantly becoming detached from the sides of the sewers and were

TABLE I.—AVERAGE ANALYTICAL RESULTS OF SAMPLES OF SEWAGE, MOORESTOWN, NEW JERSEY.

(Expressed in parts per million.)

Nature of determination.	*Raw sewage.	Grit chamber outlet.	Tank outlet.	Sprinkler effluent.	Final effluent.
Turbidity	[illegible]20	220	147	30	25
Total solids in suspension	15	17	5	11	11
Fixed solids in suspension	4	3	2	6	4
Loss on ignition	11	14	3	5	7
Organic nitrogen	30.0	49.9	43.3	33.3	31.1
Nitrogen in solution	23.3	36.6	34.4	28.3	28.2
Nitrites	0.040	0.113	0.119	0.227	0.228
Nitrates	0.08	0.45	0.39	2.63	2.90
Free ammonia	15.00	13.07	13.07	8.13	6.90
Oxygen consumed in solution	52.30	40.13	30.80	16.23	12.29
Oxyden dissolved	.....	0.85	0.87	5.87	5.72
Chlorine	33	40	36	34	31

*Determination made only on one day.

passed through the settling basin and dosing chamber and onto the filter bed. The large masses of fungus, sometimes a square foot or more in extent, gave considerable trouble in clogging the nozzles. At first the operator tried to improve conditions by inserting a baffle board in the outlet of the grit chamber, thereby retaining practically all the fungi and preventing them from reaching the settling basin. This method involved a large amount of labor daily in removing the accumulated masses of fungus from the grit chamber. The fungus thus removed was highly putrescible because of the large quantity of fecal matter which was mixed up with it, and produced offensive conditions around the plant. At the suggestion of Mr. Francis E. Daniels, Director of Water and Sewerage Inspection, New Jersey State Board of Health, an open channel was constructed connecting the grit chamber with the sludge digestion chamber, through which the floating masses of fungus accumulating in the grit chamber could be pushed into the sludge digestion chamber. This method, although a material improvement on the method first used by the operator, was not entirely successful because of the large quantity of fungus that had to be handled daily.

A thorough inspection of the town sewers showed that the sewage fungus grew in great abundance in the main outlet sewer and the laterals. Except for the fungus growth reaching the plant continuously in enormous quantities.

Remedial steps were taken to abate this nuisance, by flushing the sewers from the fire hydrants. Enormous quantities of fungus were thus dislodged and flushed out. The treatment has been quite effective and very little, if any, Leptomitus is now reaching the sewage disposal plant.

Sludge Trouble.—Considerable trouble has also been experienced in operating the remodeled plant, from floating sludge. It is the author's opinion that this trouble is not peculiar to the Moorestown plant, but is experienced more or less generally in connection with the operation of all two-story settling tanks, differing only in degree. After the Moorestown plant was placed in operation it appeared that the gas ebullition in the digestion chamber produced a very thick scum. The suspended matter in the sewage which had settled into the digestion chamber was lifted to its surface by the active gas ebullition and kept there. On reaching the surface it was matted together by means of paper, hair, fat and to some extent by vegetable molds, producing a tough floating mass which reached after several months a depth of 3 ft. or more. As the surface of the scum became weathered, the escape of the gases of decomposition was further retarded, thereby aggravating the condition. A view of the floating sludge is shown in Fig. 5.

Fig. 5. View of Top of Moorestown Sewage Tank, Showing Floating Sludge.

When the surface of the scum was lifted more than 12 ins. above the water line and there was danger of it overtopping the walls of the tank, the operator was instructed to break up the compacted sludge by means of a long-handled rake worked up and down through the scum, thus liberating the gases which were giving the mass its buoyancy. This operation was only partially successful. Although the sludge when properly broken up settled back to the level of the liquid, within 48 hours it had regained its original position and the process had to be repeated.

As this method involved considerable labor and had to be freqnently repeated, another method was tried. In the operation of septic tanks it has been noticed that rain falling upon scum in an open tank tends to break it up and causes a large portion of the floating matter to sink to the bottom of the tank. The experiment was, therefore, tried of sprinkling the surface of the scum with settled sewage, no water being available at the plant. The water thus applied softens the crust which has formed on top of the scum, giving the gases of decomposition which have collected in the mass a chance to escape. Furthermore, it seems to increase the specific gravity of the floating matter sufficiently to cause it to settle. Even with the application of water a certain amount of stirring has been found to be of value.

At the Moorestown plant the area of the sludge digestion chamber exposed to the atmosphere is approximately 28 per cent of the surface of the settling compartment. It is the author's opinion that the smaller this ratio the more serious will be the trouble from floating sludge on account of the smaller surface from which the gases can escape.

OPERATING RESULTS.

Table I gives the average analytical results of samples of sewage taken at the Moorestown sewage disposal plant, by the New Jersey State Board of Health.

Three separate determinations were made by the State Board, one on May 12, one on June 4 and one on Sept. 10. Composite samples were taken on those days at half-hourly intervals covering the period from 11 a. m. to 2:30 p. m. Only one determination was made on the character of the raw sewage, namely on Sept. 10.

Considerable sedimentation takes places in the grit chamber, far more than is indicated by Table I as the sewage on Sept. 10 was considerably weaker than that taken on the other two days It appears that the grit chamber is entirely unnecessary and should have been abandoned when the old plant was remodeled. Its continued use entails considerable work upon the operator in removing the large quantities of heavier suspended matter which settle out of the sewage in the grit chamber.

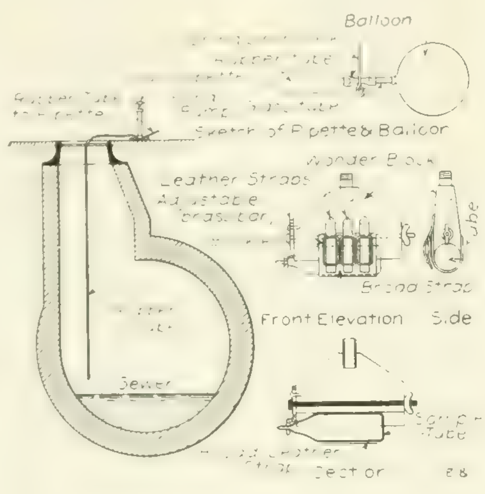
The plant yields an unsually clear effluent, free from sediment and very low in turbidity. The oxidation in the filters is very complete. Tests show the effluent to be non-putrescible during the greater part of the time.

Discharge of Inflammable Wastes Into Sewers—Problem of Prevention—The Pittsburgh Sewer Explosion Investigation.

It is thought that some of the more recent and more violent explosions in sewers were caused by the presence of gasoline vapor. If this belief is well founded it follows that the advent of motor driven vehicles has created a problem in sewer design and maintenance which may prove difficult and expensive to solve. Sewer systems are generally looked upon by the average person as a quick and easy means of disposing of any or all waste matter which can be carried away by the sewer, without any consideration whatsoever of the effect of such discharge, either upon the structure itself, or upon its maintenance and operation. The transition in the mode of travel from horse-driven vehicles to the motor-driven car and auto-truck renders the transportation, handling and use of large quantities of gasoline necessary and it is inevitable that in the handling of this material some will be spilled or wasted either by accident or design, which will find its way into the sewers. The solution of this problem is now occupying the attention of sewerage engineers. The present article, which is taken from a paper by Mr. N. S. Sprague, Superintendent of the Pittsburgh Department of Public Works, before the recent annual convention of the American Society of Municipal Improvements, discusses the discharge of inflammable wastes into sewerage systems and the problem of prevention. The scope of the sewer explosion investigation now being conducted in Pittsburgh is also described.

There are many sources which contribute inflammable wastes in a greater or less degree, ranging from the small and irregular discharges from households and private garages, which may amount to considerable in the aggregate, to the large and intermittent discharges from manufacturing and storage plants and other enterprises which use large quantities of gasoline.

In some cities (Pittsburgh included) the laws relating to the storage of gasoline require



Details of Apparatus for Taking Samples of Air Inside of Sewer. Sewer Explosion Investigation, Pittsburgh, Pa.

the tanks to be buried in the ground. These tanks, which are made of riveted steel plates, vary in capacity from about 50 to 15,000 gals. The purpose of placing them underground is to prevent possible ignition of the gas and protect them in case of a nearby fire. It is not the author's intention to discuss the advisability or the objection to placing gasoline storage tanks underground, but simply to point out the possible danger of gasoline escaping from these tanks and entering the sewers. The thickness of the steel plates of which the tanks are made, s generally from ⅛ to 3/16 in. and their only protection from corrosion is the application of ordinary structural paint. The tanks are laid directly upon the ground and then covered with earth. Under such conditions, corrosion is rapid. It is also possible, under favorable conditions, that the tanks may suffer injury due to electrolytic action. In any case, there is no opportunity for inspection or repairs and leaks can only be detected by making a comparison of the quantity of gasoline put into the tank with the quantity removed. This information is in the possession of the owners and in case a leak is disclosed by a comparison of the figures, the owners are not likely to volunteer the information to the public authorities.

The possibility of gasoline escaping from the tanks into the ground and finding its way into the sewers may be remote, but with pervious soil or a nearby catch basin or trap, the opportunity for leakage into the sewers is at least present. In certain locations it is quite possible to set these tanks above ground, where ample opportunity for inspection and repairs would be possible. The waste gasoline from households, private garages and shops is so well distributed throughout the lateral sewer system and the average amount discharged at any one time so small that it is quickly dissipated before the formation of explosive vapors can occur. It is therefore to be supposed that the formation of gasoline vapor and other explosive gases present in sewers originates from establishments which are large users or dealers in inflammable materials.

There being in most cases no laws prohibiting the discharge of inflammable wastes into sewers and the danger of such practice not being generally understood, the natural disposition of such wastes is into the sewers. These wastes comprise dirty and used gasoline, benzine, oil, washings from tanks, and refuse from gas plants, paint works, etc. The quantity of these waste products varies according to the magnitude of business and methods employed.

While the discharge of gasoline into the public sewers probably exceeds in quantity any other inflammable waste, yet the discharge of waste products from paint works, oil refineries, gas works, etc., is likely to produce conditions, which, under favorable circumstances, may fill the sewer with explosive gas. Ignition of explosive gases, when present in the sewers, may occur in many different ways —for instance: Sparks from street railway tracks, hot cinders and sparks from locomotives, stacks, etc., which may enter the manholes through the perforations in the covers, or when same are removed for inspection or repairs; also the dropping of matches or lighted cigars into manholes or catch basins; lights and sparks from tools, while making inspection or repairs within the sewer or at chambers, pumping stations or disposal plants.

The problem of preventing sewer explosions would then seem to be a question of either effectually sealing all openings into sewers or excluding or regulating the discharge of inflammable or explosive wastes.

A number of cities have attempted to solve the problem by procuring legislation prohibiting or regulating the discharge of inflammable waste materials into the sewers. Prior to the general use of motor vehicles there were many industrial and business establishments using inflammable and volatile wastes, such as dry cleaning establishments, paint manufactories, gas works, etc. Notwithstanding the fact, explosions in sewers caused by the ignition of gasoline vapor were uncommon. This fact would seem to indicate that the greatly increased use of gasoline due to the growth of the automobile industry has been responsible for many of the recent sewer explosions.

Accepting this theory as a working basis, we must determine whether or not the gasoline is discharging into the sewers in large quantities by a relatively few people or in small quantities by a great number. In the first case the situation is relatively easy to control, while in the latter, it would be difficult. Moreover, it is necessary and important to determine whether the explosive vapor is generated from the accumulative effect of a great number of small discharges or from the discharge of large doses. Past experience has shown that the ordinary means of providing ventilation in sewerage systems has been generally adequate to prevent the collection of explosive gases. If large doses of inflammable wastes are allowed to enter the sewers, other means of ventilation will have to be provided or the sewers sealed The installation of mechanical ventilation in the sewers throughout the system would remove the gases, but would involve great initial outlay and the cost of maintenance and operation would generally be prohibitive. This scheme would not seem practicable. There is no practical way of providing sufficient ventilation either by mechanical or natural means which would exhaust the air inside the sewer quickly enough to prevent the formation of an explosive compound in case large quantities of gasoline were present in the sewer. With the exclusion of large discharges of gasoline into the sewers, the danger of explosions can be greatly lessened by giving more attention to the improvement of the natural ventilation. This would probably be sufficient to prevent the collection of explosive vapors arising from the normal amount of gasoline discharged into the sewers. To form an explosive mixture a certain amount of air and gas is required. If there is a shortage of gas or an excess of air, no explosion can occur.

It cannot be ignored that many sewer explosions have resulted from the leakage of natural or artificial gas into the sewers. Evidence has been conclusive in a sufficient number of cases to show unmistakably that this is a fact. The prevention of explosions from this source, however, is well within the jurisdiction of public officials and the remedy is the tight construction of sewers and proper laying and location of gas pipes. The remedy in this case consists, therefore, in the enforcement of powers that municipalities at present possess.

Modern sewer design provides for the ventilation and inspection of the structure. The discharge of inflammable wastes into sewer systems would not of itself be a serious matter, or objectionable, were it not for the possibilities of igniting the explosive compounds. Ignition of gases in the sewers could be prevented by sealing all openings, but this would prevent inspection and create impossible working conditions inside the sewer when repairs became necessary. Moreover, the sealing of the sewers would not prevent ignition at chambers, pumping stations and disposal plants. In addition to the foregoing, there are other reasons which would make the sealing of the sewers impracticable and inadvisable.

The exclusion of inflammable wastes from a sewer system brings up the question of how it shall be accomplished. The regulations of the Municipal Explosives Commission of the City of New York, adopted Jan. 3, 1912, require the installation of oil separator traps or similar apparatus. The city of Boston requires a special trap which will prevent the discharge of the objectionable wastes into the sewers, and the city of Chicago has somewhat similar regulations to those of New York, governing this matter.

The efficiency of these devices is dependent upon the attention paid to their operation by the individual. Careless operation or neglect might render them of little value and defeat the purpose for which they were installed. Therefore, frequent inspection should be made by the proper public officers. Their general use on all sewer connections where gasoline or other inflammable waste is discharged would seem prohibitive, if found advisable, on account of the cost. The compulsory installation of devices for removing oil will generally meet with opposition by those affected, which has been recently demonstrated by the passage of an ordinance in New York City repealing the ordinance requiring the installation of oil separators. I am informed that this repealing ordinance was vetoed by the Mayor.

Formulation of legislation directed toward the prohibition of the discharge of inflammable wastes into sewers is at present receiving attention in many cities. That the same may be effectual requires the most careful consideration. It is most desirable that the necessity for such regulations be demonstrated and the efficiency of any devices thoroughly proven before they are required by ordinance.

The writer has examined the regulations of a number of cities and has come to the conclusion that it would be best from the standpoint of enforcement, to have all regulations of this nature contained in a single ordinance which would cover all phases of the discharge of wastes of all descriptions into the sewers. Such an ordinance should contain the following:

(a) Prohibition against the discharge of any inflammable gas, volatile inflammable liquid, inflammable liquid, oil or gas, or any calcium carbide or residue therefrom, or any liquid or other material or substance containing inflammable gas or which would evolve an inflammable gas when in contact with water or sewage.

(b) Regulations as to how sewer connections with establishments from which the foregoing wastes emanate may be made. This may or may not require the installation of special traps, separators or similar devices.

(c) Provision for the examination and approval of all intercepting devices and provision for their inspection, maintenance and operation.

(d) Provision with regard to the discharge or placing of obstructing material in any part of the sewer system.

(e) Regulations as to the discharge of steam or hot liquid or gaseous waste into the sewers.

(f) Regulating the location of gas pipes in city streets with reference to the sewer; prohibiting the placing of gas pipes close to or within the masonry of sewers.

(g) Prohibiting connections from manholes, gate boxes, or other apparatus of public service corporations to the sewers, except in an approved manner and when properly trapped.

Legislation alone will not secure or prevent the discharge of these objectionable wastes into sewer systems, but by informing the people of the damage resulting from this practice, the offense will be greatly lessened.

It would appear desirable, in the interest of public safety, where oil separators or similar devices are installed, for the municipality to undertake the final disposition of the residue rather than entrust it to the individual. The importance of the problem of regulating the discharge of inflammable waste, etc., and the necessity for its strict and effectual regulation has been amply and forcefully demonstrated by recent violent and destructive explosions.

As recent as Sept. 22, 1914, another serious explosion occurred in the sewer on East 42nd St., between 3rd Ave. and East River, New York, making the third explosion in the same sewer within a year. Reports state that the physical damage to the sewer, buildings and street, was not extensive. This is accounted for by the fact that the sewer was a brick lined tunnel in rock about 40 ft. below the street surface. Under less favorable conditions of location and design this result would have been far more serious.

The most disastrous and expensive sewer explosion up to the present time, although entailing no loss of life, occurred at Pittsburgh, on Nov. 25, 1913. This explosion to date has cost the city about $300,000, which may be increased by possible damage suits.

This problem is not confined to the prevention of explosions in the sewers themselves, but may extend to all kinds of sewerage works as shown by the explosion in the screen chamber at East Boston, which occurred June 1st, of this year. In this explosion, which was caused by the presence of gasoline vapor, six lives were lost and three men severely injured.

Without mention of other recent sewer explosions, it is evident from experience covering many cities, that an immediate, effectual and permanent remedy must be found to control the situation. With three explosions in the 42nd St. sewer in New York and two in the 33rd St. sewer in Pittsburgh, all within less than a year of each other, there can be no question but what the conditions inside of all large sewers draining garages, etc., are such as to produce explosions whenever ignition occurs. The safety of the public and the welfare of the community are therefore now dependent more upon good fortune than the certainty of scientific control, hence the public is always exposed to the hidden danger which only requires a chance spark to cause havoc and disaster.

The present situation can be likened to the man sitting on a keg of powder.

SEWER EXPLOSION INVESTIGATION AT PITTSBURGH.

The city of Pittsburgh, immediately after the second explosion, set about to make an investigation and study with a view of preventing a repetition of such disasters. This investigation is being conducted jointly by the city and the local office of the U. S. Bureau of Mines, who have rendered valuable assistance and advice.

The purpose of this investigation, which is still in progress, follows: (1) To locate all possible sources from which gasoline or other explosive wastes might enter the sewers. (2) To determine by a series of examinations and tests the location of the sources where the waste was discharged. (3) The determination of the presence, extent and quantity of vapor within the sewers. (4) Experiments to determine the effect and behavior of gasoline dumped into the sewer in different quantities and at different intervals.

The 33rd St. drainage basin was selected for study because of the number of garages within the basin and the fact that two explosions have already occurred, indicating the discharge of large quantities of explosive waste into the sewer. The 33rd St. drainage basin has an area of 1,642 acres, a population of 53,785, and a total of 417 structures where inflammable and explosive materials are handled. These structures are classified as follows: 295 small garages, not more than two cars, includes private and small business garages; 67 large garages, not less than three cars, includes public garages, repair shops, large business garages, etc.; 12 gasoline storage establishments; 1 dry cleaning establishment; 2 paint shops; 39 business or manufacturing places where inflammable oil or gases are manufactured, used, sold, handled or washed; includes gasoline supply establishments, large automobile establishments where gasoline is stored, sold and used in large quantities.

A map was prepared showing the outline of the basin, the sewer system within and the location of all garages, dry cleaning establishments and other places where inflammable or explosive wastes are likely to be discharged into the sewers. The map will be used in connection with studies to locate the point or points where the discharge of inflammable wastes occur.

Letters of inquiry were sent to all the principal cities in the country with a view of obtaining data and information relative to sewer explosions and what laws or ordinances were in force regulating or controlling the discharge of inflammable or explosive wastes into the sewerage systems. The answers received in reply to those inquiries were compiled and have been printed in pamphlet form and copies furnished to each city supplying information.

Apparatus was designed for securing samples of air within the sewer and for making field tests of same. The apparatus used is shown in the accompanying cut.

The results so far secured in the investigation indicate the presence of gasoline vapor in the sewers of both the 33rd St. and Negley Run systems. This latter system drains an area of about 2,500 acres with a population of about 50,000 and there are considerably less sources from which inflammable wastes are discharged than in the 33rd St. system. Analyses of a series of samples taken on the same day at various points in these systems have shown that gasoline vapor in small amounts is present throughout the sewer system. The gasoline vapor ranges from 0.012 to 0.065 per cent of the volume of sewer air in the sample. While these percentages of gasoline vapor are considerably below the danger mark, which may be taken as 2 per cent, it goes to show that the natural ventilation of these sewer systems is not sufficient to remove the effects of the ordinary or normal discharge of gasoline.

The Negley Run system drains through duplicate outlet sewers for a distance of over a mile, during which distance there are no connections known which could by any possibility discharge gasoline. Above this point, there are a number of large branch sewers of considerable length so that taking these larger sewers of the system together with the many miles of laterals, with the opportunity for ventilation provided, it must follow that natural ventilation would not suffice to remove the effects of the discharge of gasoline in large doses.

It is expected that these experiments will require considerable time before definite conclusions can be reached and preventive measures, based upon them, can be formulated, but it is hoped that some plan or action can be devised which, without imposing hardship or undue expense upon the people, will secure to them freedom from the peril and danger to which they are now constantly exposed.

The Milwaukee Sewerage Problem.

[illegible] 1914, you printed a portion of a paper entitled "The Milwaukee Sewerage Problem," which I presented at the annual convention of the American Society of Municipal Improvements recently held at Boston. In this paper I stated that the Menominee Sewerage Commission, consisting of Messrs. John W. Alvord, George C. Whipple and Harrison P. Eddy, had recommended that the preliminary treatment of sewage in Milwaukee should consist of chem[illegible] precipitation [illegible] I made an error in this statement which I wish to correct. What this Commission did recommend as a preliminary treatment for the sewage of Milwaukee, was grit chambers, screens and sedimentation tanks for the removal of suspended matters and a disinfecting station—the effluent therefrom to be discharged into the Kinnickinnic River. When this River could no longer provide dilution for sufficient oxidation, the effluent was to be carried through a 13 ft. tunnel under the bottom of Lake Michigan and dispersed on the bottom about 1⅔ miles from the shore. The Commission suggested that, in order to delay for some years the building of this large outfall tunnel, provision might be made for temporarily treating the sewage by chemical precipitation. It further suggested that Imhoff tanks might prove more advantageous than shallow horizontal-flow well baffled tanks, and recommended a careful study be made of the adaptability of these tanks to the conditions in Milwaukee.

In the event that sedimentation, disinfection and dispersion in the lake did not satisfactorily dispose of the sewage, the Commission recommended that the tank effluent be further treated by sprinkling filters.

Very truly yours,

T. Chalkley Hatton, Chief Engr.,
Sewerage Commission of Milwaukee.

Milwaukee, Wis., Nov. 9, 1914.

BRIDGES

Design and Construction of the Lower Ganges Bridge in India.

(Staff Article.)

The Lower Ganges Bridge, which is under construction near Sara, India, about 115 miles northeast of Calcutta, will connect the 5-ft. 6-in.-gage system of the Eastern Bengal Railway south of the Ganges River with the railways north of the river. The bridge, which has a total length of about 5,900 ft., consists of fifteen through truss spans of 345 ft. 1½ in. each, center to center of bearings, and symmetrical steel approaches, each of which consists of three 75-ft. deck girder spans.

The piers of the truss spans are spaced 359.1325 ft. on centers, and their caisson foundations extend to a maximum depth of 160 ft. below low water level, the total height of these piers and their foundations being 222 ft. 6 ins. The bridge has a clearance of 40 ft. at high water, the variation between high and low water levels being 31 ft.

A bridge at this site was proposed more than 20 years before its construction was sanctioned in 1908. Proposals put forward by the Eastern Bengal Ry. in 1889 were investigated by a committee and reported feasible. In 1902 Mr. F. J. E. Spring pre[illegible] committee representing commercial interests recommended one of the "Sara" sites, which also had been favored by Mr. Spring. A technical committee appointed in 1908 reported that a bridge could be constructed and advocated a site near Sara. This was followed by the sanction of the Secretary of State to the construction of the bridge at or near Sara and the appointment of Mr. R. R. [illegible] 1908. Earlier proposals projected a single-track bridge with a clearance at high flood level of 33½ ft. and protection works on each bank. Further investigation and experience with the vagaries of the river resulted in the provision, in the final design, for a double-track bridge with a clearance at high water of 40 ft., a public footway, and protection works at Raita and Sara, in addition to more strongly built protection works at the site of the bridge.

APPROACHES AND PROTECTION WORKS.

The portion of the line which includes the bridge and its approaches is about 15 miles long. It starts just north of the Bhairamara station on the Eastern Bengal Ry. (see Fig. 1) and joins the northern section of that railway about three miles north of Gopalpur. From the south (Calcutta) side the approach line to the bridge has a slope of 0.25 per cent for a distance of about 3¼ miles. On the north (Siliguri) side the track is level for 2,000 ft. at Paksey station and then grades downward for about 3¾ miles at a slope of 0.20 per cent. At the abutments the approach banks are 50 ft. high. Figure 2 shows a plan of the protection works at the bridge site and the manner of providing access to them.

Fig. 1. Site of Lower Ganges Bridge and River Bank Protection Works.

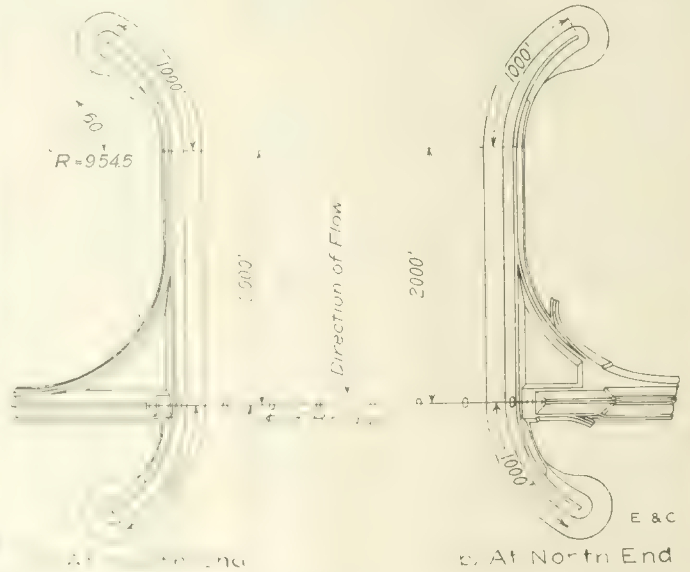


Fig. 2. Plans of Protection Works at North and South Ends of Lower Ganges Bridge—Note Approaches to Protection Works.

There are five roads under bridges through the high approach bank and two small culverts for local drainage. There are also four culverts beyond the northern approach grade for local drainage. These culverts are heavily protected against failure during floods. From the Bhairamara station on the south bank to the Ishurdi station on the north bank (see Fig. 1) the railway is double track; beyond these stations it is single track.

The problem of keeping the river to its course through the bridge was the most difficult one which confronted the engineers. The system adopted is that originated by the late Mr. J. R. Bell for rivers with erodible banks. At the bridge site there are two guide banks of the "Bell" embankment type. Each protection embankment is 4,000 ft. long, extending 3,000 ft. above the bridge and 1,000 ft. below it (see Figs. 1 and 2). The ends are curved inward and are heavily pitched with stone. The embankment consists of a core of earth raised 18 ft. above the highest flood level, protected on its river slope with a covering of pitched stone 2 ft. to 3½ ft. thick. From the toe of the slope an apron 4½ to 8½ ft. thick is spread outwards a distance of 150 ft. As the river cuts away the earth from under the outer edge of this span, the stone falls and eventually the entire apron will lie at a slope equal to that of the pitched slope of the bank to a depth of 100 ft.—the maximum known depth of bend scour. It has been possible to make these guide banks shorter than would otherwise have been necessary, due to the existence at Sara (three miles above the bridge on the north bank) and at Raita (seven miles above the bridge on the opposite bank) of benches of clay of considerable extent which form semi-permanent points in the banks of the river (see Fig. 1). To render these points permanent, and thus confine the river to a comparatively straight approach to the bridge, they have been reinforced with pitched-stone guide banks and aprons, each 4,000 ft. long.

SUBSTRUCTURE OF BRIDGE.

Caisson Foundations.—The sixteen main piers are founded on caissons sunk by the open dredging process to a depth of from 150 to 160 ft. below low water level. The subsoil through which the caissons were sunk consists of sand, with occasional strata of stiff clay. These caissons rest on a bed of sand, which lies about 50 ft. below the deepest known bend scour line of the river, and they are the deepest foundations of their kind in the world. The load on the base of the caissons, allowing for buoyancy but not for skin friction, is about 9 tons per square foot.

In plan the caissons have semi-circular ends and straight sides. These caissons are 35 ft. wide, 63 ft. long and from 150 to 160 ft. deep. Figure 3 shows a half elevation and a half longitudinal section of a typical caisson and masonry pier, and indicates the type of construction used. The caissons contain two dredging holes, each 18½ ft. in diameter. The curb (cutting edge) has a height of 15 ft. 7 ins. and weighs 140 tons. It is continued upward as a caisson by adding steel plates in unit lengths of 7 ft. to a maximum height of 65 ft. Above this level the dredging wells are lined with steel to insure water-tightness and to give increased strength. This steel lining also acts as a form for the mass concrete filling between the wells. Above the steel enclosed portion the walls consist of moulded concrete blocks weighing about 6 tons each, these blocks being carried up to the low water level. The walls are strengthened with eighteen 1¾-in. vertical rods and 6x¾-in. bond rings spaced on 12 ft. centers. After the caissons have been sunk to their final depth the tops and bottoms of the dredging wells are plugged with concrete, and the space between the plugs is filled with sand (see Fig. 3).

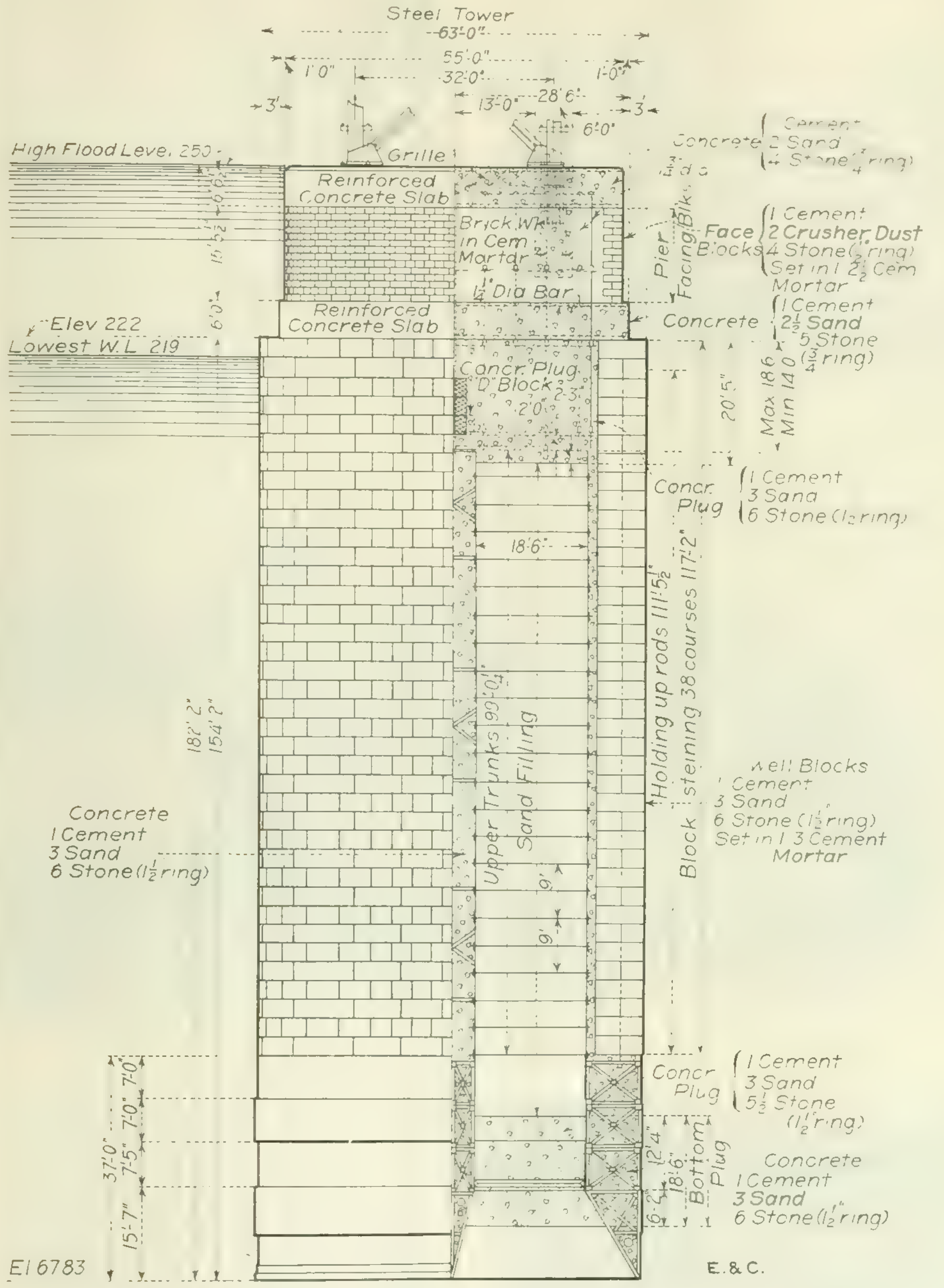


Fig. 3. Half Elevation and Half Longitudinal Section of Typical Caisson and Masonry Portion of Pier for 345-ft. 1½-in Truss Spans of Lower Ganges Bridge.

Some of the curbs, or cutting edges, were erected on the bank during the dry season, and were floated into position during the rainy season; others were launched from slips similar to those used for launching ships; most of them, however, were built on wooden pontoons and were launched by sinking the pontoons. On account of magnitude of the work, the heavy loads to be handled, and the fact that it was necessary to sink each well to its full depth in a single season (to avoid the following floods), special overhead floating stagings, equipped with electric cranes and concrete mixers, were used.

In general, the procedure followed in constructing this part of the work was as follows: The cutting edge was floated into position and placed in the staging, and the latter then moored in place. The plates were then built up, one by one, and the caisson gradually sunk to a level near the bed of the river by filling it with concrete. The caisson was grounded quickly, both to avoid the scouring of the river bed under it and to insure its being placed in exact position, the bed of the river being protected previously with a layer of sand bags. To insure a quick grounding the dredging holes were provided with water-tight false bottoms, which, when opened, permitted the caisson to sink rapidly to a firm bearing on the sand bags. The false bottoms were then removed, and the remainder of the space filled with concrete. The caisson was then sunk by dredges equipped with buckets of 100 cu. ft. capacity, the concrete blocks being built up as the sinking progressed. Each caisson contained approximately 355 tons of steel and 15,300 tons of masonry.

Piers.—To reduce the weight on the foundations it was necessary to build the upper portions of the piers (above high water level) in the form of steel towers. These towers rest on masonry piers, which are 29 ft. wide and 55 ft. long and have semi-circular ends and straight sides. The pier shafts consist of brickwork laid in cement mortar and faced with moulded concrete blocks. The bases of the piers, which rest directly on the caissons, are of concrete 6 ft. thick. The pier shafts are capped with a reinforced concrete slab 6½ ft. thick, enclosing the heavy steel grillages upon which rest the tower legs (see Fig. 3).

The two abutment-piers have been constructed of masonry to their full height, as the loads carried by them are much lighter. (Each of these piers supports one end of a 345-ft. 1½-in. through span and one end of a 75-ft. deck plate girder.) The bases of these abutment-piers are of reinforced concrete, about 11 ft. thick. Above the bases the piers consist of concrete blocks with a core of mass concrete, while the shafts are capped with a 3-ft. reinforced concrete slab enclosing the grillages. In plan, these piers are rectangular, their length being 45 ft. 6

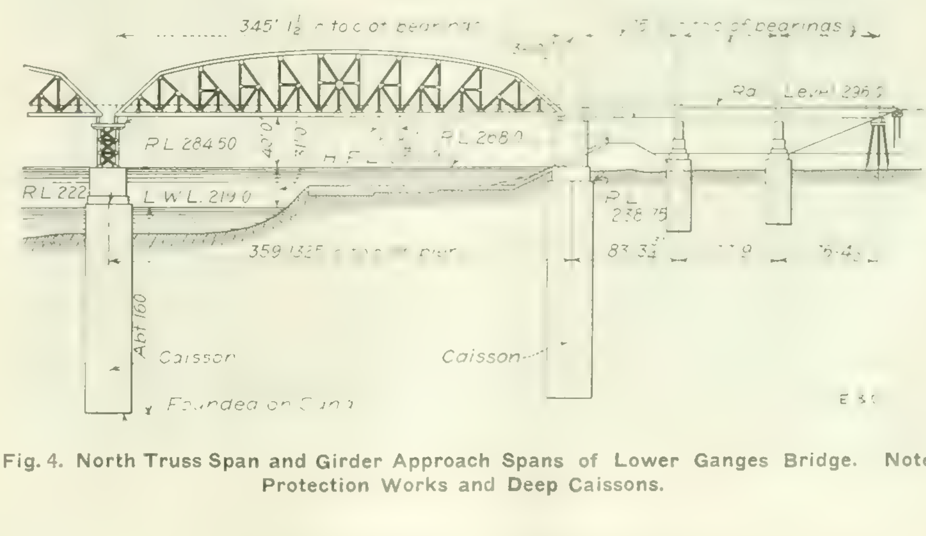


Fig. 4. North Truss Span and Girder Approach Spans of Lower Ganges Bridge. Note Protection Works and Deep Caissons.

ins. and their width 27 ft. 6 ins., with rectangular recesses on the river sides. In addition to reducing the weight of the piers these recesses tend to centralize the load on the caissons.

Fig. 5. View Showing Manner of Erecting South Truss Span of Lower Ganges Bridge—Note Falsework and Gantry Crane.

The masonry piers were constructed by means of the same stagings which were used in sinking the caissons, while the steel towers were erected by 10-ton electric floating cranes. The sixteen piers and towers contain 3,150 tons of steel and 47,800 tons of masonry.

SUPERSTRUCTURE OF BRIDGE.

Design Features.—The fifteen main through truss spans are of the Petit type. Each span has a length, center to center of bearings, of 345 ft. 1½ in., and a maximum height, center to center of chords, of 52 ft. The bridge carries two 5-ft. 6-in.-gage tracks and a 5-ft. footway bracketed from the downstream truss. The trusses are spaced 32 ft. on centers, each truss consisting of twenty-two 15-ft. 8¼-in. panels. The trusses rest on cast steel rocker bearings about 5 ft. 6 ins. high. Each main span weighs about 1,250 tons.

The steel approach at each end of the bridge consists of three 75-ft. deck plate girders, with a trough deck on top carrying the two tracks and with parapets on both sides.

Figure 4 is a general elevation of the truss span and approach at the north end of the bridge. This drawing gives the general dimensions and types of construction used, and shows the protection works at this end of the bridge.

Erection Features.—Where the river is shallow the main spans were erected on timber falsework, which was supported on piles where necessary. In deep water a 345-ft., 1,000-ton steel erection span was used. This span was supported on the masonry piers during the erection of the through truss spans upon it. The erection span was transported from span to span on two pontoons, upon which were placed timber stagings of adjustable height. Figure 5 is a view taken during the erection of the end span on the south bank of the river, while Figs. 6 and 7 are views showing the erection of a central span in deep water. It is expected that eight spans will be erected on timber falsework and seven by means of the steel erection span.

The main spans are erected by means of an electrically-driven gantry traveler running over tracks supported either on the erection span or on the falsework (see Figs. 5 and 6).

The total weight of the steelwork in the superstructure is about 21,000 tons. The 60,000 field rivets in each truss span were driven by hydraulic and pneumatic riveters. About 4,000 tons of pine timbers were obtained from America for the falsework, the fittings for which weighed about 40 tons.

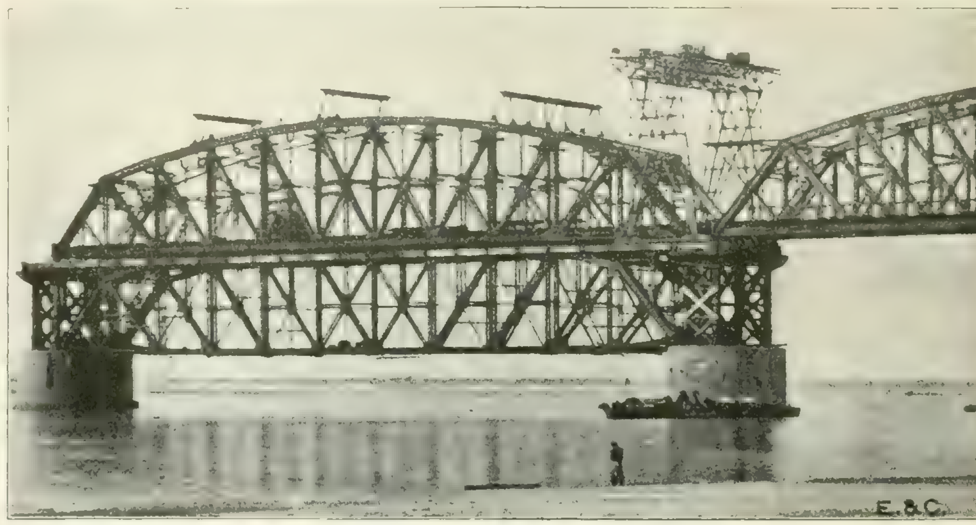


Fig. 6. View Showing 345-ft. Erection Truss Used in Erecting Main Spans in Deep Water—Lower Ganges Bridge.

Fig. 7. View of Lower Ganges Bridge Showing Erection of a Main Span on the Erection Truss—View Also Shows a Completed Span and Pier.

MATERIALS.

The stone for the protection works, that for the concrete, and for ballasting the line was quarried at three bridge quarries, one located at Phudkipur, near Rajmahal, one at Pakur, and the third at Jainti, at the foot of the Himalayas. The stone was transported from Pakur and Jainti, distant 234 and 218 miles from the bridge site, respectively, by train, and from Phudkipur, a distance of 134 miles, by a chartered fleet of steamers and flats. The building sand was delivered by train, while the cement was obtained from England through the stores department of the India office.

The concrete blocks for the caissons and for facing the piers were moulded in yards located on both banks of the river. Each yard has a full equipment, including a 10-ton, 53-ft. span traveling crane, concrete mixers, stone crushers, tramways, moulds, etc. On the north bank the yard had a special moulding shed with an overhead trolley gantry for manufacturing and handling the facing blocks for the piers. About 24,000 caisson blocks and 16,000 facing blocks were moulded in these yards.

EQUIPMENT AND POWER.

The truss spans were received in special yards and were unloaded by 20-ton steam traveling jib cranes onto skids, on which the pieces were sorted and stacked. All material consigned to the north, or Sara, bank was transported by ferry across the Ganges River by the Eastern Bengal Ry. and was run into the storeyards. The materials for the river works were taken out on pontoons and barges, being handled by floating cranes.

The power and light are supplied from two powerhouses, one on each bank of the river. The capacity of these plants is 1,125 kw. The alternating current is generated at 440 volts; it is transmitted at 3,300 volts; and is stepped down to 440 volts for the machines and to 110 volts for the lights.

The construction plant is a large item in the cost of this project, some of the important equipment being recorded in the accompanying table:

Two erection travelers.
Two 10-ton, 53-ft. span traveling cranes.
Two 20-ton steam traveling jib cranes.
Three 80-ft., 10-ton, floating derrick cranes.
Five fixed and traveling 10-ton hand cranes.
Ten 15-ton overhead cranes on the caisson sinking sets.
Two paddle steamers, 3 steam launches, 3 motor boats and about 50 pontoons and barges.
Broad-gage (5-ft. 6-in.), meter-gage and 2-ft. 6-in. gage rolling stock, including 24 locomotives, 28 brakevans and about 830 trucks.
About 81 miles of 5-ft. 6-in. gage, meter-gage, and 2-ft. 6-in. gage service track.
Pneumatic and hydraulic riveting plants.
Twenty-one concrete mixers and 5 stone crushers.
Power plant, equipment, etc.

PROGRESS OF CONSTRUCTION AND GENERAL DATA.

Preliminary surveys and prospecting for sources of stone supply were carried on during the early part of 1909. The season of 1909-1910 was occupied in opening up quarries, constructing service lines, offices, and staff quarters at the bridge site. The season of 1910-1911 was spent in completing the service works and in constructing the principal part of the north and south bank protection works. During the season of 1911-1912 the protection works at the bridge site were completed, the "Raita" bank works started, and five caissons were sunk to their full depth. During the season of 1912-1913 seven more caissons were completed, the "Raita" bank was practically finished, and one main span and the erection truss erected. Further progress might have been made on the truss spans and piers if the material had arrived from England on scheduled time, considerable delay in supplying the steelwork being due to strikes in England. The first main span and the erection truss were erected very late in the season, under great pressure and considerable risk from the rising river. Their completion, however, saved about a year in the time required for the completion of the bridge. During the current season it is expected that the four remaining caissons and all the piers will be completed and that ten main spans will be erected. It is also expected that all of the approach works, stations and land spans will be completed by the end of the rainy season. It is proposed to open the bridge to traffic in 1915. The north, or "Sara" bank, cannot be constructed until the traffic has been diverted over the bridge, as this work involves the dismantling of the existing pass at Sara.

About 299,000 tons of masonry and nearly 30,000 tons of steel will be required for the superstructure and substructure. At high flood level about 2,500,000 cu. ft. of water will pass the bridge per second. The bank protection works, when completed, will contain about 865,500 cu. yds. of pitching stone and about 1,429,600 cu. yds. of earthwork, while a reserve supply of stone of about 104,000 cu. yds. is stored near the bridge site. In each main span there are about 60,000 field rivets, and in each pier and caisson about 50,000 field rivets, a total of about 1,700,000 field rivets. The deepest caisson has been sunk to a depth of 159.6 ft. below lowest water level, or 190.6 ft. below flood level. The estimated cost of the work is as follows:

Bridge proper—	
Protection works	$ 3,633,000
Piers and steel spans	6,308,000
Construction plant, general charges, service works and preliminary expenses	2,757,000
Total	$12,698,000
Approaches (beyond steelwork)	2,724,000
Grand total	$15,422,000

In February, 1912, there were 24,400 men employed on various parts of this work, which gives some idea of its magnitude.

PERSONNEL.

The superstructure was designed under the British standard specifications by Rendel, Palmer and Tritton, consulting engineers of the India office. The Cleveland Bridge & Engineering Co., Ltd., built nine of the main spans and Braithwaite & Kirk, Ltd., the remaining six spans. We are indebted to the Railway Board of the Railway Department, Government of India, for the data upon which this article is based.

Results of Tests of Treated and Untreated Oregon Fir Piling to Determine the Effect of the Steaming Process of Creosoting on the Physical Properties.

The Atchison, Topeka & Santa Fe Ry. has recently made a series of tests to determine the effect of the steaming process of creosoting on the physical properties of Oregon fir piling. The material tested was taken from the regular stock of a treating plant, and it was considered representative timber as to density of grain, sap wood, knots and conditions of creosoting. The sap wood, however, was deeper than that ordinarily found in average material. The tests were made by H. B. MacFarland, engineer of tests, and were reported in the Bulletin of the American Railway Engineering Association, the following data being abstracted from this report:

Scope and Character of Tests.

MATERIAL TESTED.

Twenty Oregon fir logs, varying in diameter at the butt from 12 to 18 ins. and in length from 40 to 60 ft. were selected for the test. From each log two specimens, each 15 ft. long, were cut. Ten of the 40 specimens were given the regular treatment, which is known as the 18-hour treatment, and ten were given an extra heavy 26-hour treatment for the specific purpose of obtaining deeper penetration; the remainder were tested without treatment. In order to get the best comparative data as to the effect of creosoting on the different parts of the log, alternate butts and tops were treated. At the time of treatment the logs had been cut from three to six months. Just previous to treatment the logs were taken from salt water and the bark removed, the logs then being ready for treatment.

The "major" tests were made of the entire section of the log, while the material used in the "minor" tests was taken from the hearts of the treated and untreated major specimens used in the transverse tests.

DESCRIPTION OF TREATMENT AND TESTS.

Treatment.—Table I gives details of the creosoting treatment of the specimens, the process consisting of three parts, namely, steaming, removing moisture and filling with creosote. Observations as to temperature, pressure and vacuum were taken at ten-minute intervals throughout the tests.

TABLE I.—DETAILS OF CREOSOTING TREATMENT.

Time of Start. Stop.	Total time, h.	m.	Pressure, lbs. per sq. in. Max.	Min.	Av.	Vacuum, ins. Max.	Min.	Av.	Temperature, deg. F. Max.	Min.	Av.
18-HOUR TREATMENT.											
Steaming Process: Tank 1, Tank 2	3	[illegible]	92	7	[illegible]	..	.		335	275	317
Vacuum Process: Tank 1, Tank 2	10	10	.			[illegible]	21	25	265	190	219
Oil Process: Tank 1, Tank 2	4	[illegible]	114	10	77				204	190	196
26-HOUR TREATMENT.											
Steaming Process: 1:50 p. m., 2:00 a. m.	9	10	[illegible]	20	[illegible]				330	200	300
Vacuum Process: 2:10 a. m., 3:00 p. m.	12	50	...	...	...	26	11	[illegible]	276	206	225
Oil Process: 3:50 p. m., 7:55 p. m.	4	5	[illegible]	[illegible]	..				201	186	190

Tests—Major Specimens.—The transverse tests were made by placing the specimen on two 18-in. I-beams which were set across the table of a 200,000-lb. Olsen testing machine. The specimen was supported by two concave oak blocks 6 ins. wide and 12 ins. long, fitting the contour of the pile. The load was applied at the third points on concave oak blocks similar to those used to support the piling, the length of span being 13 ft. The load was applied in increments of 5,000 lbs., the head of the screw descending at the rate of 0.15 in. per minute until failure occurred.

In making the compression tests parallel to the grain a specimen 5x5x12 ins. was taken, care being taken that the specimens were free from the effects of the transverse test. The ends were made parallel and perpendicular to the axis of the piece by shaving them in a wood trimming machine. The load was applied in increments of 5,000 lbs., the speed of the machine being 0.15 in. per minute.

The specimens used in making the compression tests perpendicular to the grain were 6x6x24 ins. To obtain comparable data the specimens were tested in such a way that the load was applied through a plate 2x3⅞x8 ins. to the convex side of the growth rings. The load was applied in increments of 5,000 lbs. up to about the elastic limit when the increments were decreased to 1,000 lbs.

In making the shearing tests parallel to the grain the specimens used were 3 11/16 x 3 11/16 x40 ins. long, an opening 1½x4 ins. being cut 6 ins. from the end. A machined piece of iron 1½x3 ins. was placed in the mortise, the rate of motion of the machine head being 0.16 in. per minute.

Tests—Minor Specimens.—The "minor" tests were made to determine whether the wood in the interior and in the untreated portions of the creosoted piling was injured in any way by the creosoting process.

The transverse test specimens were 5x5x60 ins., the span being 50 ins. These specimens were placed with the side nearest the center of the original pile downward. The wood was protected by iron plates resting on the knife-edge supports and load points. The loading was applied at the third points, the load increments being 200 lbs.

The compression tests parallel and perpendicular to the grain were made on material sawed from the transverse test specimens.

Discussion of Results of Tests.

By means of descriptions, diagrams, photographs and tabular data the test of each specimen is fully covered in the bulletin. The preceding data and the following discussion of the results of the tests, however, are sufficient to indicate the character and scope of the tests and to give the effect, on the physical properties, of creosoting Oregon fir piling.

GENERAL CONDITIONS—MAJOR SPECIMENS.

Physical Defects.—It was noted from a study of the photographs of end views of treated and untreated specimens that the treated material, in general, showed clearly the effect of high temperature from pressure used in the steam creosoting process.

The original defects in untreated specimens are, as a rule, amplified by the treatment. Checks and cracks are enlarged, and in many cases the cohesion of annual growth rings has been lessened to such an extent that separation of the rings in the form of a circular check takes place, particularly where there are pitch rings.

MOISTURE CONTENT.

It is seen from the following data that the moisture at the time of test in the treated material is greater than that in the untreated material:

	Moisture content, per cent. Untreated.	Treated.
18-hour treatment	12.6	15.7
26-hour treatment	14.8	17.9

The seaming and vacuum process, instead of removing moisture from the material, introduced 3.1 per cent.

PENETRATION OF CREOSOTE.

Microscopic Examinations—Sap and Heart Wood.—It was observed, by examining very thin sections of treated and untreated Oregon fir under the microscope, that the sap wood consisted of small fibers of cellulose, approximately 0.0006 in. in diameter; that is, in a field 0.06 in. in diameter there were 100 fibers. The fibers were separated by small ducts containing coagulated sap. Resins were entirely absent. The heart wood appeared to be little different from the sap wood in size or in fineness of fiber. The ducts contained resinified sap. Minute pockets of resin were easily discerned.

Spring and Summer Wood.—On examination of the cross section of wood, the alternate soft and hard rings are easily seen. The soft rings consist of somewhat spongy material and are called "spring wood." The rings of spring wood are surrounded by rings of "summer wood." As a rule the rings of summer wood are much narrower than those of the spring wood. The summer wood, by reason of the greater vitality of the tree in summer than in spring, has a greater strength and density.

Under the microscope it was observed that the summer wood had a well defined fibrous structure, indicating a certain measure of capillarity. The spring wood, however, was poorly organized; the fibers of cellulose were widely separated by ducts probably containing albumin, starch and unorganized cellulose.

Penetration in Heart Wood.—It was noted from the photographs of cross sections of treated specimens that the treated areas were sharply defined. The untreated areas consisted of heart wood.

The impenetrability of heart wood in regard to absorption of creosote is seen by comparing the depth of penetration in treated specimens with the depth of sap wood of untreated specimens. It is to be noted that the depth of penetration in 75 per cent of the treated specimens corresponds very closely to the depth of sap wood in the untreated specimens.

The heart wood in Oregon fir is so refractory that it is almost impossible to bring about an absorption of creosote, and then only to a slight degree, even in perfectly seasoned material, without the use of great pressure and high temperature.

It was observed from the photographs of end penetration that the ends of the fibers in cross sectional treatment are almost totally resistent to treatment, indicated by the absence of any dark coloration in the ends of the specimens. That portion of the cross section containing sap wood undoubtedly takes up considerable treatment longitudinally. Wood containing checks, cracks, shakes, loose open fibers, and other physical defects, absorbs considerable creosote longitudinally by reason of the pressure used in the creosoting process.

Penetration in Summer Wood.—It was noted that there is practically no end penetration whatever in a perfectly sound specimen of Oregon fir. The dark striations separated by margins of uncolored wood, coinciding with the rings of summer wood, extended for some distance into the wood. These striations are due to longitudinal absorption by summer wood of a very small amount of creosote. They often extend to considerable distances into the treated specimens.

It is often seen in boring into a treated pile or bridge timber perpendicular to the grain that dark colored fiber is nearly always found well outside the zone of active penetration.

The alternate areas of treated and untreated material are easily seen by examination of the core extracted by an increment auger.

The dark portions of treated in untreated material are due to longitudinal capillary absorption of creosote by summer growth rings.

It is doubtful if the amount of treatment contained in the summer growth rings of heart wood is sufficient to resist the incursions of the "Teredo Navalis."

In measuring the depth of penetration of treatment in creosoted Oregon fir, that portion which contains *active* penetration should be considered and not the dark striations outside the zone of penetration.

PHYSICAL TESTS—MAJOR SPECIMENS.

Transverse Tests.—The number and kinds of failures of treated and untreated specimens in transverse test are as follows:

	Horizontal shear. Number.	Per cent.	Tension. Number.	Per cent.
Treated	13	65	7	35
Untreated	18	90	2	10

It is seen that 65 per cent of the treated specimens and 90 per cent of the untreated specimens failed in horizontal shear. The fact that more untreated than treated specimens failed by horizontal shear was due to the fact that the outer fiber in the untreated specimens was much stronger than in the treated. In the treated specimens the outside fiber was weakened to such an extent by creosoting process that in some cases failure occurred in tension before a stress sufficient to shear the specimens was reached. The failure in some specimens was influenced by knots and checks.

It has been found in tests on rectangular beams of treated and untreated Oregon fir that the common failure in treated specimens was by horizontal shear. In the untreated specimens, half failed by horizontal shear and half by tension. The reason that horizontal shear failures are more common in treated material having rectangular cross section than in treated material of circular cross section is due to the different effects of vertical stress in transverse test on the rectangular and circular cross section. In the rectangular cross section the horizontal shear is of greater intensity than in a circular cross section of the same area for the same loading.

Modulus of Elasticity.—In order to show the effect of treatment on the strength of Oregon fir piling, the following table of values giving comparative results is shown:

	Modulus of elasticity, lbs. per sq. in. Maximum.	Minimum.	Average.	Rating, per cent.
Untreated	2,235,000	1,384,000	1,846,000	100
Treated	1,692,000	990,000	1,328,925	72

The above rating is based on taking the modulus of elasticity of untreated specimens as 100 per cent.

It is to be observed that the modulus of elasticity of treated specimens is 28 per cent less than that of the untreated specimens. The modulus of elasticity is, of course, influenced by knots, checks and other defects, which cause early failures.

The minimum value, 990,000 lbs. per square inch was obtained from specimen 16B, which not only had many defects, but appeared to have been injured by the creosoting process.

The general average for untreated specimens is high, due to the excellent quality of the material and to good seasoning.

Modulus of Rupture.—For comparison, a table containing the average modulus of rupture, treated and untreated specimens, is here given:

	Moisture, per cent.	Modulus of rupture, lbs. per sq. in. Maximum.	Minimum.	Average.	Rating, per cent.
Untreated	13.68	8,233	6,002	7,028	100
Treated	16.70	5,215	2,206	4,058	58

The above rating is based on taking the modulus of rupture of untreated specimens as 100 per cent.

It is to be noted from this table that the transverse strength of the treated specimens is little more than half as great as that of the untreated. It is also seen that there is very wide divergence between the maximum and minimum moduli of rupture of treated specimens. The minimum was obtained from specimen 16B.

It was noted from the modulus of rupture (transverse tests of untreated specimens) that specimen 16A, the untreated portion of the pile, was 3½ times as strong as the treated specimen 16B. The cause of this great diminution in strength and quality was probably due largely to treating conditions. Specimen 16A contained both old and new growth, indicated by the widened growth rings in the outer layers of sap wood. This timber was abnormal in that new growth had been added to the old.

This condition was probably due either to disease or natural causes. The defective cell structure, although not revealed by transverse test of the untreated portion of the timber, rapidly deteriorated under treatment conditions, causing an early failure in transverse and compression test.

TABLE II.—COMPARATIVE STRENGTHS OF BUTTS AND TOPS OF PILING.

	Transverse test.		Compression test. Parallel to grain.		Compression test. Perpendicular to grain	
	Modulus of rupture, lbs. per sq. in.	Rating, per cent.	Maximum load, lbs. per sq. in.	Rating, per cent.	Load, lbs. per sq. in., deflection ⅜ in.	Rating, per cent.
Untreated:						
Butts	6,827	100	3,586	100	1,020	100
Tops	7,229	106	3,539	99	995	98
Treated:						
Butts	4,530	100	2,959	100	765	100
Tops	3,586	79	2,172	73	610	80

COMPRESSION TESTS.

Parallel to Grain.—The following table shows comparative results of compression tests parallel to the grain of treated and untreated material:

	Elastic limit, lbs per sq. in	Maximum load, lbs. per sq. in.	Modulus of elasticity, lbs. per sq. in.	Rating, per cent
Untreated	3,003	3,563	325,940	100
Treated	2,257	2,595	252,770	73

The above rating is based on taking the maximum load, in pounds per square inch, in untreated specimens as 100 per cent. It is seen from the table that the treated material is 27 per cent weaker than the untreated, while in transverse test the treated specimen is 42 per cent weaker. The effect of abnormal temperature and pressure is to bring about a greater loss in transverse than in compression strength.

Perpendicular to Grain.—A comparison of compression tests perpendicular to the grain, treated and untreated specimens, is as follows:

	Elastic limit, lbs. per sq. in.	Load, lbs. per sq. in. for deflection of ⅜-in.	Rating, per cent.
Untreated	664	1,007	100
Treated	313	687	68

The above rating is based on taking the load, in pounds per square inch, for a deflection of ⅜ in. in untreated specimens as 100 per cent.

It is seen that the loss in compressive strength of treated material is 32 per cent. This loss is due to the softening of the fiber by the steaming process. It was noted in making compression tests perpendicular to the grain that the fiber appeared to be very elastic. After failure by crushing the specimen returned almost to its original shape.

Shearing Strength.—The following table gives the horizontal shearing strength in transverse test, and the shearing strength parallel to the grain:

	Maximum.	Minimum.	Average.	Rating. per cent.
	Horizontal shear, lbs. per sq. in.			
Untreated ..	352	262	318	100
Treated	254	83	180	52
	Shear parallel to grain.			
Untreated ..	391	250	311	100
Treated	340	136	217	70

It is seen from the above data that the loss in horizontal shear in the treated specimens is 48 per cent, and in shear parallel to grain, 30 per cent.

Creosoting processes greatly diminish the cohesion of fibers in Oregon fir. The loss in shearing strength is a serious defect not only in bridge timbers, but also in piling. Treated bridge timbers usually fail by shearing rather than by tension. The method of driving piling causes severe and sudden vertical stresses to be applied, and in case there is especial weakness in shearing strength, failure is brought about by the "shelling off" of annual growth rings under the blows of the pile driver.

Comparative Strength Butts and Tops.—The comparative strength of tops and butts of treated and untreated Oregon fir piling is given in Table II. These results are based on taking the strength of the butts as 100 per cent.

It is seen from the above data that there is little difference in the strength of butts and tops of untreated specimens, but the treated tops are considerably weaker than the treated butts. This, no doubt, is due to the fact that, although the depth of sap wood is the same in untreated and treated timbers, the actual percentage of sap wood is greater in the tops than in the butts, due to the smaller cross sectional area. As a result, the percentage of weakened fiber due to treatment in the top is greater than in the butt, thereby causing a lower stress in pounds per unit area to produce failure.

Comparison of Strength—18-Hour and 26-Hour Treatment.—In order to show the effect of length of time of steaming on the strength of the material, Table III is given.

TABLE III.—COMPARATIVE STRENGTHS FOR 18 AND 26-HOUR TREATMENT.

	Transverse test.		Compression test. Parallel to grain.		Compression test. Perpendicular to grain.	
	Modulus of rupture, lbs. per sq. in.	Rating. per cent.	Maximum load, lbs. per sq. in.	Rating. per cent.	Load, lbs. per sq. in., deflection ⅜ in.	Rating. per cent.
18-hour	2,722	100	2,705	100	715	100
26-hour	2,405	88	2,486	92	660	92

The ratings in this table are based on taking the values for the 18-hour treatment as 100 per cent. The results of the tests indicate that the 26-hour treatment is more detrimental to the strength of material than is the 18-hour treatment.

PHYSICAL TESTS—MINOR SPECIMENS.

Table IV gives the comparative results of transverse and compression tests of treated and untreated major and minor specimens.

TABLE IV.—RESULTS OF TESTS OF TREATED AND UNTREATED SPECIMENS.

	Transverse test.		Compression test. Parallel to grain.		Compression test. Perpendicular to grain.	
	Modulus of rupture, lbs. per sq. in.	Rating, per cent.	Maximum load, lbs. per sq. in.	Rating, per cent.	Load, lbs. per sq. in., deflection ⅜ in.	Rating, per cent.
Major specimens:						
Untreated	7,028	100	3,563	100	1,007	100
Treated	4,058	58	2,595	73	687	68
Minor specimens:						
Untreated	6,150	100	4,137	100	1,117	100
Treated	4,842	79	2,781	67	745	67

The above ratings are based on taking the strength of untreated specimens as 100 per cent. It is seen by a comparison of the results that the treated minor specimens show considerable loss in strength, as compared with the untreated specimens.

The loss in strength in the modulus of rupture, transverse test, is twice as great in the major as in the minor specimens. This is due to the influence of the treated sap wood, which causes early failure in the treated major specimens. It is to be noted that the minor specimens were taken from the heart of the specimens, so that there was no influence of sap wood checks or treatments on the results of the test.

The loss in strength of the treated major and minor specimens in the comparison tests is practically the same.

GENERAL SUMMARY.

It is to be noted that in every case the treated material shows a decided loss in strength. The greatest loss is in transverse strength, due to the influence of the treated sap wood. It should also be noted that the loss in compressive strength is considerable.

As has before been noted, Oregon fir is very refractory to all kinds of treatment. Moreover, the subjection of the Oregon fir to high temperatures and pressures, extending over considerable length of time, causes permanent deterioration of the fiber.

Value of Physical Data.—The purpose for which the material is intended should be taken into account in the consideration of comparative results. The transverse strength of piling is not of as great importance as the compression strength. The loss in compressive strength is manifested in the failure of piling to withstand the sudden severe vertical stresses applied by the pile driver. It is also important that the piling have sufficient strength in shear parallel to the grain to prevent "shelling out" during the driving.

The minimum deterioration and loss of strength is obtained by the treatment of Oregon fir under as nearly normal conditions as possible. Excessive time and high temperatures of steaming should be avoided.

CONCLUSIONS.

The results of the tests indicate the following conclusions relative to the effect of the steaming process of creosoting Oregon fir piling:

(1) The depth of penetration of creosote is mainly dependent upon the depth of sap wood.

(2) The heart wood of Oregon fir piling is almost impervious to treatment.

(3) The depth of penetration of creosote is the same in the butts as in the tops.

(4) The depth of penetration of creosote should be interpreted as to mean the depth of "active" penetration.

(5) Tests of minor specimens show that injury to fiber through method of treatment is not localized to treated fiber alone, but extends throughout the whole specimen.

(6) The transverse strength of Oregon fir piling was decreased 42 per cent, due to the steaming process of creosoting.

(7) The compressive strength perpendicular to the grain was decreased 32 per cent, due to the steaming process.

(8) The compressive strength parallel to the grain was decreased 27 per cent, due to the steaming process.

(9) In general the average strength of Oregon fir piling subjected to the steaming process of creosoting was only two-thirds of its original strength.

ROADS AND STREETS

Methods and Cost of Resurfacing Asphalt Pavements in Brooklyn by the Surface Heater Method.

In resurfacing asphalt pavements the burner and surface heater method of removing the old asphalt and preparing the surface for new material has come into quite general use. In a paper before the Brooklyn Engineers' Club Joseph C. Huseman describes the methods in vogue in Brooklyn, New York, and his paper is given here in part.

The method of chopping out the old asphalt when applied to large areas, necessitating the repairing of many wear holes and depressions, is expensive and not satisfactory for these reasons: "It is claimed that the new, hot material does not weld or vulcanize onto the old, cold pavement, even though painted with hot liquid asphalt. There being no union between the old and the new material, cracks open up, admitting destructive elements, causing decomposition to set in and soon necessitating replacement." In the writer's mind, the simplest, most durable and practical method is not to disturb the old pavement in its present condition, which has settled under years of heavy traffic, but upon this foundation to lay an asphalt pavement that will not creep or shove, because its underlying surface adheres firmly to the old pavement, unifying the old foundation and the new asphalt top.

HAND BURNER WORK.

A method sometimes employed in repairing asphalt streets is by the use of hand burners. The idea of the hand burner is to blow a flame upon the pavement until the surface of the pavement has softened so it can be scraped off with a rake to such a depth as may be necessary to reach undisintegrated material. This old surface material is disposed of and new material is added. Figure 1 shows a hand burner in operation.

Machine.—This type of surface heater, commonly known as hand burner, consists of a machine which directs a blast of flame from a series of naphtha burners upon the asphalt surface. It is made with a hood 4 ft. wide, 5 ft. long, having five 2-in. burners. The frames are of 3-in. channel, supported on suitable axle and steel spoke wheels, 30 ins. in diameter. The fuel tank is of 20 gals. capacity, provided with suitable brass air pump, pressure gauge, necessary valves and metal funnel for filling. The extended handle is provided with counterweight to counterbalance the weight of the hood and burners. The machine weighs about 750 lbs.

SURFACE HEATER WORK.

The method of resurfacing employed in Brooklyn during 1912 consisted in drawing together a large volume of air, heated to the proper temperature, gently heating the old asphalt to about 300° F., thereby softening it to the same consistency as the new material;

the heating being done without flame, because of its destructive effect, in that the flame carbonizes the old asphalt surface.

Surface Heater.—The Lutz surface heater used for this work, Fig. 2, is equipped with a 50-HP. return tubular boiler, 38 ins. in diameter [illegible]

Fig. 1. View Showing Method of Operating a Hand Burner.

[illegible] filled steel.

The cut-off or reciprocating blades are of solid cast steel and are supported by six packing-strip blades, which are held in place by piston rings and are given their working pressure by six strong steel springs. The engine, having no dead center, can be started and stopped at any point. The main driving gear is of the compensating type, with bull-pinion and bevel following gear.

The fuel used in operating the Lutz surface heater is crude petroleum, which is stored in a tank over the boiler and which has a capacity of 210 gals. The oil is carried to the burner by gravity and forced into the combustion chamber by steam. The oil burner is of a self-heating type, having a chamber of sufficient size to hold enough oil that it may become heated to the proper temperature before being forced into the combustion chamber.

The heat is controlled by a cast lined No. 10 rolled sheet steel hood, with partitions sufficiently arranged to spread the heat evenly as it is blown from the heating chamber through the hood onto the pavement, and can be regulated to throw from 250° to 500° F. heat upon the pavement by a steam jet blower. The hood is raised and lowered at will by a steam cylinder, which has a lifting capacity of 1,000 lbs.

The machine is equipped with two safety valves, each set at 125 lbs. pressure; one Pem-

Fig. 2. Surface Heater in Operation.

berthy injector and one Gardner double cyl- [illegible] pumping capacity of 56 gals. of water per minute. The water tank is located under the heating chamber and has a storage capacity of 620 gals. The machine can travel from three to five miles an hour.

The surface of the old material, having been heated to about 300° F., is scraped off with the hoe and rake about ½ in. deep, which leaves a clean, corrugated surface on which the new material is immediately laid, care being taken to remove all inert, disintegrated, [illegible] material, to cut joints around all wear holes and to cushion out these holes and depressions with new material before laying the finished surface, so as to get as near a uniform compression as possible with the roller. While the surface is still hot, clean and corrugated sufficient new material (at a temperature of from 275° to 325° F.) is added to bring up the grade and contour of the street.

The temperature of both the old and new material being at the vulcanizing point, when tamped, smoothed and rolled a perfect weld is produced. The new asphalt being raked to give a depth of not less than ½ in. or more than 1 in. above the old or heated asphalt surface after final compression; as required by local conditions, depressions and wear holes were given a cushion layer sufficient to bring the surface about level with the prepared sub-surface.

Fig. 3. Typical Resurfacing. Surface Prepared by the Heater in the Foreground, New Material Added and Finished Surface in the Background.

No asphalt should be laid in wet weather, because the water is converted into steam, with the result that coherence of the asphalt mixture is prevented, and, although its surface may be smooth, the mass is really honeycombed, and when the pavement is subjected to traffic the voids open up.

SURFACE HEATERS IN OPERATION.

The paving composition is compressed by means of rollers and tamping irons, the latter being heated in a fire, and are used for tamping such portions as are inaccessible to the rollers, such as gutters, around manhole heads, etc.

Two rollers are employed. One, weighing from 3 to 5 tons and of a narrow tread, is used to give the first compression; the other, weighing about 10 tons and of a broad tread, is used for finishing. The amount of rolling varies with the depth of the material, but the rolling should continue until no impression is made upon the surface, being accomplished by first straight rolling and then cross rolling.

Before the surface heater can be used to any advantage the surface to be treated must be swept clean and dry with hand brooms, as the heater will not dry up satisfactorily any small pool of water on the pavement. The hood of the surface heater should remain down for a period of from 2 to 5 mins., the time varying according to the condition of the surface to be treated and atmospheric conditions, as on a damp day it is necessary for the hood to remain down for a greater period than on a clear day. The time required for properly heating the surface can be determined by trying the asphalt surface around the hood with a rake. The hood should in no case remain down long enough to set fire to the pavement, but it will happen where a patch has been put in recently the patch will sometimes start to burn before the remainder of the surface has been properly heated.

In accordance with the specifications for the Brooklyn work for 1912, the asphalt wearing surface and binder used in this work was to be measured in the trucks at site. This was done by taking nine soundings with an iron rod to determine the depth of asphalt in the truck, three soundings being taken in the front, three in the middle and three in the back, then averaging these and calculating the cubical contents of the truck. For convenience, a table can be prepared by calculation to give the cubic contents of the truck for different depths of asphalt.

In a long haul of about two hours from the plant the asphalt wearing surface in the truck will consolidate about 20 per cent of its original volume, as determined by measurement of the trucks at the site and comparison with the contractor's statement as to the number of boxes of asphalt in the truck, but this is not taken into consideration when measuring the trucks at the site.

The hood of the surface heater will cover about 5 sq. yds. at each heating; being a

Fig. 4. Typical Patching Work on Wabash Avenue, Chicago, Showing Area Prepared for the Addition of New Material.

little less than 10 ft. long, makes it convenient to work three surface heaters in a street having a 30-ft. roadway. In narrow streets of about 24-ft. roadway, where it is not convenient to run three machines across the roadway at a time, it may be accomplished by running one up on each side of the street, keeping about 6 ins. from the curb, and then following these with a third machine, about 15 ft. behind, and in the middle of the street, so as to cover the space left between the two side machines. An iron or asbestos guard is placed against the curb to prevent the action of the intense heat from cracking the curb, as the heat will crack all unsound and seamy stones, necessitating replacement.

One surface heater can conveniently heat 850 sq. yds. of surface in an 8-hr. working day, the machine covering 5 sq. yds. every 2 to 5 mins., and will consume about 210 gals. of fuel oil in this operation, requiring the engineer, one foreman and eight laborers. This includes the laying of the new material.

When the new material is not at the site ready to be laid the surface heater should be shut down, not only because it consumes unnecessary fuel oil, but because the surface must be reheated before the new material is laid, for if it is not warm and soft a perfect bond between the old and the new material will not be secured; also because the old material scraped off in reheating has to be built up with new material to bring up the grade and contour of the street. Figure 3 shows a typical piece of resurfacing, with a depth of new material varying from ½ in. to 1 in.

After the contractor's guarantee on an asphalt street has expired, and when local conditions require improvement over a large area, resurfacing by the heater method will prove most advantageous, when the time the street is closed to traffic has to be taken into consideration, as by the old method of taking up the asphalt and binder and disposing of this the street would be closed to traffic for several days, while in resurfacing the street is open to traffic in less than 24 hrs. It must also be observed that under the old method the old asphalt and binder are considered useless, and where resurfacing is possible it means the saving of a large amount of money by reason of the fact that only about ½ in. of the surface is removed.

PREPARING OLD BLOCK PAVEMENTS FOR ASPHALT SURFACING.

In discussing this paper, Mr. Moore of Kansas City stated that to remove a brick or stone pavement, to make way for a more modern and up-to-date pavement, necessitates taking up the old, wornout pavement and building a new concrete foundation on which to lay the new asphalt top, or whatever the material may be. Some cities have taken up the granite blocks and laid them down flat, on which they laid binder and asphalt top. This operation is not a success, because the foundation is destroyed. The binder does not adhere to the smooth surface of the granite, allowing the pavement to creep and shove. If a binder is not used, the sheet asphalt (without heating the stone) will not adhere to the granite. With the use of a surface heater this difficulty is overcome by heating the stone or brick and the asphalt top, which is not affected by heat or cold and cannot be separated.

The simplest, most durable and practical method is not to disturb the old pavement in its present condition, which has formed an excellent foundation, having settled under years of heavy traffic, but upon this foundation lay an asphalt pavement that will not creep or shove, because its underlying surface adheres firmly to the old pavement, unifying the old foundation and the new asphalt top. These results are obtained by thoroughly cleaning the pavement, after which it is heated with a machine for repairing asphalt and other hard pavements, whereby no flame is permitted to come in contact with the pavement to overheat or burst any part of it. While the brick or stone is in this heated state it is given a coat of hot liquid asphalt or other bituminous cement, and upon this is laid an asphalt top, which makes the perfect adhesion, rendering complete a smooth, noiseless and sanitary street.

Most all of the principal cities have condemned the use of a machine forcing a flame upon the pavement because of the destructive effect of intense heat. This opinion is well founded, because such appliances are not only destructive, but expensive, in that the flame carbonizes the old asphalt surface, causing the new asphalt top to creep and shove and develop holes soon after the repairs are completed.

COST.

In further discussion J. Strachon of Brooklyn stated that for resurfacing asphalt contract prices are about 85 to 90 cents per square yard. The price for asphalt wearing surface on one contract in Brooklyn, completed during September, 1912, was 64½ cents per cubic foot. The measurements of the material were made in the trucks upon their arrival on the street just previous to dumping. On this contract 49,647 cu. ft. of wearing surface were delivered and laid to a thickness, after compression, of about ½ in. The area of new pavement surface produced was carefully measured and totaled 38,580 sq. yds. On this contract, therefore, 1 sq. yd. of pavement required 1.287 cu. ft. of wearing surface and cost 83 cents.

On the contracts for entirely reconstructing streets and placing a 3-in. asphalt pavement, consisting of 1 in. of binder and 2 ins. of asphalt wearing surface, the average prices for three years are:

	Cost per sq. yd.
[illegible]	0.836
1911	0.915
1912	1.042

On these contracts the contractor sometimes maintains the pavement surface in good condition, without cost to the city, for the period of five years. On the resurfacing contracts there is no guarantee by the contractor. If the curb is in good condition and the foundation under an old pavement not too much out of grade, the burner method is quite satisfactory and has the great advantage of reducing the length of time during which the street is closed to traffic.

Cost of Boulevard Lighting and Street Sprinkling and Flushing in Grand Rapids, Michigan.

The last annual report of the Board of Public Works of Grand Rapids, Mich., gives the detailed cost of boulevard lighting and street sprinkling and flushing as shown in Tables I, II and III.

Total pounds of coal consumed	1[illegible],015,036
Average pounds of coal consumed per day	35,658
Average cost per 2,000 pounds in bins	$2.42
Total kilowatt hours for the year	2,100,500
Average kilowatt hours per day	5,755
Total number of lamp hours per year	3,982,756
Average number of lamp hours per day	10,912
Total number of hours operated per year	3,861.55
[illegible] number of hours operated per [illegible]	[illegible]
Average number of lamps in circuit	1,032
[illegible]	[illegible]
Average number of lamps operated per day	1,022
Average cost per lamp per year computed on above basis	$42.17
Average cost per lamp per year (station expenses only)	$24.72
Average cost per lamp per year (distribution only)	$17.45
Average cost per lamp hour (operation)	$0.00[illegible]
Average cost per lamp hour (distribution)	$0.00[illegible]
Average cost per kilowatt hour (station)	$0.01[illegible]
[illegible] per kilowatt hour (distribution)	$0.00848

TABLE I.—DETAIL STATEMENT OF OPERATION OF LIGHTING PLANT

	Total cost.	Average cost per day.	Average cost per lamp per day.	Average cost per k. w. hour.	Average cost per lamp hour.	Average cost per k. w. hour (station exp. only).
Coal	$15,753.74	$ 43.16	.04223	.0075	.003955	.0075
Wages	1[illegible]	[illegible]	.04190	.00744	.003924	.0025
Globes	1,196.64	3.28	.00321	.00057	.000301	
Carbons	1,864.52	5.11	.00500	.00089	.000468	
Repairs	2,925.27	8.01	.00784	.00139	.000734	.00139
Supplies	5,310.15	14.55	.01424	.00252	.001333	.00044
Oils	420.10	1.15	.00112	.00020	.000105	.0002
	$[illegible]	[illegible]	.11554	.02051	.010820	.01203

TABLE II.—DETAIL COST OF BOULEVARD LIGHTING.

Maintenance Cost Per Lamp Is Shown [illegible]

	Total No. repaired.	Cost of repairs.	Average cost.	Number in service.	Average cost.
A. B. iron case	383	$521.02	$1.36	442	$1.17
A. B. copper case	[illegible]	178.90	1.49	380	.47
Fort Wayne D. C.	[illegible]	52.38	.56	109	.48
Fort Wayne A. C.	1	1.38		55	.02
Fort Wayne magnetite				7	
G. E. magnetite	[illegible]	66.40	.54	100	.66
Westinghouse magnetite		1.50	.30	6	.25
Totals	[illegible]	$821.58	....	1,099	.....
Averages	....		$1.13	....	$0.74

TABLE III.—COST OF STREET SPRINKLING AND FLUSHING.

Street Railway.

The sprinkling season started on April [illegible], 1913, and was carried on until October 7, 1913.

Number of miles in route, 28.56.

Number of miles sprinkled, 10,669.85 at 12c per mile	$ 1,280.37
Gallons of water used, 23,415,000 at 4c per M gals.	936.60
Total cost of sprinkling	$ 2,216.97
Cost of sprinkling per mile (St. Ry.)	$0.12
Cost of water per mile	.088
Total cost of sprinkling per mile	$0.208
Average cost per foot (lineal) sprinkled	$0.0147
Gallons of water used per mile	2,194
Average cost per mile of route sprinkled	$77.62

Highway Sprinkling.

The sprinkling season started on April 1, 1913, and was carried on until October 7, 1913.

Number of miles of streets included in route, 206.970.

Number of sprinkling carts in service, 31 (750 gallons).

Number of miles sprinkled, 35,193.105 at	$15,849.23
Gallons of water used, 90,723,250 at	4,355.66
Total cost of sprinkling	$20,204.89
Cost of sprinkling per mile (labor only)	$0.418
Cost of sprinkling per mile (supplies and repairs)	.032
Cost of sprinkling per mile (water used)	.124
Total cost of sprinkling per mile	$0.574
Average cost per mile of route sprinkled	$97.62
Average cost per foot (lineal 30-ft. roadway)	$0.0185
Gallons of water used per mile	2,592

Gasoline Power Flusher.

Number of miles of streets flushed, 316, equal to 7,759,798 sq. yds.

Number of gallons of water used, 840,000 gallons.

Gallons of water used per 1,000 sq. yds., 108 gallons.

Gallons of water used per mile, 2,658 gallons.

Total cost of:

		Per mile.	Per 1,000 sq. yds.
Water	$ 40.31	$0.12	$0.0052
Labor	318.75	1.01	.410
Repairs	216.62	.69	.0279
Oils and gas	51.20	.16	.0066
	$626.88	$1.98	$0.0807

Air Pressure Flusher.

Number of miles of street flushed, 235, equal to 5,568,794 sq. yds.

Number of gallons of water used, 948,750 gallons.

Gallons of water used per 1,000 sq. yds., 170 gallons.

Gallons of water used per mile, 4,037 gallons.

Total cost of:

		Per mile.	Per 1,000 sq. yds.
Water	$ 45.55	$0.19	$0.0081
Labor	244.39	1.04	.0440
Repairs	6.06	.03	.0010
	$296.00	$1.26	$0.0531

Comparative Statement.

	1909-10	1910-11	1911-12	1912-13	1913-14
No. of lamps operated	877	879	877	933	1,022
Cost per lamp per year	$36.26	[illegible]	[illegible]	$36.11	$42.17

The total cost per lamp per annum for maintenance and operation, fixed charges, etc., is as follows:

Maintenance and operation	$42.17
Loss through non-taxation	2.80
Depreciation	5.35
Interest on bonded debt	4.90
Interest on investment over bonded debt	7.01
Total cost per lamp per annum	$62.23

	Post nights.
197—5 light posts (340 watt per post) operated from April 1 to October 1, 1913, equals	36,051
230—5 light posts (340 watts per post) operated from October 1, 1913, to April 1, 1914, equals	41,860
Total post nights per year	77,911

	Total cost of	Cost per post per night.	Cost per post per year.
Wages	$ 952.51	$0.0122	$ 4.45
Current	4,579.79	0.0587	21.42
Globes	96.68	0.0013	.47
Lamps	663.61	0.0085	3.10
Repairs, etc.	459.85	0.0059	2.16
	$6,752.44	$0.0866	$31.60

Cost of Photographic and Blue Print Work for the Engineering Department of Salt Lake, Utah.

A recent report of the city engineer of Salt Lake, Utah, states that the photographing and blue printing divisions, although not maintained under separate heads, are big factors in the work of the engineering department.

For almost every contract for public improvement there are made from ten to twelve blue prints of the plans and details, approximately six of the grade sheets and two of the assessment plats. Prints of the city maps, subdivisions, profiles and surveys are made for private parties for 7½ cts. per square foot on paper and 10 cts. on cloth. During the year approximately 26,500 sq. ft. of blue print paper and cloth was used. The cost per square foot for making cloth prints was 5½ cts.; for paper, 2½ cts.

The principal work of the photographic division consists of making negatives and prints of public improvements before, after and during the course of construction; also each page of the sewer plat books are photographed and prints made for the field party staking the house connection. Besides this, a large amount of work is done for the park department, the health department, the law department and the daily papers. During the year there were made 179 8x10 and 147 6½x8½ negatives, and 630 8x10 and 147 6½x8½ prints. The average cost per print was 74 cts.

Cost of Concrete Culverts and Bridges in Milwaukee County, Wisconsin, in 1913.

(Staff Article.)

Culverts and small bridges constructed in Milwaukee County, Wis., in connection with the concrete road described in the issue of ENGINEERING AND CONTRACTING for May 13 were built according to standard designs prepared by the Wisconsin Highway Commission, some of which designs being given in the issue of Oct. 28.

Fig. 1. Type of Concrete Girder Bridge.

TABLE I.—COST OF CONCRETE BRIDGES IN MILWAUKEE COUNTY IN 1913.

Item.	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)[4]	(9)	(10)	Totals.	Per cent of total.
Span, ft.	18	20	10	20	12	12	20	20	20	12	..	....
Length, o. to o., headwalls, ft	36	26	24	27	22	22	22	24	22	30	..	....
Concrete, cu. yds.	21	26	55	36	16	90	98	31	105	67	555	....
Steel	$ [illegible]	$ 73.02	$ 26.41	$ 125.90	$ 59.20	$ [illegible]	$ 41.98	$ 76.82	$ 140.87	$ 69.47	$ 737.11	9.2
Cement	83.17	73.14	89.36	68.29	29.41	162.04	160.13	22.79	160.27	115.40	964.00	12.2
Sand and gravel	26.00	25.00	42.50	70.00	24.75	150.99	164.81	7.50	102.95	14.95	629.45	8.0
Lumber	27.20	22.80	48.20	50.20	22.28	99.02	107.97	97.19	264.28	60.00	799.14	10.2
Hardware	1.79	4.00	7.88	5.34	2.37	12.98	12.38	4.72	13.58	3.50	68.54	0.9
Labor	251.30	143.90	480.63	211.60	124.70	615.25	742.28	728.07	567.19	459.58	4,324.55	54.7
[1]Workmen's compensation	11.58	8.40	17.08	13.06	6.16	26.60	30.22	23.04	30.71	17.77	184.92	2.3
[2]Machinery depreciation	12.25	8.88	18.07	13.81	6.83	28.13	31.96	24.36	32.47	18.79	195.55	2.5
Total cost	[illegible]	[illegible]	$730.18	[illegible]	$276.00	$1,136.81	$1,291.73	$984.49	$1,312.32	$759.46	$7,903.26	100
Cost per cu. yd., concrete	15.90	13.80	13.30	15.50	17.25	12.62	13.20	....	12.50	11.30	14.25	....

[1]State law requires compensation for lost time, injury, etc. Prorated from total of all classes for work of the year. [2]Depreciation prorated from total of all classes of work during the year. [3]Old rails. [4]Slab only—no abutments. Other work included.

TABLE II.—COST OF CONCRETE CULVERTS IN MILWAUKEE COUNTY IN 1913.

Item.	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	Totals.	Per cent of total.
Span, ft.	[illegible]	3-small	1-6 ft.* 1-4 ft.* 1-7 ft.	6-ft.* [illegible]	1-5 [illegible]-small	5-small	1-6 (ext.) [illegible]-small	1-3½	1-small	4-small		
Length, o. to o., headwalls, ft.	30	[illegible]	7 ft.-27	1-30 [illegible] 1-27	29	30	20	30	62	25		....
Concrete, cu. yds.	[illegible]	19	70	193	95	134	119	54	17	42	843	..
Steel	$ 22.79 [illegible]	$ 11.92	$ 8.29 4.49 33.15	[illegible]	[illegible]	$ 62.77	16.48	$ 22.79	$ 4.97	$ 19.89	$ 380.05	3.7
Cement	141.17	76.87	92.27	336.48	143.08	261.57	113.98	75.56	22.18	84.29	1,410.75	13.9
Sand and gravel	107.00	54.60	[illegible]	[illegible]	103.88	36.57	27.20	66.20	5.00	78.00	684.22	6.7
Lumber	2.00	51.00	123.14	249.27	82.66	116.55	22.74	34.80	....	....	685.46	6.8
Hardware	2.85	13.38	0.91	[illegible]	17.07	1.00	12.53	1.36	....	....	83.82	0.8
Labor	523.12	439.88	518.30	1,509.86	289.23	1,237.13	998.11	421.63	183.83	318.55	6,439.65	63.3
[1]Workingmen's compensation.	20.21	16.07	21.36	57.84	16.89	42.24	30.82	15.30	5.31	12.31	238.33	2.3
[2]Machinery depreciation	21.40	16.99	22.52	61.16	17.86	44.67	32.60	16.18	5.61	13.01	252.00	2.5
Total cost	[illegible]	[illegible]	[illegible]	[illegible]	$721.79	$1,805.50	$1,317.74	$653.82	$226.90	$526.05	$10,185.47	100
Cost per cu. yd., concrete	12.25	14.00	13.00	12.80	7.60	13.50	11.10	12.00	13.35	12.50	13.50	....

[1]State law requires compensation for lost time, injury, etc. Prorated from total of all classes of work for the year. [2]Depreciation prorated from total of all classes of work during the year. *Extension.

The structures are designed to safely bear an 18-ton road roller. Cement was purchased in large quantities and was of a tested uniform quality. A sandy gravel was used for aggregate. All the costs of concrete, culverts and bridges are given in the accompanying Tables I and II. The percentage the average cost of each of the main items bears to the total cost of the structure is noted in both tables. The work was accomplished entirely with labor hired by the county. Foremen were paid $125 a month. Laborers received $2.50 a day and teams $6.

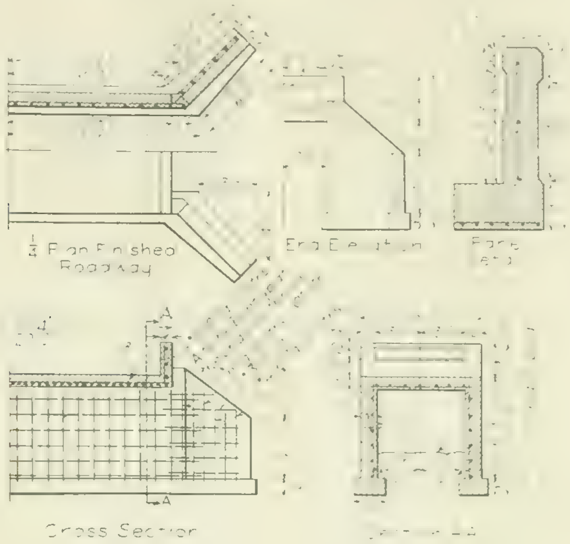


Fig. 2. Type of Concrete Culvert.

Figure 1 illustrates the type of bridge constructed and Fig. 2 shows the type of culvert used. The work was supervised by A. J. Kuelling, county highway commissioner.

Cost of Supervision and Work Accomplished by the Ohio Highway Department.

A recent publication of the Ohio Highway Department states that the cost of engineering supervision of road and bridge construction work by the department has been as follows:

Year.	Percentage of total cost.
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
1909 and 1910	5 39
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]

The total mileage of roads in Ohio (exclusive of municipalities) is approximately 90 000. The state road system includes 9,400 miles in the inter-county system and 2,400 miles in the main market road system. Of the above system approximately 900 miles will have been placed under contract during the year ending January 1, 1915. During the current year the state will have expended $3,150,000 and the counties $3,450,000, a total of $6,600,000 in the construction of their system.

WATER WORKS

Michigan Supreme Court Decision in Case of Kalamazoo vs. Standard Paper Co. to Recover Value of Water Taken from Fire Lines for Industrial Use.

Herewith is presented the text of the decision of the Supreme Court of Michigan, recently handed down, in the case of the City of Kalamazoo, plaintiff and appellee, vs. Standard Paper Co., defendant and appellant. This case, it will be recalled, is an action brought by the city of Kalamazoo against the Standard Paper Co. to recover the value of water which the plaintiff alleges was converted by the defendant during the period from Nov. 17, 1906, to Nov. 17, 1910. The case is of great interest, on account of the precedent which it appears to establish, to all water works interests. We here quote the decision in full, since the line of defense is of great interest, as well as the court's reasoning and findings:

The defendant company is a manufacturer of paper, and in the process of manufacture uses a large amount of water. It had constructed an aqueduct by means of which it could bring water from the Kalamazoo River to reservoirs near its mill. In 1906 an automatic sprinkling device was installed by the defendant, ostensibly with two sources of supply, one being a pressure pump pumping directly from the defendant's reservoir, the other being the city water brought to the sprinkler system by a 6-in. line connected with the city mains. This is a customary arrangement to insure a supply from the factory's own source and a supply from the outside in case of injury to the private supply during a fire. The city water was furnished by direct pressure from the city water works, and the defendant's pressure pump and the city water works were connected on the same line. This arrangement is shown by the defendant's exhibit 4 here set forth. (A reproduction of said exhibit was hereto attached.)

At this time there was no meter on the city's line to this sprinkler system, nor on the lines to sprinklers in other factories. There was a clapper valve in the line designed to keep the water pumped by the pressure pump from entering the city mains when the private pressure exceeded the city pressure, and only to allow the city water to pass into the sprinkler system when the private pressure fell below the city pressure.

During the aforementioned period there were three water service lines to the defendant's property. One was a ¾-in. metered pipe furnishing water to the office. Another was the line to the sprinklers, just described. The third was a 4-in. main, with a 2½-in. metered connection running to one of the two overhead tanks which supplied the filtered water for the manufacturing process. The city installed a sealed 2-in. by-pass around the meter, to be used only when the meter should be taken out for repairs.

It is charged by the plaintiff, and support thereof is found in the evidence, that the defendant opened the 2-in. by-pass around the meter, tapped the sprinkler system in numerous places, made connections on the ¾-in. pipe ahead of the meter, and unlawfully took and used city water from these three sources without its having been registered by meters and without paying for it, in violation of Ordinance No. 197 of the City of Kalamazoo, which provided rules and regulations under which water was to be used. It is charged, and there was evidence to support it, that there were 27 taps put on the sprinkler system and water taken through them under orders from the defendant's superintendent and with the approval of the defendant's manager.

In August, 1910, computations having been made which showed that less than 50 per cent of the water delivered into the mains by the city water works was being paid for, Ordinance No. 305 was passed by the city council, requiring meters on all water lines save as therein provided. Accordingly, on Nov. 17, 1910, a Hersey Detector Meter was installed on the defendant's sprinkler line, and a 28-hour test revealed that the defendant was receiving a much larger amount of water than it was paying for, and that the conditions during this 28-hour period were no different than they had been before that time. Immediately thereafter the fire pump was dismantled and repaired and the taps on the sprinkler system disconnected. An investigation was made by city officials, which revealed that the pressure pump gage showed a pressure equal to the city pressure. But when the city main was suddenly cut off the gage on the defendant's pressure pump dropped from 68 lbs. to 5 lbs. A further investigation by a council committee followed, in which it was found that the defendant was taking city water in the three ways hereinbefore described. It is the plaintiff's claim, and evidence was presented tending to show, that the defendant's pressure pump was a mere blind, and that it never did maintain a pressure equal to or greater than the city pressure.

On the other hand, it was claimed by the defendant that the taps had been put on the sprinkler system by its mechanics for their own convenience, and without the knowledge of the defendant; that its pump maintained a pressure greater than the city pressure up to a short time prior to the suit; and that up to said time no city water was drawn from the taps. It was also claimed by the defendant that the water was taken from the 2-in. bypass on the 2½-in. line openly, with the constructive knowledge of the plaintiff's inspector; also that at the time the Hersey meter was read the pressure pump was down for repairs and there was a large leak in the basement, causing a large temporary waste of water; that the flow during the 28 hours during which the test was being made was unusual and therefore not a fair basis on which to estimate the amount used in four years.

The evidence in support of these claims being submitted to a jury, a verdict was found for the plaintiff in the sum of $15 840.76, and judgment was entered thereon by the court.

Defendant's motion for a new trial was overruled. The cause being removed here by writ of error, the assignments of error are grouped by counsel for convenience as follows:

1st. Errors in the ruling of the court during the trial and errors in ruling on misjoinder of assumpsit and tort counts.

2nd. The error committed by the court in permitting a statement or account, not an exhibit in the case, prepared by plaintiff's counsel, of plaintiff's claim, to be taken to the jury room.

3d. Improper argument of counsel.

4th. Errors in the court's charge and refusals to charge.

5th. Excessive verdict.

The declaration of the plaintiff contains six counts. It is appellant's claim that the first four counts are all in assumpsit and the last two are in tort, and that these two actions cannot be joined and the plaintiff must elect. The plaintiff does not dispute the latter contention, but says that the first three counts are purely counts in trespass on the case and the last three are in trever. The claim that the counts are in assumpsit is urged because of the fact that Ordinance No. 197 of the City of Kalamazoo with relation to the water works is set forth in these counts, and that in effect a contract relation between the parties is alleged. The plaintiff's claim is that the defendant, without any regard for the ordinance, took water wrongfully and unlawfully. It appears from a reading of these counts that the ordinance is set forth for the purpose of showing that the conduct of the defendant in this case was unlawful, unauthorized, wrongful, and in violation of the law of the City of Kalamazoo, which makes these counts sound in tort instead of in contract. In the case of Dillon v. Craig, 168 Mich. 216, relied upon by the defendant, the declaration showed a contract obligation, and there was no statute or ordinance which made the violation of the contract tortious. There is no merit to appellant's contention as to a misjoinder of counts.

During the final argument of plaintiff's counsel, in the discussion of the question of damages, two typewritten computations which had been prepared were handed to the jury, showing how the plaintiff estimated its damages. The statements were as follows:

FIRST STATEMENT.

Hersey Meter.

27,144 cu. ft. in 24 hours.
27,144x365 days equals 9,907,560 cu. ft. per year.
9,907,560x4 years equals 39,630,230 cu. ft.
Water rates are
$1.00 for each 1,000 cu. ft. up to 3,000.
.80 for each 1,000 cu. ft. from 3,000 to 10,000.
.60 for each 1,000 cu. ft. over 10,000.

3,000 cu. ft. at $1 equals..................$	3.00
7,000 cu. ft. at 80 cts. equals............	5.60
39,620,230 cu. ft. at 60 cts. equals......	23,772.14
	$23,780.74
Take out $444.93 paid by defendant	444.93
	$23,335.81
Add interest at 5 per cent from Nov. 17, 1910, 1 year, 2 months and 25 days, equals	1,442 25
	$24,778.06

SECOND STATEMENT.

May 17, 1909, got 3d machine working
May 17, 1909, to Nov. 17, 1910, equals 18 months.
That leaves out of the 4 years, 30 months defendant had two machines.
With 3 machines defendant used 1,500,000 gals.
With 2 machines defendant used 1,000,000 gals.
27,144 cu. ft. for 24 hours equals 9,907,560 cu. ft. per year, or 14,861,340 cu. ft. for 18 months, using in the mill 1,500,000 gals. per day.
2 machines used 1,000,000 cu. ft. in 24 hours, or ⅔ of 9,907,560 equals 6,605,040 per year.
30 months equals 2½ years.
6,605,040 x 2½ equals 16,512,600
Add together what 3 machines used in 18 months and what 2 machines used in 30 months
14,861,340 plus 16,512,600 equals 31,373,490 cu. ft.

3,000 cu. ft. at $1 equals.................$	3.00
7,000 cu. ft. at 80 cts. equals	5.60
3,136,940 cu. ft. at 60 cts. equals........	18,818.36
	$18,8[illegible]
Take out $444.93 paid by defendant.....	444.93
	$18,382.02
Interest at 5 per cent from Nov. 17, 1910—1 year, 2 months, 25 days....	1,136.93
	$19,518.06

These statements were read in open court in the presence of counsel. An examination of these statements shows that they are nothing more or less than computations in arithmetic based on facts claimed to have been proved by the plaintiff in the case. When the jury was about to retire plaintiff's counsel called the court's attention to the fact that some of the statements were still in the hands of the jury, and said with reference thereto:

It is simply a computation or claim of the plaintiff as to the amount of damages along the two lines mapped out on that paper. We claim it is for the consideration of the jury the same as any other fact in the case, not binding, but that they may consider it in arriving at their verdict. We have no objection to the jury taking that to their jury room and if the other side have some figures they wish to give to the jury we have no objection.

It is claimed that these statements were never given to defendant's counsel for examination. The record does not show that copies were requested, and counsel must have heard the contents thereof as they were read to the stenographer. Counsel was unquestionably entitled to copies, and undoubtedly such copies would have been furnished if a request had been made.

It is urged that allowing the jury to take the memoranda to the jury room was error, under the decision of this court in Harroun v. C. & W. M. Ry. Co., 68 Mich. 208. It seems to be clear that according to the rule announced in that case the practice followed in the instant case is not to be considered, and that it was improper for the court to allow these memoranda to be taken into the jury room without the consent of the opposing counsel. We are of the opinion, however, that the defendant was not prejudiced thereby, as the verdict of the jury was in a sum considerably less than the amounts claimed in the computations.

In his charge the court gave the following:

5. I instruct you that the plaintiff has called the following witnesses: Alexander Clark, William Wilhelm, Fred Phetteplace, Jason Spade, Jeremiah Linehan, Frank Handy, William Hewitt, Walter Jones, Perry Norman, Warren Johnson, William Gaylor, Ernest Shepard, and George Harvey, under Act 307 of the Public Acts of Michigan for the year 1909, which provides as follows: An act to authorize parties litigant when they call as witnesses in their behalf the oppsite party, employe, or agent of said party, to cross examine such witnesses, and providing that they shall not be bound by their answers.

The people of the State of Michigan enact:

Section 1. Hereafter in any suit or proceeding in any court of law or equity in this state, either party, if he shall call as a witness in his behalf the opposite party, employe or agent of said opposite party, or any person who, at the time of the happening of the transaction out of which such suit or proceeding grew, was an employe or agent of the opposite party, shall have the right to crossexamine such witness the same as if he were called by the opposite party; and that answers of such witness shall not interfere with the right of such party to introduce evidence upon any issue involved in such suit of proceeding, and that the party so calling and examining such witness, shall not be bound to accept such answers as true.

That is the end of section 1. And in so calling said witnesses, or any of them, the plaintiff is not bound by the answers given by said witness or witnesses, nor is said plaintiff bound to accept the answer or answers of any one or ones of said witnesses as being true answers and the plaintiff may accept such part or parts of the answers of anyone or ones of said witnesses as the plaintiff elects so to accept and so far as the plaintiff is concerned such of said answers, if any, which the plaintiff does accept as being true, are for the consideration of the jury the same as any other facts or circumstances in the case.

With reference thereto counsel for appellant says in brief:

While the plaintiff is not bound to accept the answers as true, if the answers stand uncontroverted, they stand as facts in the case, and just exactly with the same force and effect as if they had been sworn to by any other witness in the case, admitting of course the jury's right to consider the credibility of such witnesses, but the Judge said that the plaintiff need not accept anything which was adverse to it, and that they were only to consider such things as were favorable to the plaintiff. In other words, only consider such things as the plaintiff did accept.

It is also claimed that by this instruction the trial court swept aside and eliminated from the case some of the most valuable testimony of the defendant as to the capacity of the pressure pump, as several witnesses called under this statute had given testimony with reference thereto.

It is true that this instruction of the court, taken by itself, in explaining to the jury the effect of the statute, was somewhat incomplete and confusing. However, the court did not say that any testimony that was favorable to the defendant and which plaintiff did not accept was not for the consideration of the jury, and in this connection the following instruction of the court given to the jury should be considered:

And I further instruct you that any statements or testimony submitted by the defendant or any employe or former employes of the defendant called by the plaintiff as to the condition of this pump during the entire four years mentioned in the plaintiff's declaration is not necessarily to be taken as true, but should be considered by you, and you are to determine the condition of this pump during said period and the question of whether or not it did actually maintain the pressure claimed by the defendant by all of the facts and circumstances and testimony left in this case for your consideration.

In our opinion, this instruction taken in connection with the other made it clear to the jury that they must consider all the testimony in the case, and there was not eliminated from their consideration such testimony favorable to the defendant, given by witnesses called under the statute, which had not been accepted by the plaintiff. See Jones v. P. M. R. R. Co, 168 Mich. 14; Johnson v. Union Carbide Co., 169 Mich. 659.

The court charged,

If you believe that these taps or connections did exist ahead of the meter and that the water passing through these pipes was not registered by the meter then I instruct you, gentlemen of the jury, that any water taken through such pipes by the defendant company, was taken wrongfully and unlawfully and your verdict should be for the plaintiff, the City of Kalamazoo, in the amount of the value of the water so wrongfully taken to be determined by you as I shall hereafter instruct you, even if the defendant's officers, directors, and stockholders, any or all of them, did not know water was being so used by the defendant;

and it is contended that no verdict should be rendered against the company unless the fact of the wrongful and unlawful taking of the water is brought home to some officer of the corporation. In this case, there being evidence to warrant the jury in finding that the defendant established the sham pressure pump and the improper openings on the sprinkler system, it is not unreasonable to assume that the wrongful means were installed to take the plaintiff's water without metering and paying for it. By so doing, the defendant invited its employes to use the unlawful means for the unlawful purpose. As is said by counsel in his brief:

This is a case of the employe using the means furnished by the employer to accomplish an unlawful purpose in furthering the master's interests and out of which nobody but the master received the benefit.

Cooley, in his work on Torts (3d ed.), p. 1017, says:

The master is reliable for the acts of his servants not only when they are directed by him but also when the scope of his employment or trust is such that he has been left at liberty to do, while pursuing or attempting to discharge it, the injurious act complained of. It is not merely for the wrongful acts he was directed to do but the wrongful acts he was suffered to do that the master must respond.

See also 21 Cyc, pages 1582-1585.

There was an error in the charge as given.

If entitled to damages—and a reading of this record is convincing that the jury were warranted in finding that unmetered water was taken by the defendant's servants and employes for the use and benefit of the defendant, for which the defendant did not pay—the damages should be the full value of the property at the time of conversion, together with the interest on that value from that time. It is clearly impossible for the plaintiff to prove with absolute certainty the amount of water the defendant converted, and this want of certainty is properly attributable to the fault of the defendant. The claims made by the plaintiff were for the highest amount of water which could possibly have been used by the defendant by means of the unlawful taps and by-passes. The defendant sought to reduce this by attempting to show that such an amount of water could not have been and was not taken from the city's mains as alleged. Taking the water in the way it did, the defendant must bear the risk of the uncertainty thus produced. Allison v. Chandler, 11 Mich. 542; Gilbert v. Kennedy, 22 Mich. 117; Krumenaker v. Dougherty, 74 App. Div. (N. Y.) 452. We are satisfied that a large amount of unmetered water was taken by the defendant, and while the amount taken must be necessarily an approximation based on the test made by the Hersey meter and all the other facts and circumstances shown in the case, we are not convinced that the amount found by the jury is excessive.

During the closing argument of plaintiff's counsel an exception was taken by defendant's counsel, as follows:

Mr. Irish: I will take an exception to counsel talking about stealing water.

The Court: I think the argument may proceed, that has been indulged in by the other side, as a matter of law. I will charge the jury about its weight.

Mr. Irish: An exception.

The trial court, in charging the jury with references to this, said:

The jury are instructed that under the declaration as it now stands they are not to consider the question as to whether the Standard Paper Co. or any of its agents have been guilty of stealing water. There can be no stealing or larceny of water unless the elements of larceny or stealing are charged in the declaration and proven in this case. No larceny is charged or proven in this case. I will not say proven.

An examination of the excerpts from the arguments of defendant's attorneys shows that the statement of the trial judge is justified, and it appears that language with reference to the stealing of water had been used by the attorneys for the defense in their argument. Although the question of the stealing of the water was not in the case, having been made prominent and commented upon by defendant's attorneys in their arguments, there was no prejudicial error in plaintiff's attorney using the same language in his reply. People v. Swift, 172 Mich. 473. We are not satisfied that the argument contributed to the result. Houser v. Carmody, 173 Mich. 121.

Criticism is also made that the trial judge unduly emphasized plaintiff's claim in his charge, but a careful reading of the entire charge is satisfying that the questions in issue were fairly submitted to the jury.

Finding no prejudicial error in the record, and being convinced that the verdict finds support in the evidence in the case, the judgment is affirmed.

Design and Cost of Two Types of Concrete and Metal Fence.

The present article describes the design and gives the cost of two types of combined concrete and steel fence. Both fences were designed by and built under the direction of the corps of engineers of the U. S. Army. They are described in Professional Memoirs for March-April, 1914, by Mr. Frank D. Holbrook. The information given here is from that article.

Dam No. 15.—The first fence was made by the day labor force at Dam No. 15, Ohio River. This fence is about one mile long, and it was built to enclose the Government property at Dams Nos. 14, 15 and 18. All posts are 1:2:4 concrete, mortar faced, beveled corners and reinforced with ¼-in. steel rods. Ordinary posts are 5 by 8 ins. at the base, tapering to 5 ins. square at the top, with one rod in each corner. They are 7 ft. long and set 2 ft. 6 ins. in the ground. Angle and gate posts are 10 ins. square for their whole length, with a 2 in. projection 2½ ins. below the top and reinforced with eight rods, four at the corners and four in the faces. They are 7 ft. 6 ins. long, 2 ft. 6 ins. in the ground and project 6 ins. above ordinary posts. Two lugs, 2 ins. by ⅜ in., running through the posts, carry the panels. These lugs extend a short distance on two sides of the post, and have the 2 in. dimension horizontal.

The panels and gates are made up of 3-in. 5-lb. channels with ⅜-in. round pickets. The pickets are caulked tight in the channels on the under side. The lugs are bent after the posts are seasoned to suit the slope of the fence. Panels secured to the posts by ⅝-in. bolts extending vertically through channel and lug.

At the site of most of the Ohio River dams the property is overflowed in extreme floods. The panels are then simply removed and laid flat on the ground so as not to obstruct the passage of drift and debris with which the river is filled at such times. After the floods have passed the panels are brushed off and replaced.

Through some of the lowest ground where the current was very strong during floods the posts were made 7 ft. 6 ins. long and set 3 ft. in the ground, the hole for the post being filled with poor concrete to within about 10 ins. of the surface.

If the posts are brushed over with the following mixture much will be added to the appearance of the fence: 20 parts cement, 20 parts very fine sand, 1 part yellow coloring matter. Add sufficient water to make a thin grout, keep well stirred and apply with a small whitewash brush. At Dam No. 15 this cost 6 cts. per post. The following cost data have been taken from the record at Dam No. 15:

Angle and gate posts, each	$3.35
Ordinary posts, each	1.23
Fabricating panels, each	2.78
Painting panels (three coats), each	.74
Setting fence (295 panels at Dam No. 15), each	.91

Above prices include cost of forms for posts and all materials and labor.

Panels are 9 ft. 11 ins. center to center of posts, large gate has 10 ft. clear opening and small gates 4 ft.

Cost of about 3,000 lin. ft. of fence erected at Dam No. 15, $0.57 per lin. ft. in place.

The cost of setting up the fence will vary with the nature of the soil. At Dam No. 15, for about one-half the fence erected, the soil was a heavy clay with much shale embedded. This made the digging of post holes rather expensive.

Washington Aqueduct.—The accompanying cost data, Table I, and the two illustrations given pertain to the concrete and steel fence at the Georgetown Reservoir of the Washington Aqueduct. This fence was designed by Captain Hannum, Corps of Engineers, U. S. Army. A line drawing of a typical post and panel of this fence is shown in Fig. 1, and Fig. 2 is a view of the same portion of the completed fence. The gates have a clear opening of 9 ft. and consist of ¾-in. square pickets caulked into top and bottom horizontal channel irons 2x1x 3-16 ins. in section. The cost data given in Table I pertain to this fence at the Georgetown Reservoir.

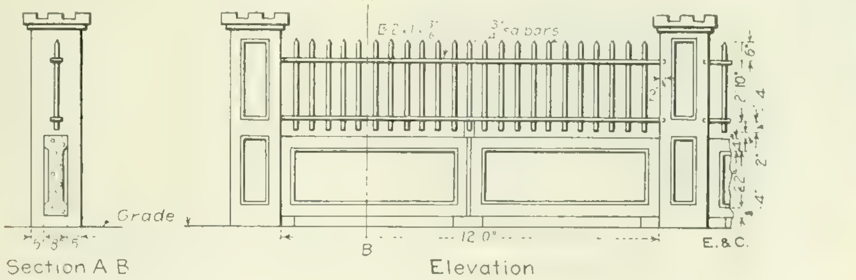


Fig. 1. Detail of Typical Post and Panel of Concrete and Metal Fence Around Georgetown Reservoir, Washington, D. C.

TABLE I. COST OF CONCRETE AND STEEL FENCE AT GEORGETOWN RESERVOIR.

Steel fence and gates	$1,092.32
Paint	105.00
Cement	784.44
Sand	22.54
Gravel	51.75
Stone dust	133.00
Granolithic	423.00
Lumber for forms	74.82
Brass letters	28.70
Wire brushes	4.76
Labor, surveying	72.53
Excavating and placing foundation	220.67
Building forms	73.18
Mixing and placing concrete	1,442.18
Removing forms and scrubbing concrete	341.17
Hauling materials	44.80
Erecting steel fence and painting	121.88
Grading	213.24
Superintendence and office work	139.06
Main office charges	145.42
Total	$5,498.26
Number of linear feet of fence	2,158
Cost, per linear foot	$2.55

The Aeration Basin of the New Water Purification Plant at Miraflores, Canal Zone.

A water purification plant is now under construction by the United States Government near the Miraflores locks of the Panama Canal. This plant will purify water for the six towns along the canal south of the Culebra Cut. The raw water is taken from an arm of the Miraflores Lake, which will always be more or less polluted. The water is to be subjected to aeration, sedimentation, coagulation, filtration and sterilization. The several units of the plant are: Aeration basin, head house and mixing chambers, sedimentation basin, filter building, injection chamber, effluent controllers, alum and hypochlorite mixing apparatus and hypochlorite dosing apparatus. The entire plant was described by Mr. George M. Wells, Division Engineer, in a paper before the annual meeting of the New England Water Works Association. The following description of the aeration basin is taken from Mr. Wells' paper:

All ground waters on the Isthmus of Panama, at least all within the Canal Zone, when allowed to stand in reservoirs or pipes, at times give off objectionable odors when discharged. These odors result from the hydrogen sulphide gas given off by the large quantities of decomposing vegetable matter present in the raw water. Aside from being acutely disagreeable to the sense of smell, this gas attacks all paints in the immediate vicinity having lead or zinc bases, and also discolors metals. The odor given off by the Gatun Lake water as it flows over the spillway at times reaches such volume and strength as to be plainly noticeable down the wind for a distance of nearly two miles. It is obvious that in the treatment of water of such char-

Fig. 2. View of Typical Panel of Georgetown Reservoir Fence.

acter more or less elaborate aeration seems warranted.

The iron content in the water of the Miraflores lake generally runs from 0.8 to 2.5 parts per million. This iron is in solution, and experiments conducted, using compressed air at low pressure and considerable volume, indicated that aeration might be expected to prove of valuable assistance as a preliminary treatment to the application of sulphate of aluminum. The changing of the iron from the ferrous to the ferric state during aeration and its partial elimination by precipitation in the aeration basin seemed to result—at least in the case of the water passing the Gatun purification plant at Agua Clara—in an appreciable reduction in the amount of aluminum sulphate required to remove the color. This becomes a vital point in waters of low alkalinity and high color, and Mr. Wells in designing the new plant for Colon and Cristobal attempted a more or less elaborate aeration system. This plant is just going into operation, and definite data on results are therefore not available at this time.

For similar reasons, although the iron content in the Miraflores water is a little lower than in the Colon water, the aeration basin becomes an important part of the Miraflores plant.

This basin, as designed, will be a reinforced concrete structure, rectangular in plan, 86 ft. wide by 125 ft. long, inside wall dimensions. The floor will be flat, resting throughout its area directly on clay foundation, and will have a thickness of approximately 6 ins. The side walls, 6 ins. in thickness, will extend 6 ft. above the floor, and are merely to prevent the waste of water that would result from the striking of the spray on the concrete floor after falling from a height of 15 to 20 ft. The elevation of the floor has been fixed at 126.

The 30-in. main from the raw water station will extend the full length of the north side of the basin and will be outside the wall, distant therefrom about 4.5 ft., and below the floor level approximately 2.5 ft. From the top of this 30-in. pipe will be taken off seven 12-in. pipes spaced at intervals of 16 ft. Each of these pipes will lead horizontally through the side wall on to the floor of the basin, and at a point 15 ft. inside will split into two 8-in. pipes, which will form a loop extending across the basin. The pipes when assembled will present the appearance of a grid of 14 8-in. pipes, with each pair cross-connected at the southerly side of the basin and fed by a single 12-in. line at the opposite side.

At intervals of 8 ft. on the 8-in. pipe will be located 4 in. outlets extending vertically upward. These outlets will be staggered in such a way as to cause them to be located in triangular plan. To each of these outlets will be located the bronze aeration nozzle. This will thus give a total of 105 nozzles.

The nozzle adopted will consist of a special flanged outlet within which will fit a bronze truncated cone having a face angle of 20° inclination from the vertical. The diameter of the outlet will be 3⅝ ins., but the maximum opening between the lip of the outlet and the face of the cone will not exceed 3/16 in. measured perpendicular to the cone for maximum required discharge. The cone will be held in place by an adjustable bronze bolt extending into a cross rib located just above the plane of the flanges connecting the nozzle to the 4-in. outlet from the 8-in. feed pipe.

The design of these nozzles was determined upon after experiments with nozzles of different types and by full sized tests had shown the character, volume, and shape of the spray delivered under varying heads.

Under operating conditions it is expected that the water from these nozzles will be thrown from 15 to 20 ft. into the air, and the tests showed that the water was broken from a thin sheet into coarse drops about two-thirds of the distance up, changing to spray at the top of the rise and falling as such to the basin floor.

Each nozzle at full pressure will deliver approximately 200,000 gals. per day, but the opening is so designed that when delivering 50 per cent of this amount at a low pressure, satisfactory breaking up of the water may be expected.

Outside the basin wall, in each 12-in. feed pipe, will be located two valves, one hand-operated and one hydraulically operated. The piping from the latter valves will lead to controllers located in a float box in the head house. This box will be connected by a 12-in. pipe to the receiving chamber at the entrance to the filter building, and the water level rising or falling in this box simultaneously with the level in the receiving chamber will actuate floats carrying stems to small piston valves, which will open or close the hydraulic valves at the aeration basin, cutting out or in the various banks of nozzles according as the throwing in or out of filters increases or decreases the draft on the sedimentation basin. By this means the quantity of water admitted to the aeration basin will be automatically regulated to meet the more or less constantly varying outflow to the clear-water basin, resulting from throwing filters in and out of service.

The float controller for the hydraulic valves will consist of a bronze shell, within which will be placed a cylindrical piston attached by a vertical stem to a copper float resting on the surface of the water in the float box. The shell has been so designed that the feed water for operating the hydraulic cylinders on the valves will pass into it through ports, and, according to position of the piston, will flow to the top or bottom side of the valve cylinder, under pressure. The water from the opposite side of the hydraulic valve piston will escape through a waste port in the shell. In short, the controller is a simple adaptation of a cylinder and piston valve for accomplishment through float movement of the same result obtained by the ordinary hand-operated four-way valve. The hand-operated 12-in. valves referred to are to be operated only at times when the occasion may arise to remove or repair the hydraulic valves.

The water will pass from the aeration basin to the mixing chambers in the head house over three weirs 12 ins. deep by 15 ft. in length, which will be simple slots in the back wall of the head house. These weirs will have their crests at such an elevation that for maximum flow the depth of water on the floor of the aeration basin will not exceed 9 ins.

Disastrous fires at the docks along the Seattle water front have led to an ordinance providing that all docks constructed hereafter shall be provided with fire walls spaced not farther apart than 500 ft. on centers and fire stops not more than 100 ft. apart. These provisions are not required in structures fully equipped with automatic sprinklers.

Waste-Preventing Fountain for Drinking and Horse Watering, Somerville, Mass.

The present article describes a simple, inexpensive, waste preventing drinking fountain, for the use of men, horses and dogs at Somerville, Mass. This fountain was devised for the two-fold object of saving water and, by using the pail system of watering and of abolishing large, open watering troughs, of preventing the spread of glanders among horses. The Somerville fountains are still being tried out with every indication of ultimate success. Our information pertaining to this fountain is taken from a paper by Mr. Frank E. Merrill, before the latest annual meeting of the New England Water Works Association. Mr. Merrill is Water Commissioner of Somerville and originated the design here described.

In ordinary open watering troughs, in which the flow is not controlled by a ball-cock, the waste of water is very high, amounting oftentimes to 99 per cent of the total flow through the trough. In the pail watering system, however, this waste is eliminated. The type of watering station here described has the advantage of low cost and small size. It can therefore be installed at frequent intervals in congested value districts. While the application of faucets to fire hydrants, for the purpose here described, is a simple and easy way of furnishing watering stations in sufficient frequency, this user of a city's fire service is not to be commended. This is especially true where hydrants are not equipped with outlet valves, and this use of hydrants should be regarded only as a temporary expedient.

View of Inexpensive, Waste-Preventing Watering Station for Men and Animals, Somerville, Mass.

The keynote of the Somerville watering station construction is "simplicity." It may be described as being in its essential parts, a piece of 12-in. cast iron pipe set in the ground with its bell end upwards, the face of the bell being 29 ins. above the sidewalk grade and the other end being a sufficient distance below the surface to obtain stability. A 1-in. service pipe from the street main makes into a 2-in. riser which comes up through the center of the standard and is held in position by a cast-iron strainer resting in the bell of the large pipe. This riser is capped with a side-outlet cross standing 12 ins. above the top of the bell end of the pipe, which forms a convenient ledge upon which to rest a pail while filling. Into one of the outlets of the cross is inserted a ¾-in. self-closing hose bibb so that a hose line may be attached if needed for any purpose; into two other outlets are inserted self-closing plain bibbs and into the outlet facing the sidewalk is fitted a bubbler controlled by a self-closing cock. In this condensed space there is found, therefore, opportunities at once for three teamsters to draw water and for another one to obtain a refreshing draught for himself.

Attached to the side of the 12-in. standard near the sidewalk grade is a bowl for dogs which is kept supplied with fresh water by the drip of the bubbler overhead, the water being caught in a tunnel set underneath the strainer and conveyed through a small pipe into the dog bowl.

The waste from the faucets and dog bowl is discharged into the interior of the 12-in. standard which has a cement bottom, at which point an opening allows the waste to escape into a drain pipe leading to the sewer. On the side of the 12-in. standard under the bubbler is fastened a step at a convenient height for children to reach the water from that fixture.

All the materials used in the construction of this watering station are, with the exception of the castings for the strainer, dog bowl and step, such as are found in every water works shop or supply house. The cost of the outfit made up in the shop and ready to set in position will be found to be about $20, and if as much more is added for the cost of installing it, with supply and drainage connections, there results a pretty complete water combination at a very moderate outlay of money.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design and to Methods and Cost of Construction

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The Highway Commission of Alabama and Some of the Results It Has Accomplished.

The efficiency of state highway commissions is best judged by the results accomplished. The southern states have been very slow to adopt the commission form of state road administration and in those states where it has been instituted the funds provided have been insufficient for the highest efficiency. Results obtained by these commissions have, however, been gratifying to the engineer and citizen that is anxious to see the expenditures of public funds placed in the hands of those trained in the fundamentals of economy and skilled in the details of satisfactory road construction.

The Alabama highway commission has authority to enforce its regulations as to the quality of the work accomplished under its direction. At the same time the ideas of local self government so strongly rooted in the minds of the inhabitants of the southern states have not been too rudely violated by making the commission autocratic. The commission controls the quality of work performed but the disposition of funds and the acceptance of state supervision is with the people and the county unit.

The division of state funds into small allotments while undesirable perhaps from a standpoint of economical work, has accomplished a much more important result than the saving of a few dollars. In each county that has asked state aid there has been constructed a section, or sections, of road that conform to the best principles of road building. The educational value of such work, as illustrated by the work of the United States Office of Public Roads and the beginnings of many other state highway commissions, is of extreme importance.

The commission has successfully supervised work accomplished by different methods of construction: by day labor, by convict labor and by contract. This fact demonstrates the flexibility and efficiency of a state highway commission in supervising work in a sparsely settled state, and the fact is also demonstrated that state supervision and control is not dependent on the building up of a large force of employes paid by the state. However, it is probable that future developments will indicate the need of absolute state control. But in the early stages of the development of state supervision in the south the authority vested in the Alabama commission appears to fit conditions.

The most important result accomplished by the commission, however, is in the unifying of the various types of road construction used throughout the state and promoting efficiency in local county organizations. Results in this form have been most marked. State road work is on a sound basis in Alabama. The foundation of efficient work once laid, large works may be undertaken with confidence that economical returns will be secured from the expenditure of public funds.

Preference of Employment to Residents of Cities.

A plan of employment which has much to commend it has recently been advocated in Chicago and will probably be put into effect in the near future. The purpose of the proposed plan is to give preference in the matter of employment to residents of Chicago. The scheme has been given countenance by the Chicago Association of Commerce, and Superintendent Calley, of the Bureau of Employment, Department of Public Welfare, with the approval of Mayor Harrison, has drafted a form of identification and employment card which first is to be made use of among those men out of employment who are on the lists of the county agent. The preferential employment card devised is drafted in the following form:

PREFERENTIAL EMPLOYMENT CARD.

Mr. .... is a resident of Chicago. He is supporting a family in Chicago and has complied with the conditions which give him first preference in employment, other things being equal.

Signature......................

Superintendent Bureau of Employment,
Department of Public Welfare.

Signature of owner..........................

Address ....

(Please return to Department of Public Welfare when employment has been secured.)

The back of the card contains the following:

DEPARTMENT OF PUBLIC WELFARE,
CITY OF CHICAGO.

PLEASE TAKE NOTICE—

1. That this card does not guarantee employment. It is simply an arrangement to identify the owner so that employers of labor in Chicago who desire and are willing to give preference in employment to residents of Chicago, all other things being equal, may feel assured that the holder of this card is a permanent resident of this city.

2. It is the purpose of the department of public welfare to add to the number of employers who have already indicated their willingness to recognize this card. This card should be retained by the owner until employment has been secured, because of its preferential value when large fields of employment are open in this city.

Signature......................

Superintendent Bureau of Employment,
Department of Public Welfare.

The problem of providing food and lodging (or rather employment so that men may earn money with which to buy these necessities) for the floating population of our large cities has long been recognized as a serious one, and very often this class of intermittent labor has taken the places of permanent residents who have families to support. In the main the class of men which concentrates in our large cities, especially in the winter, is inefficient, as it lacks the main incentives for work—pride in its work, and responsibility for the maintenance and welfare of others.

In our issue of November 18, 1914, we commented editorially on "Inefficiency in Hiring and Discharging Men"; and the subject under discussion is closely linked with that discussed in our previous issue. In the editorial referred to the fact was pointed out that, in a certain group of factories, it was necessary to hire 42,571 men within a period of one year in order to increase the permanent force of this group of factories from 42,000 to 48,697. In other words, only about 16 per cent of those employed were retained as a part of the permanent force. The economic loss of indiscriminate hiring and discharging of men is emphasized by the foregoing example, which we believe is not an exceptional case. It is often impossible for those in charge of employing workmen to determine definitely in advance the attitude which the men employed will take toward their work, or the ability of those men to perform successfully the particular kind of work required. With the active co-operation of those in charge of the city departments of public welfare, who are in a position to investigate individual cases, much of this inefficiency could be eliminated.

To be of real value this scheme of preferential employment must be adopted generally by cities. Little will be accomplished by its trial in a single city. The scheme has great possibilities, however, if cities throughout the country will adopt a similar plan of employment. Following the general adoption of such a scheme there should naturally follow an interchange of identification and employment cards between the different cities. In this way no injustice would be done to any worthy man who for good reasons moved from one city to another. The essential thing to accomplish is to eliminate from consideration the man who refuses to accept responsibilities—the man who expects food and shelter, but refuses permanent employment. The proposed plan should operate to do this, and should tend to better existing conditions which now only serve to increase the floating population of our principal cities.

Bituminous Nomenclature.

The definitions used in connection with bituminous road materials are fairly well established. Some confusion, however, exists in defining the methods of construction in which such materials are used. Simple terms are always best for general reference to a method or process, qualifying words or phrases being used to denote the various refinements in methods.

The term bituminous is generic in nature and used alone means nothing more than that the pavement is of a bituminous type. It is, however, a convenient word, although superfluous, and has come into general use. Its use merely differentiates the bituminous from the mineral cements.

The best results in the use of bituminous binders have undoubtedly been obtained in those pavements in which the mineral aggregate is proportioned for voids. By common usage the word concrete has come to mean a mineral aggregate composed of various sized particles, proportioned in such a manner that the percentage of voids is reduced to a minimum, bound together by a cement. Bituminous concrete, then, properly may be said to mean a pavement in which the aggregate is proportioned for voids and a bituminous cement used. Should a more specific term be desired say tar concrete, or still more specifically, water gas tar concrete and coal gas tar concrete. Likewise, say asphaltic concrete and, if desired, oil asphaltic concrete and natural asphaltic concrete. Sheet asphalt partakes more of the nature of a bituminous mortar but is in reality a bituminous concrete.

Macadam is fundamentally a pavement composed of stones none of which exceed, say, 3½ ins. in diameter, laid in such a manner that a mechanical interlocking is secured and bonded by the use of water and stone dust, loam or, in some cases, clay, asphalt or tar, Portland cement grout or silicate cement grout such as rocmac matrix, or other binders such as glutrin. The term macadam has, like the word bitumen, become generic in usage when applied to pavements and requires a modifier when referring to a method or process, i. e., water bound macadam, bituminous macadam and rocmac or glutrin macadam.

With this conception of a macadam pavement the term bituminous macadam is correctly used to denote a macadam pavement bound with bitumen. More specific reference requires the use of qualifying terms, such as, asphaltic macadam and tar macadam; likewise, say coal tar macadam and water gas tar macadam. Should reference to the method of construction be desirable say asphaltic macadam by the penetration method or asphaltic macadam by the mixing method. A similar method of reference is used for the tar products.

Different kinds of pavements such as the various types of macadam and Portland cement and bituminous concrete pavements are sometimes treated with bitumen after their completion. Such treatment may be referred to as a paint coat if only a light application is made, or a bituminous carpet if several applications are made and mineral aggregate incorporated with the bitumen to form a protecting coat varying from ¼ to ¾ in. in thickness.

The very light oils, tars and other dust palliatives are correctly classed with water as dust layers, although their value is greater than that of water. Patented bituminous mixtures are correctly called by their trade names.

The nomenclature outlined is that adopted by this journal as a matter of convenience and conforms to that used by most authorities on the use of bitumens. It is desirable that the interpretation of terms be universally the same and to this end convenient and simple terms are necessary in speaking and writing of bituminous methods and processes.

BRIDGES

Design and Construction of the San Jacinto St. Reinforced Concrete Bridge, Houston, Texas.

(Staff Article.)

The San Jacinto St. bridge is one of several reinforced concrete bridges which are included in a general program of bridge building by the city of Houston. It spans Buffalo Bayou and is located in the heart of Houston, forming a connecting link between the business section and one of the manufacturing and jobbing districts. Buffalo Bayou is a navigable stream, although at the site of this bridge the water traffic is confined to comparatively small crafts. The volume of traffic, however, is relatively large, the stream at this point having a width of about 100 ft. and a depth of 10 ft. A few miles further down is the Turning Basin, which is the head of navigation for ocean-going vessels and at which point the city of Houston is preparing to build extensive docks, wharves and warehouses. From the Turning Basin to the Gulf of Mexico—a distance of about 50 miles—a channel has just been dug to a depth of 26½ ft.

GENERAL FEATURES.

The bridge has a total width of 70 ft., with a 50-ft. clear roadway and two 8-ft. 11¾-in. clear sidewalks, supported on cantilever brackets. The total length of the structure is 500 ft., consisting of a skew arch with a clear span of 110 ft., 31-ft. reinforced concrete girder spans, and retaining wall approaches. The approaches have a combined length of 178 ft., and the bridge proper a length of 322 ft. between abutments. The arch is on a skew of 65° with the axis of the bridge. The columns supporting the girder spans have spread footing, while the arch is founded on pre-cast concrete piles. In addition to providing for a large volume of heavy traffic, provisions were made in the design for future street car traffic. The bridge is paved with 3½-in. creosoted wood blocks, the blocks being laid on a ½-in. mortar cushion on a 6-in. concrete base.

LOADS AND ALLOWABLE STRESSES.

Live Loads.—The live load assumed for the central portion of the roadway covered by car tracks consisted of two 50-ton cars en train plus 50 per cent impact, while that on the remainder of the roadway consisted of a 15-ton road roller with no allowance for impact. The live load on the sidewalks was taken at 100 lbs. per square foot. In designing the arch a variation in temperature of 40° F. was considered.

The trucks of the car considered are spaced 22 ft. on centers, the axles of each truck being spaced 6 ft. on centers. The distance between centers of rear truck on front car and front truck on rear car is also 22 ft. For the road roller, 6 tons are carried on the front axle and 9 tons on the rear axle, the axles being spaced 11 ft. on centers.

Allowable Stresses.—In general the recommendations of the "Joint Committee on Reinforced Concrete" were followed in the design. The elastic arch theory, as given in Turneaure and Maurer's "Principles of Reinforced Concrete," was followed in the design.

The allowable bearing value of the clay was taken at 4,000 lbs. per square foot for the spread footing. The piles under the abutments were figured for a bearing value of 15 tons each.

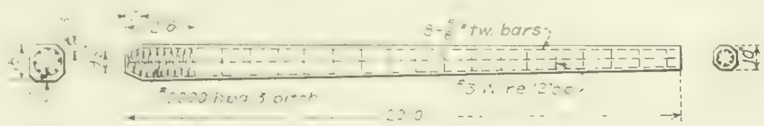

Fig. 1. Details of 20-ft. Reinforced Concrete Piles Used in San Jacinto St. Bridge, Houston, Texas.

CONCRETE PROPORTIONS.

The concrete used in the balustrade was a 1:2:3 mix, the coarse aggregate ranging in size from ¼ in. to ½ in.

A 1:2:4 mixture was used for piles, columns, footings, beams, slabs, retaining walls, arch and abutments. In the arch and abutments the coarse aggregate ranged in size from ¼ in. to 2 ins.; in the concrete for the remaining parts the maximum size used was 1 in.

A 1:2½:5 mixture was used for sidewalk foundations on fill and for the pavement base, with coarse aggregate up to 1½ in.

A 1:5 mixture was used for the conduits, hydrated lime being used to replace 10 per cent of the cement to insure a smooth surface for pulling the cables. The aggregate consisted of a mixture of sand and fine gravel.

DESIGN FEATURES.

Concrete Piles.—The abutments of the 110-ft. arch are founded on pre-cast reinforced concrete piles having a length of 20 ft. A total of 455 piles was required for the two abutments, the piles being made at the site. The piles were octagonal in section, with a small diameter at the butt of 15 ins. and at the tip of 10 ins. Figure 1 shows the details of these piles and indicates the type of reinforcement used. As the footings of the abutments extended 23 ft. below the water level in Buffalo Bayou a length of pile of 20 ft. was sufficient, the piles extending 6 ins. into the footing. The reinforcement used in each pile was sufficient to enable it to be picked up from a horizontal position by attaching a line near its end. Spiral hooping was used at the butt of the pile to insure it against damage in driving.

It was decided to use concrete piles, instead of timber ones, because a lesser number would be required, thus affording a better spacing and permitting the piles to be driven in a shorter time, which is an important item when an excavation is opened up to the depth reached in this case.

Approaches.—The approaches consist of an earth fill between reinforced concrete retaining

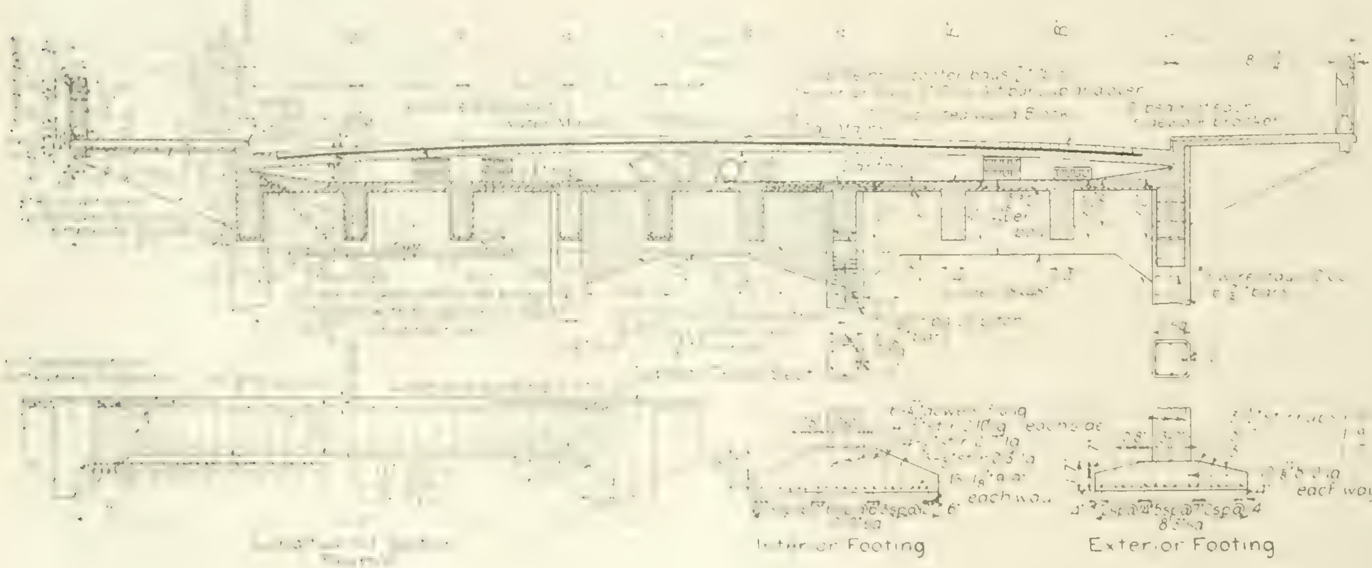

Fig. 2. Details of 31-ft. Girder Spans of San Jacinto St. Bridge. Note Earth Fill for Pipes and Conduits.

walls. The walls are of the cantilever type, counterforts being used for walls having a height greater than 12 ft. For a 12-ft. wall the thickness at the top is 8 ins., with a coping projecting 3 ins. beyond the face of the wall. The front face is vertical and the rear face has a batter of 1:15. The footing has a width of 7 ft. and a thickness of 15 ins. For a 17-ft. wall

to be dredged to a greater depth if required later. The footing of the south abutment has a width of 36 ft., and that of the north abutment a width of 40 ft.

The arch has a clear span of 110 ft. and a rise of 28 ft. 9 ins., the distance from the average water level to the soffit being 36 ft. 6 ins. The thickness of the arch rib at the crown

clam-shell buckets. Timber cofferdams were constructed, the sheeting for which consisted of 8x10-in. by 30-ft. timbers with tongue-and-groove strips, or cleats. These timbers were driven in place, and were braced with 12x12-in. members at 7-ft. intervals vertically as the excavation progressed. The timbers were well braced and bolted (see Fig. 4), and this type

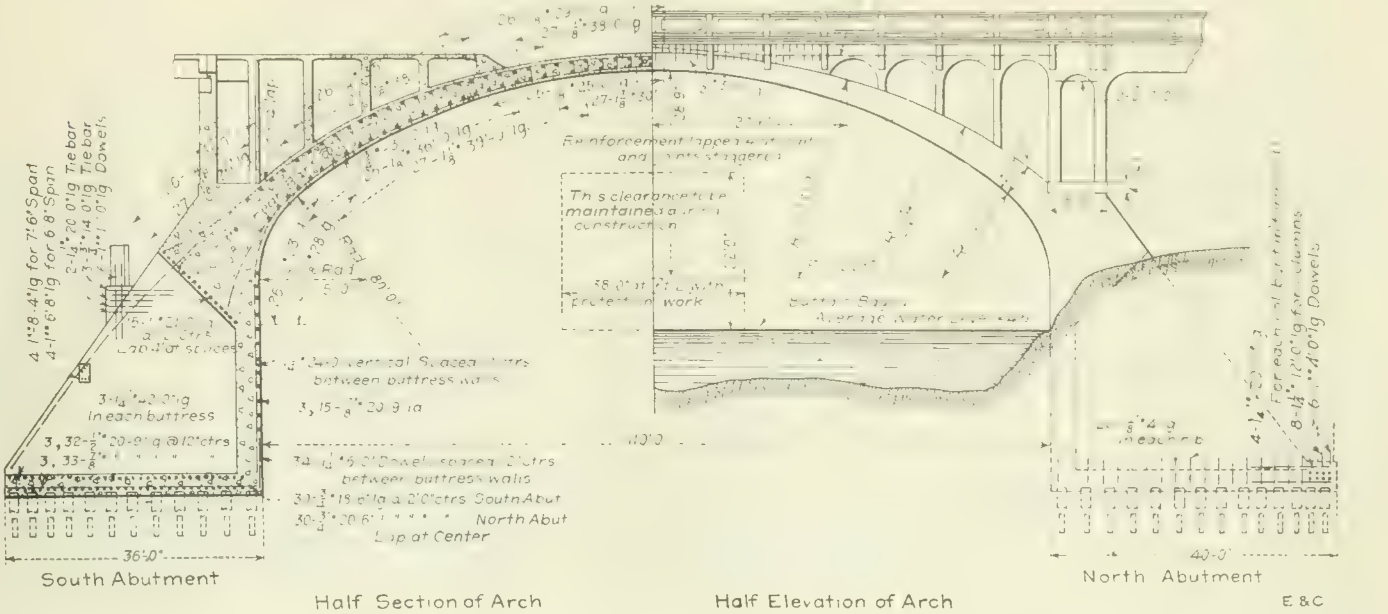

Fig. 3. Half Section and Half Elevation of 110-ft. Arch Span of San Jacinto St. Bridge. Note Buttress Type of Abutments and Depth of Footings Below Water Level.

the thickness at the top is 8 ins. and at the bottom, 11¼ ins. The footing has a width of 10 ft. and a thickness of 15 ins. The counterforts have a width of 14 ins. and are spaced 9 ft. on centers. The footings of both types project 12 ins. beyond the face of the walls. The retaining walls are reinforced with "corrugated" bars.

Girder Spans.—The girders, which have a section of 18x48 ins. and are on a skew of 65° with the axis of the bridge, are spaced 35 ft. on centers and are carried on four lines of reinforced concrete columns. The outside lines of columns are spaced 50 ft. on centers, measured at right angles to the axis of the bridge, or 55 ft. 2 ins. on centers, measured along the girder axis; the inside lines are each 7 ft. 6 ins. from the center line of the bridge, measured at right angles to the axis of the bridge. The floor slab is carried on ten lines of floorbeams. The inside lines of beams have a depth, including the floor slab, of 3 ft. 2 ins. and a width of 16 ins.; the two outside lines have the same depth but their width is increased to 18 ins.

The sidewalks are carried on cantilever brackets, which project 10 ft. beyond the center lines of the outside columns.

The columns have a section 2 ft. square, the inside lines being reinforced with vertical and spiral reinforcement. The vertical reinforcement of the outside columns is arranged in the form of a square, the horizontal reinforcement consisting of hoops. The spread footings of the inside lines of columns are 10 ft. square, while those of the outside lines are 8 ft. 3 ins. square.

Figure 2 shows details of the girder spans and indicates the type of construction used. It will be noted that the floor slab carries an earth fill in which are embedded the water and gas mains and the conduits.

Arch Span.—The arch and its abutments contain about three-fourths of the total yardage in the bridge, the abutments being of the buttress type. A heavy skewback is supported on nine thick buttresses and a front wall which is securely dowelled to the footing, the latter being 3 ft. thick and reinforced both top and bottom. As has been noted the abutments are founded on reinforced concrete piles. The footings were carried to a depth of 23 ft. below the water level, which will enable the channel

is 2 ft. 6 ins. and at the abutment, about 4 ft. 8 ins.

Figure 3 shows a half section and a half elevation of the arch. This drawing indicates the type of construction and gives the governing dimensions and the system of reinforcement used.

Expansion joints are provided at each end of the arch and also at the end abutments. At these joints ¾-in. steel plates are set under the beams, and sheets of zinc are placed between the bearing plates.

Balustrade.—The concrete balustrade of the bridge extends 3 ft. 4¾ ins. above the sidewalk

of cofferdam proved very effective. "Emerson" pumps were used in the cofferdams, although the amount of water pumped was not large, due to efficient caulking.

A hoisting derrick was located at each cofferdam, and the material after being excavated was hauled away in dump cars over a tramway. This tramway was also used to transport materials throughout the work.

Casting and Driving Piles.—In making the concrete piles several rows of large sills were laid on the ground, and 2x12-in. planks were laid at right angles to the sills. The forms consisted of 2-in. dressed pine. The reinforcement

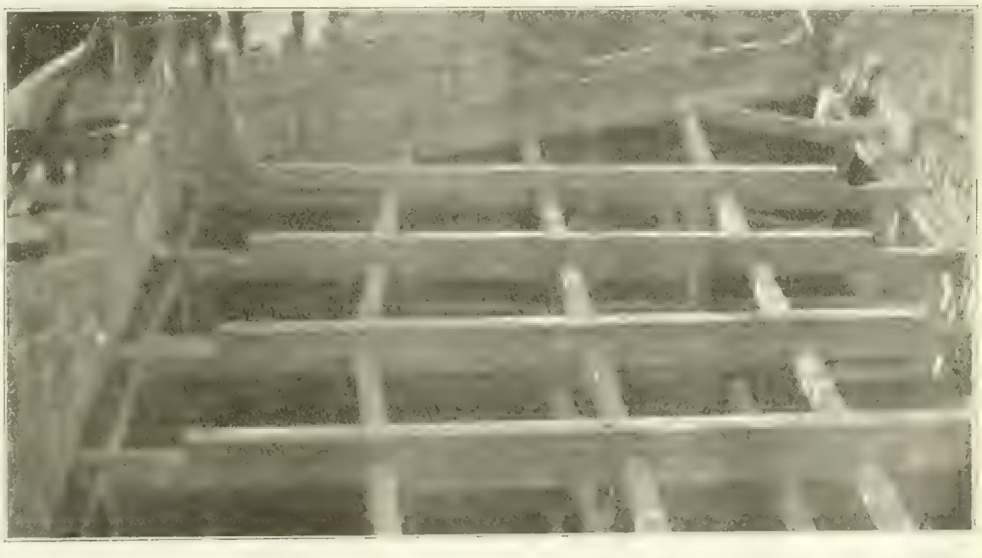

Fig. 4. View of Top of Cofferdam for Arch Abutment of San Jacinto St. Bridge. Note Heavy Bracing and 8x10-in. Timber Sheeting.

slab. It is reinforced for temperature stresses only. Between the 10-in. posts, which are spaced 10 ft. 4 ins. on centers, the balustrade is paneled, nine oblong openings being left in each panel.

CONSTRUCTION FEATURES.

Excavation and Cofferdams.—Excavation for the abutments was made with orange-peel and

for each pile (see Fig. 1) was first made up into a unit and then placed in the form. After the reinforcement had been centered in the form the concrete was puddled around it. Although the specifications required that the piles be cured for only 30 days, it was decided to cast them early, and as a result no pile was driven until it was five months old.

The piles were driven by a "Vulcan" steam trip hammer through strata of red sandy clay and stiff clay. The hammer weighed 5,000 lbs. and had a 42-in. drop, and the piles were driven until the penetration did not exceed about ¼-in. Although the concrete at the head of the [illegible] down to the reinforcement in many cases the pile was not otherwise damaged in driving. A number of different methods were tried to protect the head of the pile, the most satisfactory cushion tried consisting of cement sacks filled with cotton waste.

Forms and Centering.—Timber forms were used for all parts of the bridge, the forms being constructed in units which could be handled by two or three men and which could be wrecked easily without wrecking the adjacent units. The forms were made of 1½-in. lumber. The supports for the beam and slab forms were 6x6-in. timbers spaced about 4 ft. on centers. They were capped with timbers of the same size and were well cross braced.

In casting the concrete conduits wood cores covered with galvanized iron were used. The railing forms were made light and free from joints, which made the surface finishing (which was done with carborundum blocks without wetting) an easy process. The forms were brushed with parafin oil, and the railing was cast monolithic. The forms were removed after 24 hours.

As Buffalo Bayou is a navigable stream it was necessary to leave a large opening in the arch centering and its supports. This opening was about 38 ft. wide and 24 ft. high above the average water level. The arch centering was supported by timber piles spaced in such a manner as to throw a maximum load of 12 tons on any pile. It was braced with the object of counteracting, as far as possible, the "bucking-up" tendency of the forms during the concreting of the arch. Wedges were used to bring the decking to the proper elevation. Nine lines of special 30-in. 200-lb. "Bethlehem" beams were used over the channel to support the arch and its centering. A special study was made of side thrusts, etc., caused by the arch being on a skew, and a system of struts and braces was used to resist the thrusts. The centering was designed in the St. Louis office of the contractor.

Figure 5 shows a cross section of the arch centering and its supports. This drawing gives the dimensions of the members, their spacing and indicates the type of construction used. It will be noted that a line of protection piles was driven along each side of the opening. Figure 6 shows a view of the partially finished arch centering, and Fig. 7 shows the completed centering and supports.

Placing Reinforcement.—Great care was used in wiring the reinforcing steel securely in place to prevent displacement during concreting. Spacers, consisting of beveled concrete blocks with embedded wires for fastening them to the bars, were used at the sides and bottoms of beams. The upper and lower layers of arch reinforcing bars were connected with ⅜-in. shear bars.

Concrete Plant.—The chuting system was used in placing the concrete. As another concrete bridge had just been constructed a short distance from this structure, the same plant was used as for the first bridge, which required the use of long chutes. The hoisting tower was 185 ft. high, the concrete being hoisted in it by a 25-hp. motor. A second tower was built near the new bridge, the two towers being connected by a chute about 500 ft. long. Between these towers there was erected a third tower to sup-

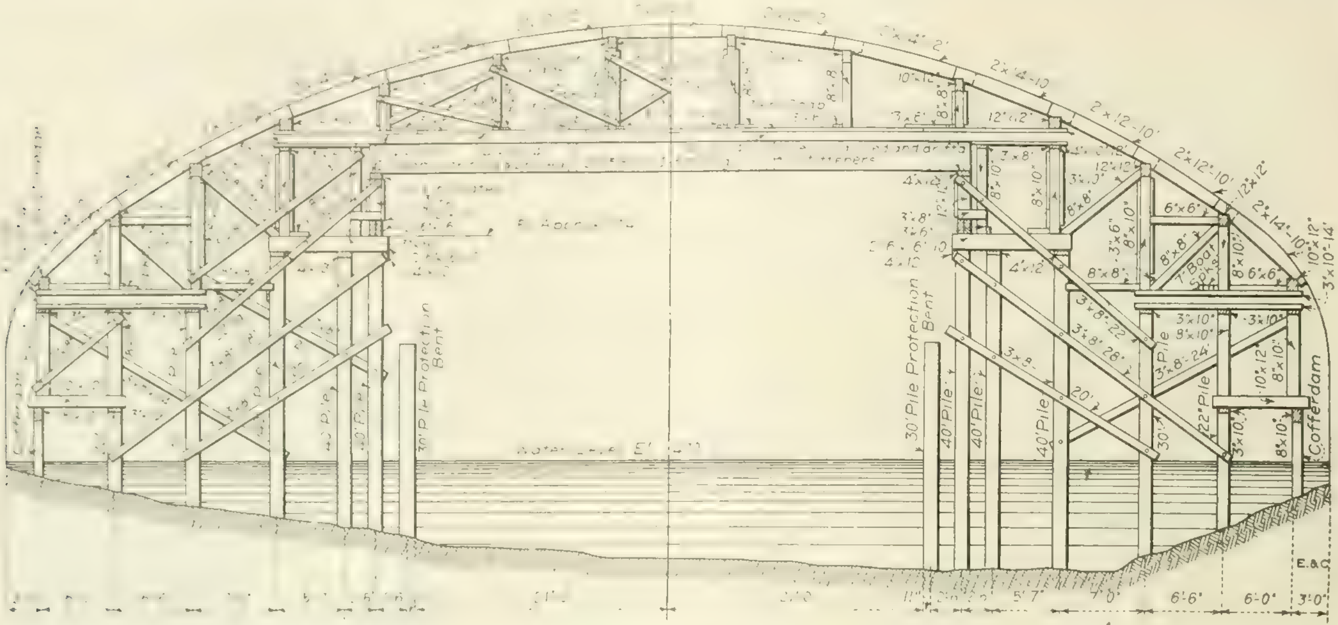

Fig. 5. Details of Arch Centering and Supports for 110-ft. Span of San Jacinto St. Bridge. Note Size of Opening for Navigation.

Fig. 6. View of Partially Completed Centering for 110-ft. Span of San Jacinto St. Bridge.

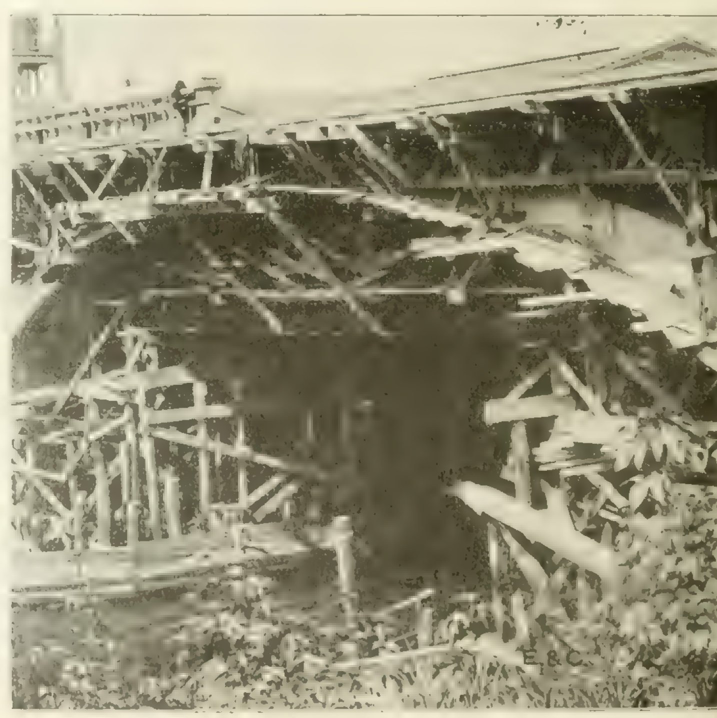

Fig. 7. View of Centering and Supports for 110-ft. Span of San Jacinto St. Bridge. Note Opening for Navigation.

port a branch chute. Figure 8 shows a view of a part of the main chuting system and the distribution chutes. The maximum distance which the concrete was chuted was about 900 ft. Before concreting, the chutes were always washed with water, followed by a batch of cement grout.

Fig. 8. View of Portion of Concrete Spouting System for San Jacinto St. Bridge. Note Wooden Distribution Spouts.

The mixing plant was located on the bank of the stream. The sand was hoisted directly into the hoppers from barges, and the gravel and broken stone were unloaded from cars on a spur track. A ½-cu. yd. "Ransome" mixer was used, although a small quantity of concrete was mixed with a small portable gasoline mixer.

Concreting Procedure.—The abutments were concreted in sections, each section containing a day's run. In addition to keying the sections, the concrete surfaces were roughened. One-half of the reinforcing bars of the abutments projected a distance of 4 ft. and the other half a distance of 8 ft. to give staggered splices. A bulkhead was placed on a radial line at each skewback, keys being provided for the arch ring.

The arch ring was concreted continuously, beginning at both skewbacks and meeting at the crown. The concrete distribution system is shown in Fig. 8. The concreting of the arch ring required about 70 hours. The maximum settlement of the piles which supported the centering was ¾-in., while the 30-in. I-beams, deflected about ½-in.

QUANTITIES OF MATERIALS AND COST.

The arch ring between skewbacks contains 816 cu. yds. of concrete, the total quantity placed in the entire bridge being about 5,500 cu. yds. About 200 tons of reinforcing bars were used. The total cost of the bridge, including engineering and supervision, was approximately $150,000.

Figure 9 is a view of the completed structure. This view shows clearly the general design features of the arch and girder spans.

PERSONNEL.

The work described in this article was in charge of E. E. Sands, city engineer of Houston. The bridge was constructed by the Wm. P. Carmicheal Co., of St. Louis, Mo. W. W. Washburn was resident engineer for the city, to whom we are indebted for the data upon which this article is based.

A motor omnibus service in the national territories of Argentina is under consideration by the government. The lack of transportation facilities in some of the tobacco-growing districts has caused great losses and the establishment of a motor omnibus service would relieve that situation.

Properties of Vanadium Steel and Its Use in Long-Span Bridges.

Some interesting and instructive comment has been brought out in the discussion of J. A. L. Waddell's paper on "The Possibilities in Bridge Construction by the Use of High-Alloy Steels." The following data are from the discussion of this paper by George L. Norris, in Proceedings, American Society of Civil Engineers, Vol. XL, p. 2342. The need of a moderately cheap structural material possessing a high elastic limit and capable of being worked readily in the shop has long been felt by designers of long-span bridges.

With alloy steel it is entirely feasible to obtain elastic limits of from 90,000 to 100,000 lbs. per square inch by heat treatment, and even in the condition as rolled, but this, of necessity, would increase considerably the difficulties of shop manipulation. In the condition as rolled, the alloy steels would naturally be too variable in hardness for safe use.

In the case of nickel steel, in order to attain an elastic limit of 60,000 lbs. or more, it is necessary to increase the percentage of carbon to undesirable limits.

To produce steel having an elastic limit approaching 100,000 lbs. per square inch, and workable under ordinary shop manipulations, the writer believes that recourse will be had to the use of vanadium alone, or with some other metal such as chromium in the steel. Vanadium is undoubtedly the element which, together with carbon, acts with the greatest intensity in improving alloys of iron; that is, in very small percentages. The quantity of vanadium generally alloyed with steel, excepting tool steels, is about 0.15 or 0.20 per cent. The price of vanadium has been reduced 60 per cent within the past three years, and it is now low enough to bring it within consideration for use in steel for eye-bars and other parts of long-span bridges.

Vanadium has generally been used in combination with chromium, although when used with nickel or nickel and chromium it gives greatly increased physical properties; these latter combinations are naturally more expensive. Added to simple carbon steel, especially if the percentage of manganese is more than 0.60 per cent, it gives an increase of about 40 per cent in the elastic limit and about 20 per cent in the tensile strength, with practically the same or even greater ductility.

What is known as Type "A" chrome vanadium steel will fulfill the requirements of high elastic limit and be workable under shop manipulations. The cost of this steel would be only slightly in excess of that of 3¼ per cent nickel steel. Its chemical range would be:

	Per cent.
Carbon	0.17 to 0.27
Manganese	0.40 to 0.60
Chromium	0.60 to 0.90
Phosphorus, not more than	0.05
Sulphur, not more than	0.05
Vanadium, not less than	0.15

As rolled in plates, shapes, and bars this steel would have approximately the following properties: Elastic limit, 60,000 to 80,000 lbs. per square inch; tensile strength, 85,000 to 110,000 lbs. per square inch; elongation in 8 ins., 12 to 20 per cent; and reduction of area, 45 to 60 per cent.

By the simple operation of heating to 1,100 or 1,150° F. any irregularities in hardness and strength, due to variation in rolling temperatures or uneven cooling, can be removed without appreciably decreasing the elastic limit and tensile strength, and, at the same time, increasing the ductility. This operation would not be an expensive one, and experience might show that it would be unnecessary for this grade of steel.

Type "A" chrome vanadium steel can be sheared, punched, reamed, bent, etc., without any considerable increase of shop manipulations.

Much higher elastic limit and tensile strength can be obtained from this steel by heat treatment, quenching and tempering, but this would increase the shop manipulations very materially, necessitating drilling instead of punching and reaming. It would also add materially to the cost. Any bends would have to be made before heat treatment.

Rather than heat-treat steel of this type it would be preferable to attain elastic limits of 80,000 to 100,000 lbs. by increasing the carbon percentage, and normalize the steel by heating to 1,100° to 1,150° F.

The range in chemical composition for this grade would be:

	Per cent.
Carbon	[illegible]
Manganese	[illegible]
Chromium	[illegible]
Phosphorus, not more than	[illegible]
Sulphur, not more than	[illegible]
Vanadium, not less than	0.15

Plates ⅛ in. thick, in the upper range of this composition, have shown the following

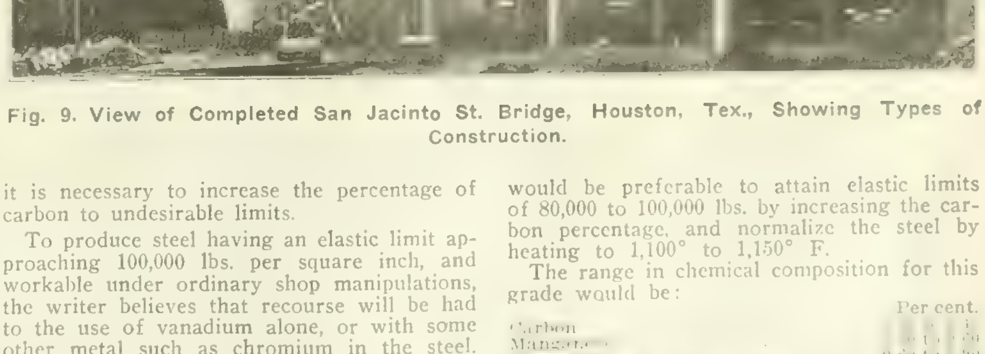

Fig. 9. View of Completed San Jacinto St. Bridge, Houston, Tex., Showing Types of Construction.

physical properties: Elastic limit, 125,000 lbs. per square inch; tensile strength, 145,000 lbs. per square inch; elongation in 2 ins., 18 per cent; and reduction of area, 58 per cent. These plates could be punched and sheared as rolled, but it would be advisable, no doubt, to normalize, as before described, and also to drill rather than to punch the holes.

Plates ⅝ in. thick, at the low range of this composition, have shown the following physical properties: Elastic limit, 83,000 lbs. per square inch; tensile strength, 116,000 lbs. per square inch; elongation in 2 ins., 20 per cent; and reduction of area, 45 per cent.

Normalized, this grade of steel would give, in plates and shapes, the following physical properties: Elastic limit, 75,000 to 100,000 lbs. per square inch; tensile strength, 100,000 to 125,000 lbs. per square inch; elongation in 8 ins., more than 12 per cent; and reduction of area, more than 50 per cent.

The cost of these chrome vanadium steels would be about 3 cts. per pound more than for ordinary carbon steel.

The writer believes than a simple carbon vanadium steel would prove commercially more attractive than the vanadium steels containing chromium or nickel, or both. It is possible to obtain a very material increase in elastic limit by the addition of vanadium to a simple carbon steel, and such steels can be manipulated almost, if not quite, as readily as ordinary carbon steel.

Tests from 1 1/16-in. round, rolled bars of acid open-hearth casting steel of the following composition give a very good idea of what can be expected from this type of steel and how it compares with simple carbon acid open-hearth casting steel:

	Per cent.
Carbon	0.26 to 0.27
Manganese	0.60 to [illegible]
Silicon	0.25 to 0.25
Vanadium	0.21 to 0.00
Phosphorus, less than	0.05
Sulphur, less than	0.05

These tests gave the following results:

Elastic limit, lbs. per square inch	70,000 to 56,000
Tensile strength, lbs. per square inch	88,000 to 72,000
Elongation in 2 ins., per cent	[illegible] to 31
Reduction of area, per cent	57.5 to 58

From these and other tests, it would seem feasible to specify as follows, for this type of steel:

	Per cent.
Carbon	0.20 to 0.30
Manganese	0.60 to 0.80
Phosphorus, not more than	0.05
Sulphur, not more than	0.05
Vanadium, not less than	0.15
	Lbs. per sq. in.
Elastic limit	50,000 to 70,000
Tensile strength	[illegible] to 100,000
	Per cent.
Elongation in 2 ins., not less than	20
Reduction of area, not less than	[illegible]

The additional cost af vanadium steel of this grade over that of ordinary carbon steel should not be more than 1 ct. per pound, and very likely it would be less.

For built-up members of vanadium steel, the writer believes that vanadium steel rivets should be used, in order to utilize more fully than would be otherwise possible the high physical qualities of the vanadium steel shapes and plates. The rivets could be either of chrome vanadium steel or simple carbon vanadium steel. The composition for chrome vanadium steel rivets should be:

	Per cent.
Carbon	0.15 to 0.20
Manganese	0.60 to 0.80
Chromium	0.40 to 0.60
Phosphorus, not more than	0.05
Sulphur, not more than	0.05
Vanadium, not less than	0.15

This steel, as rolled in rounds, would have the following properties: Elastic limit, 50,000 to 65,000 lbs. per square inch; tensile strength, 70,000 to 90,000 lbs. per square inch; elongation in 8 ins., more than 18 per cent; and reduction of area, more than 50 per cent.

Single and double shear tests of rivets of steel of this type show values for single shear of 20 per cent and for double shear of 30 per cent greater than for ordinary rivets with a tensile strength of 5,000 lbs. per square inch.

The simple carbon vanadium steel should have a chemical range of:

	Per cent.
Carbon	0.15 to 0.20
Manganese	0.60 to 0.80
Phosphorus, not more than	0.05
Sulphur, not more than	0.05
Vanadium, not less than	0.15

This steel, as rolled in rounds, would have the following properties: Elastic limit, 40,000 to 55,000 lbs. per square inch; tensile strength, 65,000 to 85,000 lbs. per square inch; elongation in 8 ins., more than 18 per cent; and reduction of area, more than 50 per cent.

There should be no difficulty in driving rivets of either of these types of steel.

In the case of eye-bars, possibly the conditions are more favorable for the use of alloy steels than for built-up members. They can be more readily and advantageously heat-treated to develop high elastic limits.

In 1909 a number of tests were made of full-sized eye-bars, heat-treated, of chrome vanadium and chrome nickel vanadium steel. These eye-bars were made from bars 14x2 ins., had 34-in. heads with 12-in. pin-holes, and were 25½ ft. long over the pin-holes.

The chrome vanadium steel bars experimented with were too high in carbon, and the results obtained for elongation in 20 ft. were not quite as good as in the case of the bars from the chrome nickel vanadium steel. Tests from this latter steel gave results ranging as follows, depending on the drawback or annealing temperature after quenching:

	No. 1.	No. 2.
Elastic limit, lbs.	63,280	80,480
Tensile strength, lbs.	93,500	99,800
Elongation, 12 ins., per cent	35	32.5
Elongation, 20 ft., per cent	14.2	7.9
Reduction of area, per cent	50.8	52.3

Tests on 2x½-in. specimens turned up from the disks cut out in machining the eyes check the elastic limit and the tensile strength obtained from the full-sized bar very well. A test from the eye disk of bar No. 2 showed:

Elastic limit, lbs.	83,040
Tensile strength, lbs.	94,140
Elongation, 2 ins. per cent	25
Reduction of area, per cent	71.9

The chemical composition of this steel was approximately:

	Per cent.
Carbon	0.25
Chromium	0.90
Nickel	1.20
Vanadium	0.17

Based on these tests, it is perfectly feasible to specify as follows for heat-treated eye-bars of chrome vanadium or chrome nickel vanadium steel:

Elastic limit, lbs.	65,000 to 80,000
Tensile strength, lbs.	85,000 to 105,000
Elongation in 2 ins., per cent	20
Reduction of area, per cent	50

The cost of heat treatment for eye-bars would probably be from ¼ to ⅜ ct. per pound. The cost per pound of the chrome vanadium or chrome nickel vanadium steel would be about 3 cts. more than that for carbon steel.

The writer believes that eye-bars made from simple carbon vanadium steel of the following composition, either heat-treated, quenched and annealed, or normalized, heating to about 1,100° or 1,150° F. after the heads have been forged, will give elastic limits approaching those of the more expensive chrome or chrome nickel vanadium steels:

HEAT-TREATED OR NORMALIZED.

Chemical Composition.	Per cent.
Carbon	0.30 to 0.40
Manganese	0.60 to 0.80
Phosphorus, not more than	0.05
Sulphur, not more than	0.05
Vanadium, not less than	0.15
Physical Properties.	
Elastic limit, lbs.	60,000 to 75,000
Tensile strength, lbs.	85,000 to 100,000
Elongation in 2 ins., more than, per cent	.18
Reduction of area, more than, per cent	.45

The writer has given considerable attention to simple carbon vanadium steel containing from 0.60 to 0.80 per cent of manganese. This steel will be the commercial or every-day vanadium steel of the immediate future. It is much cheaper than the vanadium steels containing chrome or nickel. It is at least 40 per cent better than simple carbon steel of otherwise the same composition, and is bound to be used extensively in the near future for rails, general and locomotive forgings, and special structural purposes. It presents no especial difficulties in manufacture over simple steel.

The quantity of vanadium which remains in the steel is about 80 per cent of that added. It is very evenly distributed. No instances of segregation are known, and it has a strong influence in overcoming the segregation of other elements, particularly carbon.

Vanadium will readily alloy with nickel, and better results can be obtained from a 2 per cent nickel vanadium steel than from a 3½ per cent straight nickel steel of the same carbon content.

There would be absolutely no advantage gained in the use of titanium with vanadium. Vanadium is an alloying metal and is used as such, not as a scavenger or deoxidizer. It is only the vanadium which alloys with the steel that can be considered as influencing the quality of the latter. Titanium has a little merit as a deoxidizer over silicon or aluminum, and its action is apparently a surface reaction. It is a question whether the observable reaction, when evident, is not with the highly oxidizable basic slag with which it comes in contact, or even with the atmosphere.

WATER WORKS

Works for the Improved Water Supply of Columbus, Ga.

A new water supply will soon be secured by Columbus, Ga. The plant now in use will be replaced except for the distribution system which will be extended and reinforced. The new supply will be drawn through a 30-in. suction line, 400 ft. in length, extending out to the intake crib in the Chattahoochee River just above its junction with Roaring Creek. The location of the intake crib is shown in Fig. 1. The two pumps drawing from the crib will be direct-connected centrifugal pumps, each of 10,000,000 gals. daily capacity. The pump discharge lines will run to the sedimentation reservoir which will have a capacity of 40,000,000 gals. This will provide a sedimentation period of from 8 to 15 days, according to the rate of consumption, before the water is treated in the purification plant. The general layout of the purification plant is shown in Fig. 2.

The water will enter the sedimentation reservoir at the upper end and be taken out through a gate tower near the lower end, arranged so that the water will always be taken from near the surface, after having secured the maximum improvement by plain subsidence. From the reservoir the water will flow by gravity to a mechanical filtration plant of 6,000,000 gals. capacity per day, passing through a coagulating basin of 1,200,000 gals. capacity, to better prepare the water for successful treatment by the filters. From the filters the water will flow by gravity into a filtered water reservoir having a capacity of 2,500,000 gals. From the filtered water reservoir the water will flow by gravity to the pipe distribution system in the city. A 50,000-gal. tank on a 75-ft. tower is to be erected near the filter plant to provide available water under suitable pressure for use around the filter plant. There will be two buildings—the pumping station and the filter house.

Intake.—The supply-line will be of 30-in., bell and spigot cast iron pipe. This will extend from the intake crib to the sand pit and suction chamber of the new pumping station. The pipe will be laid on the rock bottom of the river, which will be blasted out for a suitable trench where necessary to secure the proper grade of the pipe. The fall of the supply line is 1 ft. in about 375 ft. from the crib to the sand pit. Where the pipe is laid on the river bottom it is to be secured to the bottom by a sufficient number of strong bands securely anchored into the rock bottom by bolts driven with wedge spreaders at the bottom, the intention being to have the suction securely fastened in place either in a rock trench in the river bed or securely anchored to the rock bottom. The crib intake will have two 30-in. by 30-in. openings with cast iron linings through the wall, with screens constructed at the outer edge of the wall, as shown in Fig. 1. It will also be covered with a reinforced concrete cover, except for a section of 2 ft. wide, which is to be made of iron bars hinged and securely fastened to the top of the intake chamber.

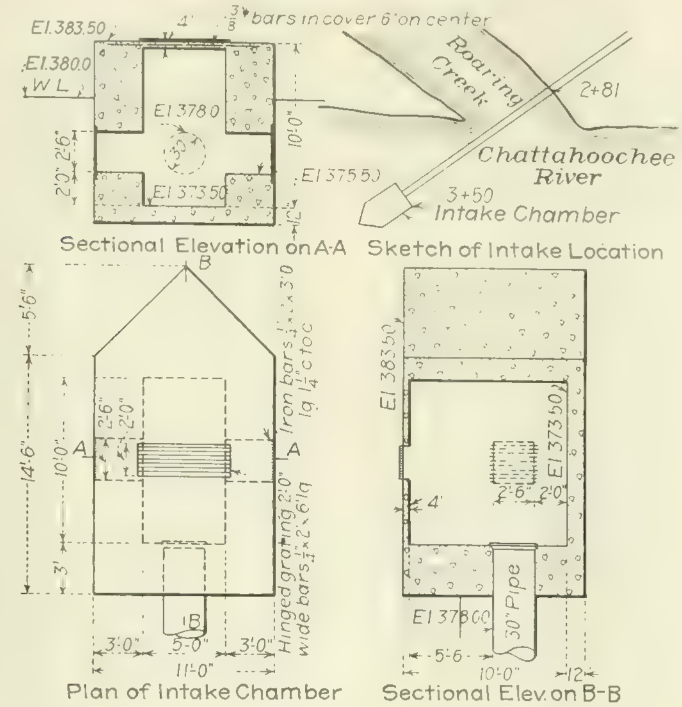

Fig. 1. Details of Intake Crib of Proposed New Water Supply, Columbus, Ga.

Pumping Station.—The pumping station will be 40x50 ft. in plant. The building will be of 1:3:5 concrete with a slate roof. Hydrated lime will be used for waterproofing the lower portion of the substructure. The roof trusses of the station building will be of long leaf pine. The same grade of timber will be used for purlins and rafters. The sheathing will be 1½x4-in. tongue and grooved stock. The roofing slate will be Georgia or Buckingham slate, cut 12x18 ins. and laid with a lap of 3 ins. Galvanized iron nails will be used to fasten down the slate. The top ends of all upper courses under the ridge covering, and for 2 ft. from all eaves, will be bedded in "slater's" cement. A layer of heavy building paper will be laid beneath the slate with a lap of 2 ins. The downspouts will be 3 ins. in diameter, with strainers at top and elbows near the ground fitted into an 8-in. pipe sewer drain.

The bottom of the sand pit will be 7½ ft. lower than the invert of the supply pipe. The pit will extend the full width of the building and will be 9 ft. wide. An 8 in. sand pipe will extend along the center line of the pit bottom. Shear valves placed in this pipe will be operated by rods extending to the concrete roof of the pit, which is outside but adjacent to the station building. The sand suction pipe will lead from one end of the pipe described to the centrifugal sand pump located in the station building. The sand pump discharge line is also 8 ins. in diameter. The suction well proper lies on the opposite side of the station building end wall from the sand pit. The two chambers are connected by three openings through the wall. These openings, which are at the same level, will be 12 ft. long by 3 ft. high. Bar screens, consisting of ½x3-in. iron bars spaced 1½ ins. on centers, will be fitted into these openings. The suction well will be 5 ft. wide, 14 ft. 6 ins. deep and will extend the full width of the station building. The pump suctions will be 18 ins. in diameter terminating 4 ft. above the well bottom and fitted at their lower ends with foot valves and strainers.

Sedimentation Reservoir.—The capacity of this reservoir will be 40,000,000 gals when filled to within 3 ft. of the top of the reservoir banks. The raw water enters the sedimentation reservoir through a 24-in. cast iron pipe line extending across its upper end. The discharge from this supply pipe is from 5 equidistant 24x10-in. tees. Each pipe joint is supported on a reinforced concrete pier 3 ft. square at the bottom and 1x3 ft. at the top. Figure 3 is a vertical section through the reservoir wall, gate tower and outlet pipe leading to the coagulation basin. The basin will be built in excavation and embankment, it being the engineer's intention that enough material be excavated within the lines of the reservoir to make the embankment. It is believed that all the material in excavation will be suitable for promiscuous placing in the embankment, but should it develop otherwise, the better material will be placed on the inside of the embankment. The embankment will have a width at the top and inside and outside slopes as shown by Fig. 3.

The material may be taken out of the excavation with any vehicles desired, but it will be placed in the embankment in layers not exceeding 12 ins. and each layer will be well moistened by sprinkling cart or hose and well compacted with a grooved roller weighing not less than 150 lbs. per lineal inch of roller. Before a new layer is put on, the finished and rolled previous layer will be run over with a light harrow to loosen the surface. The number of times that the roller will pass over each layer will be determined by the character and dryness of the material, but it will be sufficient to make a well compacted embankment.

Before the embankments are begun, the surface to be covered and where excavation is to be made, will be stripped clean of all vegetation and surface soil to the subsoil, the strippings to be used in building up the outside face of the embankment. The surface thus exposed beneath the embankments will be plowed to a depth of 8 ins. and harrowed until all the surface lumps are broken up, then rolled and treated in the same manner as described above for other layers in the embankment. When the embankment is completed the inside will be carefully dressed to true lines and slopes.

The upper portion of the reservoir will have a 4-in. concrete lining as shown in Fig. 3. Before placing this lining the earth embankment will be trimmed down to a depth of 12 ins. to compact material. Allowance for this trimming will be made in building up the embankment. The lining will be constructed in sections 10 ft. in horizontal width. It will be floated and troweled to a smooth finish surface.

The openings through the embankments will be as follows: A 30-in. cast iron outlet pipe, weight 292 lbs. per lineal foot, and a 24-in. cast iron inlet pipe weight 204 lbs. per lineal foot. The outlet pipe will be laid in trenches of proper depth beneath the embankments with two concrete cut-off walls, at intervals of 20 ft. The trenches will be filled with material selected to pack well, to be filled in 12-in. layers properly wet and thoroughly packed with heavy rammers. The outlet pipe will extend from the gate chamber inside the reservoirs to the coagulating basin as shown in Fig. 2.

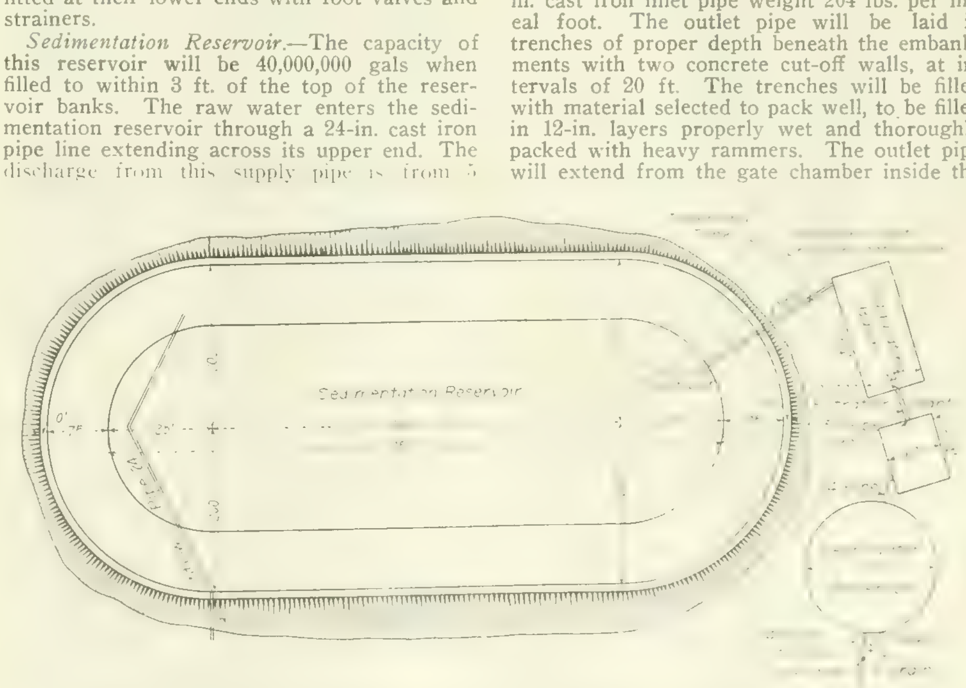

Fig. 2. General Plan of Reservoirs and Filtration Plant, Columbus, Ga.

The gate chamber, shown in detail in Fig. 4, will be built of 1:3:5 concrete. The foundation will be 3 ft. thick, 11 ft. 6 ins. square at the bottom and 9 ft. at the top. The walls will be 18 ins. thick at the bottom and 12 ins. at the top. Inside the chamber will be 5 ft. square and 25 ft. high. The chamber will be equipped with two 24 in. by 36 in. and one 12x12 in. sluice gate with riser stems and platform stands as shown.

Filter House.—The filter house will be a brick and reinforced 1:3:5 concrete structure with stone trimmings and slate roof. Filter walls will be waterproofed by the addition of hydrated lime. Brick walls will be 12 ins. thick. The belt course around the entire building will be of gray granite. This course will be 8 ins. high, extend into the wall 6 ins. and project 2 ins. beyond the face of the wall with a 2 in. chamfer on top. The roof trusses, purlins, rafters and sheathing will be of long leaf yellow pine. The roofing slate will be as described for the pumping station building.

Coagulating Basin.—The coagulating basin will be near the filter house as shown in Fig. 2. It will be rectangular in plan with inside dimensions 60x140 ft., with concrete walls 22 ft. 6 ins. in depth. The capacity will be approximately 1,200,000 gals. when filled to within 1 ft. of the top of the walls. The basin will be 6 ins. thick. The inside of the walls and bottom will be plastered with a ½ in. coating of 1:2 Portland cement mortar, placed while the concrete is fresh and moist. The baffle walls will be 9 ins. thick. These walls are three in number and are 40 ft. apart. They extend entirely across the basin. The last one is 20 ft. from the basin's outlet. Water is admitted to the basin at the inlet chamber which lies inside the basin at one corner and is 10x40 ft. in plan. A float valve in this chamber regulates the flow into the basin. The long side wall of the chamber extends to the flow line of the basin while the end wall is 4 ft. less in height. The first baffle wall has two openings, one a rectangular notch 3x20 ft. at the upper right hand corner and the other a semi circle of 2 ft. radius at the lower left hand corner. These openings are reversed in the second baffle wall. In the third wall the semi-circular opening is like that in the first wall and is similarly placed. In addition to this opening there are two rectangular notches in the top of the wall each 3 ft. deep, below the flow line, the one on the right hand to the direction of flow being 20 ft. and that on the left hand being 10 ft. wide. Inlet and outlet pipes are 30 ins. in diameter. A 12 in. cast iron sewer line drains the basin.

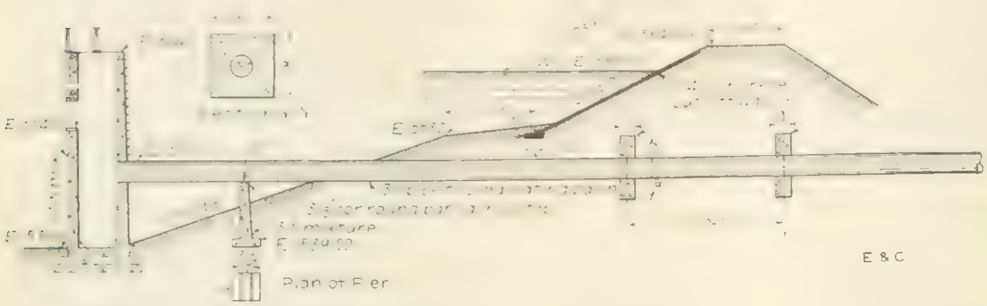

Fig. 3. Cross Section of Sedimentation Reservoir Through Gate Chamber and Outlet Pipe, Columbus, Ga.

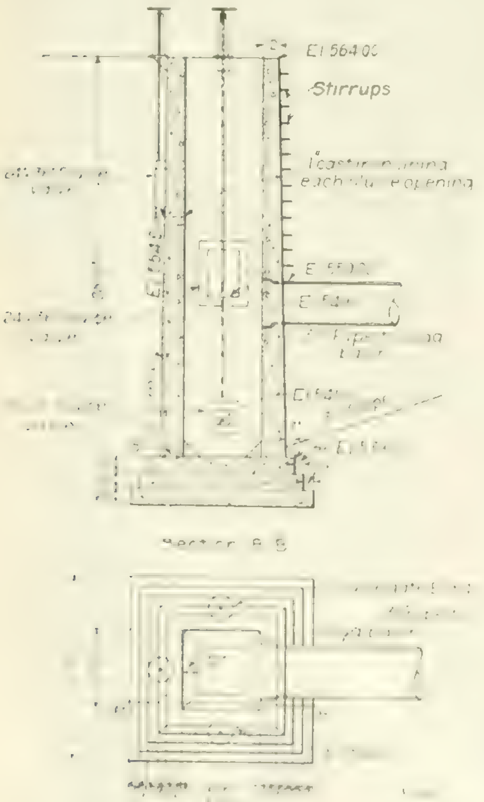

Fig. 4. Plan and Sectional Elevation of Gate Chamber, Columbus, Ga.

Filtration Plant.—The filtration plant will be of the rapid gravity type installed complete in the filter house. It will be of modern standard design and construction, and each unit will have a guaranteed net capacity per 24 hours of not less than 1,000,000 gals., guaranteed to deliver the water from the sedimentation reservoir, after passing through the coagulating basin, to the filtered water reservoir uniformly clear, free from color and turbidity, free from matter in suspension, and with a high standard of chemical and bacteriological hygienic purity, at the above rate; the bacteriological efficiency is specified to be not less than the removal of 97 per cent of all bacteria when exceeding 3,000 per c.c. in the raw water; bacteria in filtered water taken direct from the filter shall not exceed 100 per c.c. when the bacteria in the raw water number less than 3,000 per c.c.

The walls of the filters are to be a part of the filter house and built monolithic therewith. Of the six filter units provided, four are to be equipped for service with proper blanks placed in the walls of the remaining two units for equipment in the future. The filter boxes are rectangular in plan 16x24 ft. with a total depth of 7½ ft. The equipment is to be complete and in accordance with the best modern practice.

The filter beds will be 40 ins. total depth; 34 ins. will be carefully graded quartz sand with an effective size of 0.35 mm. and uniformity coefficient of 1.5. The supporting gravel bed will be 6 ins. thick composed of rounded gravel of a size that will pass a ⅝-in. mesh and be retained on a 3-16-in. mesh sieve.

The manifold and strainer system will be in accord with standard practice to provide durability and efficiency. It will be designed to pass wash water at 5 times the normal rate of filtration when delivered to the strainer system at 60 ft. head. The air wash system will be made with ½ in. brass pipe spaced 4 ins. on centers with 3-64 in. holes drilled 2½ ins. on centers on the under side of the pipe. The air supply will be sufficient to give 3 cu. ft. of free air per minute per square foot of filter area when two filters are being washed at the same time. The wash trough will be placed all around each filter. These troughs will be of concrete and monolithic with the walls as shown in Fig. 5. There will be two alum and two lime tanks and one hypo tank, all of reinforced concrete. The coagulant tanks will be at such an elevation as will give reliable gravity feed to the inlet of the coagulating basin. Standard porcelain lined cast iron orifice boxes with standard fittings for adjusting and regulating the flow of the solution will be used. Each filter will be provided with rate controller, loss of head gage, sample pump, and operating table.

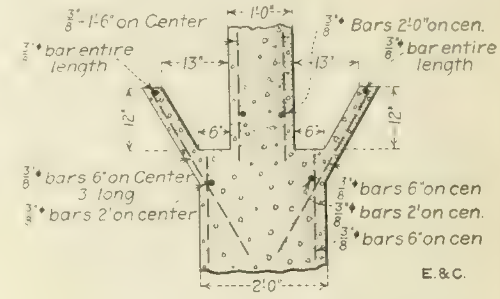

Fig. 5. Section of Filter Partition Wall Showing Type of Wash Water Gutters, Columbus, Ga.

The machinery room will be equipped with two suitable motor driven Root blowers, of not less than 2,300 cu. ft. per minute capacity each, and two motor driven 500-gal. per minute centrifugal pumps for taking water from the clear water basin and delivering to water tower, with all necessary piping and connections to clear water basin and to the riser pipe of tower and tank. The pipe to clear water basin and elevated tank will be 12 and 10 ins. respectively.

The laboratory will be equipped with a complete operating board and suitable apparatus for plating, incubating, examinations and analyses in connection with the proper operation of the plant.

Filtered Water Reservoir.—The filtered water reservoir will be 150 ft. in clear inside diameter with a total depth of 20 ft., holding 2,500,000 gals. when filled to a depth of 19 ft. This reservoir will be constructed of 1:3:5 concrete. This wall will have a total depth of 22 ft. 6 ins. with reinforcement as shown in Fig. 6. The bottom will be 6 ins. thick. The inside of this reservoir will be plastered as described for the coagulating basin.

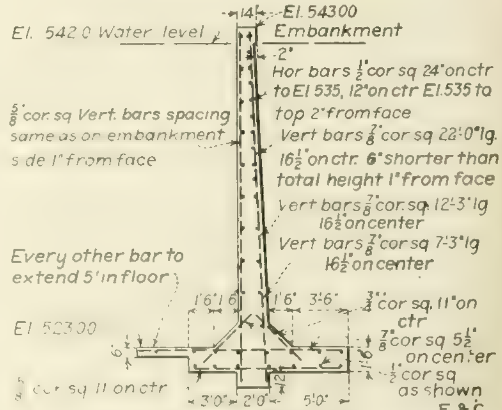

Fig. 6. Section of Wall of Proposed Circular Clear Water Reservoir, Columbus, Ga., Water Purification Plant.

The works here described were designed by Mr. J. L. Ludlow, consulting engineer, Winston-Salem, N. C., from whose plans and specifications this article has been prepared.

The new third lock of the St. Mary's Falls canal system, said to be the longest lock in the world, was formally opened to navigation, Oct. 21. The lock is 1,250 ft. long, 80 ft. wide and 23 ft. deep. Its construction was begun in 1908, and the cost is given as $6,250,000.

Data and Discussion on Allowable Leakage from Cast Iron Water Mains.

In recent extensive improvements to the water works distribution system of Akron, Ohio, special attention was given to the curtailment of joint leakage and to the measurement of such leakage as resulted. The consulting engineers, Messrs. F. A. Barbour of Boston and E. G. Bradbury of Columbus, Ohio, specified a maximum permissible leakage of 200 gals. per mile per day per inch of diameter of pipe. This allowable leakage is about 1.6 gals. per lineal ft. of lead joint. The pipe mentioned consists of new supply lines, lines parallel to and reinforcing old mains of inadequate capacity, and ordinary pipe line extensions into previously unsupplied sections of the city. The new supply and reinforcing lines were laid by contract; the extensions were put in by water department employes. The contract work was supervised by the designing engineers with Mr. E. A. Kemmler in charge; the department work was performed under the supervision of Mr. H. H. Frost, superintendent of the Akron water works. All pipe was purchased under specifications and was properly inspected at the foundry. In the case under consideration all lead joints had a depth of 2¼ ins. The present article, based on information given in a paper before the New England Water Works Association by Mr. Bradbury, and published in full in the Journal of the Association for September, describes construction and test methods and gives the results of the leakage tests. It is not the purpose of the paper to discuss at length the miscellaneous losses of water from existing pipe systems, but rather to consider the reasonable standard of workmanship in the laying of new pipe.

TABLE I. — PROPORTION OF MAIN JOINT LEAKAGE DETECTED AT WASHINGTON, D. C., COMPARED WITH TOTAL UNDERGROUND LEAKAGE.

Year.	Total underground leakage, gal. per day.	From joints in mains, gal. per day.	Per cent.
1908	4,243,900	1,013,900	24
1909	6,657,635	1,345,620	20.2
1910	6,364.000	1,034,000	16.2
1911	6,921,916	2,562,461	37.1
1912	5,115,320	746,305	14.6
1913	4,196,070	962,310	23
Average			23

Numerous investigations conducted within the past few years have shown enormous losses by leakage and carelessness. The fact that willful waste and defective plumbing are responsible for the major part of such losses, and are, moreover, more easily located and stopped, has resulted in greater attention being paid to them than to the leakage occurring underground, although considerable work has been done in the way of discovering and correcting large breaks.

It is well known that the water unaccounted for in the average system ranges from 20 to 60 per cent, and published records of measured leakage vary from 2,000 to 200,000 gals. per mile of pipe daily. In many of the water waste survey reports, underground leakage and the losses through unmetered services and sprinkler systems are combined as a single item. The Bureau of Economy and Efficiency in Milwaukee finds that about 23 per cent of the total pumpage is lost in this manner. The New York reports refer to joint leakage as "relatively small," while showing that underground losses of various kinds reach a very large figure. Mr. Phillips places the underground leakage and willful waste at 57 per cent of the total amount of water unaccounted for in Chicago. The city of Washington presents the only evidence in the hands of the writer indicative of the relative proportion of main joint leakage, the reports of the engineer department of the District of Columbia containing figures showing that such leakage averages approximately 23 per cent of the entire amount of water lost beneath the ground surface. These data are shown in Table I.

The records of fully metered cities and the results of water waste surveys prove that, although possible, it is the rare exception when underground leakage is kept down to a low figure. Very few cities keep such loss below 3,000 to 5,000 gals. per mile daily, and many very greatly exceed these figures. Mr. Kuichling, in a paper before the American Society of Civil Engineers (Transactions, Vol. 38), set the familiar standard of one drop per second from each joint, five drops from each hydrant or stop valve, and three drops from each service pipe as a fair average in a well-constructed system, resulting in a total of 2,500 to 3,000 gals. per mile daily.

Main joint leakage is so distributed, except in case of an occasional blow-out, as to make repairs more expensive than can be justified by the saving accomplished, and yet may in the aggregate result in serious losses. This fact imposes on the builder of water lines the duty of exercising every reasonable precaution to insure the tightest possible work. Pipe systems necessarily deteriorate more or less from year to year, but carelessly made joints must be more likely to loosen than thorough and painstaking work, and the ratio of loss between tight and slovenly work will at least not decrease with age.

Statements of leakage in terms of percentage of total water pumped, or gallons per capita, mean very little. The only rational unit is one which takes into consideration length and size of pipe; the writer prefers the number of gallons daily per mile for each inch of diameter, expressed as "gallons daily per inch-mile." "Gallons daily per foot of lead joint" is an equally rational unit—in fact, slightly more so—but is less convenient, especially if used with exactness. The length of lead joint per inch-mile is about 120 ft.

The practice of applying a test of some character to newly laid pipe is probably general at the present time. This test should preferably be made in the open trench, examining all joints under pressure before covering, but if the pipe is necessarily covered before testing, a maximum allowable leakage per inch-mile should be specified and rigidly enforced.

By the method of open trench inspection it is possible to make a new system practically bottle tight. The measured leakage in 5.5 miles of 6 to 12-in. pipe laid and thus tested by the writer at Grandview Heights, a suburb of Columbus, Ohio, amounted to 2.3 gals. per mile daily, or 0.31 gal. per inch mile. This system is supplied through a meter from the city of Columbus, and every service is metered. The actual amount of water entering the mains in the 2.5 years since its installation was 2,104,000 cu. ft., according to bills rendered. The quantity sold through individual meters for the same period was 1,998,600 cu. ft., and about 30,000 cu. ft. was used in flushing sewer trenches. In addition to this, contractors for several macadam streets were permitted to use water without measurement. Even without making any allowance for this last item, nor for the fact that nearly a mile of extensions has been laid during the period, it will be seen that the water unaccounted for amounts to less than 20 gals. per inch-mile daily.

In the opinion of the writer, the inconvenience to which the public is necessarily subjected by this method of testing is well justified by the certainty of results and the perpetual saving accomplished, except in the busy streets of the downtown district in cities of considerable size. Where the demands of traffic are such that it is absolutely impossible to hold the trenches open, the specified leakage test must be substituted.

The method used in all tests at Akron was as follows: After completion of laying, the pipe was filled with water and usually allowed to stand for about 24 hours, to permit a yarn in the joints to become saturated. The pipe havig been tapped for a ¾-in. connection, a small handpump was connected by wrought iron pipe and fittings, on which a gage was set, the suction being placed in a barrel of water. All gates were then closed and a hydrant valve opened to determine whether gate leakage existed in amount sufficient to flow. The depth of water in the barrel and its diameter at water level were then measured, and pumping begun. If the pressure was readily raised to the required amount, the time was noted and the amount of water required to hold the gage stationary for a period of from 10 to 30 minutes carefully observed. The leakage was computed from the quantity so used. If too great difficulty was found in raising the pressure, or if the leakage was found to exceed the allowed quantity, pumping was stopped and the gage observed to see if it would remain stationary after dropping to city pressure, thus indicating gate leakage; if such leakage was not demonstrated, effort was made to find defects in the pipe. In one or two instances pipe which could not be pumped up to pressure on the first trial were successfully tested on the following day, no satisfactory explanation being found. In a considerable number of cases the contractor was required to locate leaks and recalk joints, sometimes causing the reopening of the trench for considerable distances. In two cases cracked pipe was located and removed.

The final results of the tests have been very gratifying. One hundred tests have been of pipe laid by contract, of which 86 are included in the summary given in Table II. Of the remaining 14, there were 8 in which all joints were visible and no leakage existed, but on account of loss through gates no measurement was made; 3 in which the measured loss somewhat exceeded the specified maximum, but evidence of gate leakage was such as to satisfy the engineers that the specifications were complied with; and 3 in which the leakage was above the specified amount and the work not accepted. Of 40 tests made by the superintendent of water works, of pipe laid by his department employes, 38 have been tabulated, the remaining 2 exceeding the permissible loss; these are to be dug up if necessary, to locate and repair the leaks.

TABLE II. — LEAKAGE TESTS ON NEWLY LAID CAST IRON WATER MAINS AT AKRON, O.

86 Tests of Pipe Laid by Contract.*

Size, ins.	Length, ft.	Leakage in gallons daily per inch-mile.	Number of tests.
4	717	23	1
6	31,066	66	34
8	6,882	42	6
10	5,123	81	5
12	9,704	102	8
16	8,792	135	11
20	8,389	69	10
24	6,358	69	3
30	11,445	82	8
	88,476 16.76 miles	83.4	86

38 Tests of Pipe Laid by Water Department Employees.

Size, ins.	Length, ft.	Leakage in gallons daily per inch-mile.	Number of tests.
6	37,524	59	32
8	6,509	63	5
10	2,850	133	1
	46,883 8.9 miles	61.7	38

*Pressure from 66 to 152 lbs.; 35 lbs. greater than static head when improvements are complete.

It is to be noted that all the above figures are probably high, as they include whatever leakage may have occurred through gates as well as actual loss. Such leakage was known to exist in many cases, and in 20 of the tests of pipe laid by contract and included in the above summary every joint was visible and tight, although usually some water was required to keep up the pressure. The amount pumped also covers any contraction or absorption of air contained in the pipe. The presence of any considerable amount of air makes the tests unsatisfactory, and occasionally it becomes necessary to tap the pipe to release it.

Check tests made several months after the original test in two districts of about one mile each, of 24 and 30-in. pipe, verified closely the previous work, showing losses of 55 and 79 gals. per inch-mile respectively.

Assuming the average diameter of pipe in a complete system to be 9 ins., each 100 gals. daily per inch-mile saved or lost has a value of $3.28 per year for each mile of pipe at a production cost of $10 per 1,000,000 gals., or, capitalized at 7 per cent, is worth $46.86. Applying these figures to a concrete example, a city of 100,000 population with a production cost of $25 per 1,000,000 gals., and 160 miles of [illegible] per year in a saving of 400 gals. per inch-mile; and, figuring interest and sinking fund at 7 per cent, could afford to have spent $75,000, or about $470 per mile, more on construction to accomplish this result, and the same amount lost per inch-mile would cost a city of 200,000 population with 300 miles of mains and a production cost of $50 per 1,000,000 gals., $19,700 per year—equal to an investment of $281,430.

The expense of testing in the manner described is not heavy, averaging about $50 per mile. Occasionally where careless work has been done and the pipe covered, the contractor foots a rather heavy bill of expense; but the knowledge that the work is to be tested discourages carelessness, and most of the tests are made quickly and without trouble.

The writer believes that the results at Akron are as good as can usually be obtained under practical conditions. Many years ago Mr. C. F. Loweth stated before the American Society of Civil Engineers that he had satisfied himself of the possibility of laying water pipe with a leakage not exceeding 60 to 80 gals. per inch mile daily. This is borne out by the data given here. It is apparent that to go farther than this in a specification is not safe, on account of gate leakage and other internal conditions.

In view of all the above facts, the writer proposes as a standard for allowable leakage in new cast iron water pipe an average of 100 gals. daily per mile per inch of diameter for each complete contract or district with a maximum limit of 200 gals. daily per mile per inch of diameter not to be exceeded in any single test. The recommendation is made, however, that open-trench testing, with all joints visible, be specified, covering being done only by special order, and the allowable leakage clause used only when it is entirely impracticable to keep the ground open.

Use of Magnetic Dip Needle for Locating Lost Shut-off and Valve Boxes.

In new water systems, in small but growing towns, there is frequent necessity of laying pipes in streets and private ways where the grade is not established, the boundary lines are crude or imperfect, and where later there may be considerable change as improvements are made and sidewalks constructed. Houses are often built in advance of such improvements, water supply is furnished, and the records of stop-box locations are not wholly satisfactory or the ties of gate boxes reliable. When a sidewalk is made, the grade is often raised a few inches, or low spots are leveled up, and unless the waterworks man is at hand, or notified, the service box is subject to being buried, particularly if it were not in sight at the beginning of such work. Later, when it may become necessary to use such a box, it is of great assistance to be able to locate it exactly, particularly if the ground is frozen. This is the function of the dip compass, or "detector," which can be used with most satisfactory success when the box is covered not over 6 or 7 ins., which will include most all cases. The notes here given on the use of the dip compass in finding hidden service and gate boxes are from the paper on this subject by Mr. Edward D. Eldredge, superintendent of the Onset Water Co. of Onset, Mass., before the New England Water Works Association. Additional notes are included from the discussion of the paper as published in the Journal of the Association for September.

The dip compass consists of a magnetized needle or pointer, mounted on a horizontal axis and free to revolve in a vertical plane, normally that of the magnetic meridian. The needle is accurately balanced for the latitude of the locality in which it is to be used, so it will assume a horizontal position when held in a north and south line, and subject only to the earth's magnetic attraction.

A service or gate box is a sufficiently large mass to offer a strong attraction to the needle, particularly as they are set in a vertical position, which is the most favorable. If the detector is held as described, with the needle at rest on the north and south line, and moved carefully, to avoid oscillation, over the ground and as close as possible to it without touching, when within 6 or 7 ins. in any direction from the hidden stop box, the needle will begin to show a deflection or dip, and by following it up to the point of greatest dip, the exact location of the box is discovered. As magnetism, unlike electricity, is not insulated by any substance, the attraction is ever present through air, frozen earth, cement, stone, tar, or anything likely to be encountered.

In a case where a cement sidewalk 5 or 6 ins. thick was laid over a stop box, the box was located exactly, although the deflection is not so decided and quickly recognized in such a case as when a lesser covering exists. The same process, of course, applies to the ordinary iron gate boxes in street mains, which, in some of our unfinished streets, are often covered 3 or 4 ins.

DISCUSSION.

In the discussion Mr. L. M. Bancroft told of locating, by means of the dipping needle, a service box under a tar concrete sidewalk. It was 13 ins. from the top of the walk to the box. Mr. J. M. Diven told of locating a valve box covered to a depth of 14 ins., and also of locating a curb box through a concrete sidewalk. Mr. Diven said he had formerly had some trouble with the demagnetization of the needle. This was caused by hanging the instrument on a gas pipe where the constant contact with iron sapped the needle's magnetism. Since discovery of the cause of the trouble the compass has been kept in a wooden drawer, protected from the influence of all iron, and no further trouble has been experienced.

Mr. C. D. Sharpe advocated the use of a pipe locating apparatus in conjunction with the magnetic dipping needle. The former gives the line of the pipe and the latter locates the buried box readily without first locating buried horseshoes, etc.

Method and Cost of Constructing Shaft and Tunnel at Lake View Pumping Station, Chicago Water Works.

The present article gives the methods and cost of constructing by day labor a shaft, at the Lake View Pumping Station of the Chicago water works, and the tunnel connection between this shaft and the proposed Wilson Avenue supply tunnel which will be described in a later article. The connecting tunnel is 8 ft. in diameter and it is a spur from the proposed Wilson Avenue tunnel to the north wet well in the station. The tunnel is 1,381 ft. long and is lined with three rings of brick. The information here given is based upon the latest annual report of Mr. Henry W. Clausen, engineer of water works construction.

THE SHAFT.

Work was begun on April 4, 1913, when the excavation for the shaft was started, by breaking out an opening 16 ft. square in the concrete floor of the old coal room of the Lake View Pumping Station, now torn down. Sheeting, made up from 2x10-in. and 2x8-in. yellow pine, in 12-ft. lengths, was driven around the four sides and held in place by 3 sets of 6x10-in. braces 16 ft. square. Work on the square excavation for the shaft was completed by April 10. Elevation bottom of hole, —1.6. At this depth, or 12 ft. below the surface, water was encountered and the work was discontinued until the steel shell could be placed.

The steel shell is a hollow cylinder of ⅜-in. boiler plate, 11 ft. 6 ins. in diameter and 30 ft. long with both ends open. It was made in five 6-ft. sections. The two bottom sections were riveted together in the shop and provided wih a beveled cutting edge stiffened around the inside with an additional plate and a 6x8-in. angle 1 ft. above the cutting edge. The total weight of the shell was 21,385 lbs. Each section of the shell was thoroughly caulked, and as the shell was made with flush joints and countersunk head rivets on the outside the resistance to sinking it was reduced to a minimum.

The two bottom sections were first placed in the square excavation above described and lined up by means of vertical guide timbers on four sides. The third section was then added and the 8½-in. brick lining started, the 6x8-in. angle serving as a footing. Preparations were then made to sink the 18 ft. of erected shell deep enough to permit the erection and riveting of the remaining two sections.

A No. 4 Nye pump was suspended within the shell as the material first to be excavated was the water-bearing stratum of lake sand. This sand was 11 ft. deep and overlay the soft blue clay, through which the balance of the shaft was sunk. After excavating the sand from within the shell, it finally settled sufficiently so that the remaining two sections could be added. The 8½-in. brick lining was then carried up to elevation —0.66 or 4 ft. 8 ins. below the top of the fifth section. The excavation within the shaft was then resumed, no additional weight to the shell being necessary other than the brick lining to sink it to the soft clay stratum encountered at elevation —12.40.

At this step it became necessary to cover the top of the shell with a 10x10-in. timber platform on which pig iron was loaded, leaving a square boxed opening in the center for the hoisting bucket. In all, 198,900 lbs. of iron was put on before excavation was resumed. With this weight the shell was gradually forced to its final depth, the cutting edge being at elevation of —26.0. It had then penetrated 13.6 ft. into the soft blue clay which had closed around the shell and effectually shut off the water. The weight on the shell was then removed and excavating for the shaft carried 5 ft. below the cutting edge or to the top of the tunnel bore and lined with a 13-in. brick wall. The brick lining in this one section was suspended by eight 1-in. hanging rods having plate washers. Each rod had an eye at the lower end which supported the 8x12-in. washers and provided for hooking on additional rods below if they were needed. In the 6x8-in. angle which supported the brick lining for the steel shell, short eye bolts were placed before any brick work was started and from these were hung the above set of rods. The shaft was then carried down to its full depth and the two tunnel eyes excavated and lined 10 ft. each way from the shaft.

The total cost of the shaft before the gate and fish screens were installed was as follows:

Item.	Cost.
28,127 sewer brick at $8	$ 225.00
72 bbls. Utica cement at 95 cts.	68.40
36 bbls. Portland cement at $1.60	57.60
3,500 ft. B. M. lumber at $27 per M	94.50
Steel shell	1,289.00
Hanging rods and ladder irons (material)	10.00
Labor and teams	1,918.85
Engineering and inspection	265.66
Rental of pig iron and freight	163.50
Total	$4,092.51

In addition to the above, 20 cu. yds. of beach sand, taken from the excavation, was used. The total depth of shaft below the surface is 54 ft., making the cost, including the turning of the two tunnel eyes, $75.80 per ft.

TUNNEL CONSTRUCTION.

After the shaft had been carried down to its full depth and the two tunnel eyes excavated and lined 10 ft. each way from the shaft, the balance of the work was confined entirely to the tunnel. The diameter of the excavation was made 10 ft. 2 ins., to provide room for the three rings of brick of the tunnel. Excavation was carried on mostly with clay knives, the bore throughout its entire length lying in a stratum of medium stiff blue clay.

The mortar used in the brick lining for both shaft and tunnel was mixed uniformly the same, a batch being made up as follows:

2 bags of Utica cement	4 cu. ft.
2 bags of Universal Portland cement	2 cu. ft.
3 wheelbarrows of beach sand	9 cu. ft.

or, in the proportion of 1 of cement to 1½ of sand.

An 8-in. brick bulkhead was built at the Wilson avenue end and another one 5 ft. thick about 60 ft. farther back, having a 3-in. wrought iron drain pipe through it near the bottom with a valve on its north end left closed.

The tearing up of the track and floor, removal of air pipe and sweeping out of tunnel with steel brooms was then resumed and completed by Aug. 27. The total cost of work done during 1913 on the construction of the tunnel itself was $30,558. This divided by the actual number of lineal ft. completed (1,336) makes the cost per foot nearly $23.

The construction work on the Wilson avenue tunnel connection was in local charge of Mr. E. P. Scott, assistant engineer.

Should Meters Register in Gallons or Cubic Feet?—The question as to whether water meters should register in gallons or cubic feet is still a live one, as indicated by the comments thereon by the Committee on Meter Rates of the New England Water Works Association. We quote from the report as follows:

It is most unfortunate that two units of measurement are in common use. It would be much better if one could be discarded. More than three-fourths of the meters now in service record the quantity in cubic feet. According to replies to the committee's inquiries, schedules of water rates are about equally divided between cubic feet and gallons. The Coffin committee on Meter Rates recommended the use of cubic feet in its report submitted in 1905. The number of meters now being sold to register in gallons is increasing much faster than the total number sold. This indicates a gain in popularity of the gallon. There is much to be said in favor of each of these units. Replies to the circular letter of the committee indicated very evenly divided preference. In view of this condition, and in view of the further fact that members of the committee have different preferences, it is not thought best to attempt to recommend at this time that either unit shall be used to the exclusion of the other. The committee recommends further general discussion of this subject, and suggests after such discussions that a vote might be taken to show the preference of the members.

It is absurd that two units should continue in general use. The advantages of adopting either one would be vastly greater than any possible advantage in either. It would seem that the Association could determine which it prefers and throw its influence toward its universal adoption. The committee feels that this should be done by action of the whole Association and not by a committee.

In the interest of economy of labor, it is obviously desirable that the same unit should be used in any system for both meter registers and schedules. Thus the use of meters reading in cubic feet, with a schedule of rates in gallons, is undesirable and should be avoided by changing as rapidly as is possible so that both are on the same basis.

Proper Installation of Service Pipes.—The following notes on the proper installation of service pipes are taken from the latest annual report of the Lincoln, Neb., Water Department, Mr. James Tyler, Water Commissioner: Just inside the basement wall of the dwelling, or building, into which the service pipe extends, a stop and waste cock should be conveniently located, and arranged so that the water may be drawn back and all the pipes within the dwelling, or building, emptied through the stop and waste by opening the faucets at the highest points therein and allowing the air to enter the pipes. All of the pipes connected with the service inside the dwelling, or building, should be laid with an inclination toward the point in the cellar where the stop and waste is located, without any sags or pockets, so that the pipes may thoroughly empty themselves from water when the waste is opened. But in cases where such sags or traps are unavoidable an additional stop and waste should be put in. The service pipes in the dwelling or building should be located in the parts thereof best protected from frost, and should in no case be carried any considerable distance alongside walls, but should be carried immediately under the bottom of the basement or cellar to the center or least exposed point therein, previous to being carried upward into the inhabited parts of the dwelling or building. No water supply pipe should be laid in the same trench with a sewer. At Lincoln a record of the locations and sizes of all service pipes laid, with all necessary data, are kept so that in case the boxes are covered up the services can readily be located from the accurate measurements recorded.

Operating Costs of Ultra-Violet Sterilization Plants.—The following statement as to the operating costs of ultra-violet ray water sterilization plants is quoted from a paper before the American Water Works Association by Dr. Max von Recklinghausen of New York City:

Operating costs will vary with the size and the running hours of the plant, and the coefficient of safety for the ultra-violet ray treatment. According to the quality of the water I expect in large plants which run 24 hours that the current consumption will vary between 30 and 125 kw. hours per 1,000,000 gals., allowing for a large safety coefficient. The labor charges are negligible as the apparatus only needs an occasional cleaning and starting of lamps. Apart from this the lamps have to be repumped and repaired from time to time. When the water is of variable physical quality, one will have so to establish the plant that all the lamps will be running during the period of least transparent water and only some of them during the period of best transparency.

GENERAL

Some Design and Operation Features of Refuse Destructors.

For the sake of a clear definition, furnaces operated at a temperature of about 1,500° F. or less, will be called incinerators, and those which have a working temperature of over 1,500° will be called destructors. In America these furnaces are called low and high temperature incinerators, respectively. It seems preferable to know what is referred to by a distinctive name rather than by a qualifying term which is often omitted.

Fire is the oldest and best known method of disposing of offensive rubbish. It has been adopted by all races and in all ages, with more or less satisfaction. Burying offensive matter is another ancient and effective way of getting rid of it.

The first engineer who successfully designed a furnace to effectually destroy town refuse by fire was Mr. Alfred Fryer of Nottingham, England. This was in 1874. He built two such furnaces in Manchester in 1876 and these have been extended and improved and are still in use, which fact, at least suggests that Fryer had evolved a scheme based on right principles.

This innovation, however met with a hostile reception. The public prejudices against the adoption of such furnaces were so strong that it is surprising it survived.

Following Fryer, there were several aspirants who claimed to have designed furnaces superior to Fryer's, for example, Pickard's "Gommand," Wilkinson of Birmingham, Healy of Brighouse, Heard of Paddington, Burton of London, Stafford and Pearson and others, but Fryer's furnace held its position. These, and some which were designed later on, were of the incinerator class—low temperature, slow burning, producing soft clinker and obnoxious gases. Mr. Charles Jones (Ealing) in 1885 introduced his "fume cremator" which was an ample proof that something was needed to improve the combustion.

Mr. William Horsfall in 1887 brought out the first high temperature furnace, which improvement gave a new impetus to the process. Horsfall also improved the furnace by arranging for a front exhaust of gases, by which means all gases had to pass over the active grate before being discharged and by so doing the fume cremator was found to be unnecessary and was ultimately abandoned.

From 1887 to the present time many improvements have been introduced, such as steam jet blowers, forced air draft, regenerators or air preheaters, continuous grates, twin grates, mechanical feeders and clinkering, dust catchers, steam producers, and so on. To deal with each of these would entail a long paper, as each maker has devised certain features in connection with furnaces, etc., they will therefore be referred to collectively as far as possible.

Incinerators, then, are the survivors of the earliest types and destructors are the later developments.

Incinerators are furnaces capable only of slow combustion and require some form of drying-hearth or device for reducing the moisture contained in the refuse, preparatory to cremation. Some incinerators are charged with large quantities of refuse at one operation, the result being the lowering of the temperature of the furnace, the prolonging of the period of cremation as the combustion is more often local than general, the passage of green gases and the production of soft clinker and little steam. An analysis of the gases and a record of the maximum and minimum temperatures in incinerators will doubtless show that the work done is not altogether satisfactory.

Experience has shown that with destructors, the same as with other furnaces, the percentage of carbon dioxide, oxygen and carbon monoxide contained in the gases in the main flues, is in proportion to the completeness of the combustion of the fuel and the carefulness of firing and clinkering.

It is manifest that not even the best coal-fired furnace can produce satisfactory percentages of the above constituent gases of combustion unless the temperature of the furnace is uniformly maintained, the range of heat fluctuations kept as small as possible, the quantity of primary and secondary air supplied is controlled to satisfy as nearly as practicable the requirements of the fuel used, and that the fuel is charged in small and regular quantities. Although coal is not usually quite constant in its calorific quality, it is vastly more so than town refuse, and if care is needed in the case of coal-fired furnaces, it is reasonable to advocate somewhat similar care in the cremation of refuse, if scientific and sanitary results are to be achieved.

Many incinerators require coal or other fuel to assist in the combustion of refuse, although the proportion of combustible matter in such refuse approximates that contained in refuse in other places where with destructors no other fuel is required or used, and good clinker and power are produced.

Engineers have during the last century developed the design of coal furnaces for steam raising and countless other purposes. Different grades of coal require different types of grates, etc. It would not be expected that a grate designed to burn steam coal with the maximum of efficiency would produce good results if bark was consumed.

A prudent manufacturer who requires a furnace for any ordinary industrial purpose would ordinarily adopt one that has already proven efficient, and if for any special purpose which needed some experimentation, he would start where others had left off (unless he had good reasons for doing otherwise), and so make use [illegible]

Bearing in mind what has been achieved in many parts of the world with destructors, it would seem inadvisable to erect incinerators.

The more municipal engineers investigate this important problem, the more readily they will appreciate the necessity for the fullest consideration of the capabilities of various types of furnaces which it is contemplated to install.

There are several makers of destructors and incinerators on the market. Each maker claims to have some features of superiority over others and doubtless some do possess certain advantageous qualities.

The original furnaces were top-fed and some are now being built on this plan, but in course of the last ten years or so back feed and front feed arrangements are found to answer rather better. Top feeding allows of mechanical labor-saving devices to be used, but even then it entails more arduous labor to the men below in raking in the material by long rakes over the fires. Back feeding was introduced in 1891 and is claimed to be under better control while front feeding means a better concentration of labor. Conditions will vary in different places and these must be taken into account when deciding the method of feeding.

Destructor furnaces require a strong draft in addition to the normal pull of the chimney, although as one writer stated that since it has been demonstrated that incandescence can be maintained apart from the use of fans and regenerators, the question to be considered is whether it is good practice to run machines which absorb up to 15 per cent of the total steam produced. The steam jet is a simple, effective and economical blower, and the fan, being entirely mechanical, is more subject to wear and tear and liable to breakdowns and therefore should be in duplicate. The furnace pressure should be in balance, that is, the force of the blast under the furnace should be slightly in excess of the draw of the chimney, so as to render the intake of air at the doors during the time charging or clinkering takes place, as nearly nil as possible. The aim, nevertheless, of the furnacemen is to have these doors opened as few times as possible.

The advantage of either forms of blast does not appear to be very pronounced and to provide for all contingencies, the makers now usually include both in recent installations.

Preheating of the primary air is claimed to be of considerable value. The air after being preheated absorbs the moisture in the refuse most readily, and this is found to be done sufficiently quickly that refuse can be cremated without the intervention of a drying-hearth. In ordinary steam boiler practice, preheated air is found to be of such value, for it increases the amount of water evaporated per hour. Brislee gives an example of the advantage of preheated air. Two boilers were tested with the following results:

	[illegible]		[illegible]	
	Cold	Hot	Cold	Hot
	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]				
[illegible]				
[illegible]	[illegible]		[illegible]	[illegible]

The increased evaporation was due to the extra heat brought into the furnace and the increased rate of combustion of fuel due to it.

In the case of the boiler No. 1 the evaporation was increased 79 per cent and the No. 2 55.4 per cent.

Brislee points out that if air is heated before combustion is allowed to take place, then the heat in the air is added to the heat of combustion, and the quantity of heat available for raising the temperature of the products is therefore greater. In other words conditions being equal in other respects, the preheating of air means economy of coal and the increased output of the boiler.

In destructors, regenerators, however, do not constitute the only method of preheating of air. Regenerators consist of stacks of pipes, through which the hot gases pass out on their way to the chimney and around which the cold air passes into the furnaces. The sensible heat of the hot gases is partially imparted to the cold air and in this way the temperature of the air is increased from 200° to 300° or often more. This, of course, means the conservation of heat which would otherwise be lost in the chimney. Such regenerators offer some obstruction to the outgoing gases and to the incoming air, and to overcome this it is necessary to absorb some energy in driving fans. About 5 to 10 per cent of the total steam produced from the refuse is thus utilized. Regenerators have in some instances been taken out for different reasons. It is easy to arrange for the primary air to be heated by passing it through flues built in the walls of the furnaces. This is being done in gasworks practice and has been found to be very effective. Indeed such method of preheating air or an adaptation of the idea is provided in some destructors. Incidentally, the air required for the furnaces is taken from the vicinity of the refuse and by this means the building is ventilated.

Continuous grates in one chamber are supplanting individual cells and grates and the advantage is evident. In the case of individual or single cells when refuse is charged into the furnace, it has the tendency of lowering the temperature of that cell, and the effectual mixture and burning of the gases must take place in the combustion chamber. Whereas in the case of the continuous grates, two or more grates are built side by side in one chamber with independent ash pits beneath. When any refuse is charged onto any one of these grates, the cooled gases therefrom are mingled and burnt in the furnace itself and no green gases can escape into the combustion chamber without first passing over an incandescent body. Meldrum's Simplex Destructor was the first constructed on this principle and its usefulness was appreciated by other makers. Heenan and Froude, Horsfall, Fryer, Dawson and Manfield and other destructor builders now design some form of continuous, twin or series grates.

Mechanical stoking and clinkering and other forms of labor saving devices are installed in a number of destructors. Boulnois and Brodie's charging trucks, Marten's charging apparatus, Horsfall's tub feed, Heenan and Froude's hydraulic feeder and others are examples of the methods most in use. Heenan & Froude's trough grate and hydraulic ram clinkering machine and Sterling's clinkering grate are installed in many plants. According to Mr. Fetherston's report the arduous work of charging and clinkering at the Clifton (New York) destructor has been reduced by the use of the hydraulic ram charger and clinkerer and trough grate. The following figures are extracted from his report:

COMPARISON OF WORK ELEMENTS—OFFICIAL TESTS.

Plant (both in New York).	Cost per ton, supervision and labor.	Pounds burned per furnace man per hour.	Pounds burned per sq. ft. per hour.	Per cent of time furnace door open.
West New Brighton, 1908	$0.76	1,357	54.3	73.7
Clifton, 1913	0.41	3,330	144.2	5.1

Mr. Fetherston, however, expresses his opinion that some time must elapse before the complete economy of the mechanical devices is demonstrated.

Conveyors have been tried in several installations, but owing to the heterogeneous character of the refuse, they have not always been found to answer. The same remark applies to refuse elevators, etc. The three buckets and rake elevators in use at the Hackney destructor (London, England) capable of raising 10 to 12 tons per hour, are found to be very expensive to maintain and are liable to serious breakdowns.

Hoppers or storage bins above the destructor furnaces are often bound to be unsatisfactory owing to the tendency of the refuse to bind and arch over the opening, and the bulky nature of the refuse often renders the hoppers inadequate in capacity, in which case some city authorities store the refuse in the carts or wagons and thus obviate creating a nuisance. In some cases the refuse in bins tends to ignite and cause an emission of noxious gases. In recently built destructors the hoppers are located behind or in front of the furnaces and are sufficiently large to hold one or more days' supply. Such a position is both cool and handy for hand firing.

In the more recent developments of destructors the authorities have observed that it was possible to derive considerable steam power. The ordinary steam producing capacity of destructors is calculated at one pound of steam for each pound of refuse burned and as one horsepower may be based on 30 lbs. of steam per hour, it will be seen that one ton (2,000 lbs.) will on the above basis produce 67 HP., but a large quantity of steam is required for the plant itself so that the net quantity of steam available for other purposes is less.

There are plenty of instances where the production of steam has exceeded one pound per pound of refuse.

Official tests made at the Clifton destructor in 1913 with a winter mixture of refuse gave a gross equivalent evaporation from and at 212° of 1.00 to 1.11 lbs. at a pressure ranging from 117 to 126 lbs. per square inch, but at the West New Brighton destructor the results were 1.10 to 1.41 lbs. of steam at a pressure of 130 to 137 lbs. per square inch. The production of steam during the year 1911 was 1.23 lbs. per pound of refuse.

In Milwaukee the evaporation was 1.34 to 1.45 lbs. and at Westmount, P. Q., from 1.48 to 2.11 lbs. In Calgary, Alta, the average evaporation at tests was 1.13 lbs. In Darwen (England) the average evaporation during the year was 1.23 lbs., while on a test when burning unscreened refuse and slaughter house refuse was 1.55 lbs. of steam from and at 212° F. In Huddersfield (England) with one part of sewage sludge and two parts of refuse 1.4 lbs. of steam were obtained.

A test was made in Rochdale, England, for the purpose of comparison. By using ordinary coal slack having a calorific value of about 12,500 B. T. U., 7.33 lbs. of steam were produced from and at 212° F., as compared with 1.97 lbs. from refuse. At Montgomery (Ala.) in 1911 the following test results were secured: 1.37 lbs. of steam per pound of refuse; carbon di-oxide in the waste gases, 11 per cent; temperature in combustion chamber, 1,920° F. The refuse consisted of 25 per cent ashes, 42 per cent garbage, 13 per cent rubbish, and 20 per cent manure.

The steam produced by burning refuse is utilized for a variety of purposes, such as pumping sewage, or water, generating electricity, etc.

Part of the steam generated from refuse in London, Ont., is used for heating the Victoria Hospital, which is situated 50 ft. away. In one city 72,000,000 ft.-lbs. of energy is derived from every ton of refuse and used for pumping sewage. In Liverpool and Rotterdam, for example, the electric energy generated is used for street railway operation. In West New Brighton and Clifton plants already referred to, a large quantity of steam is not utilized owing to the fact that the New York City charter prohibits its sale and consequently no revenue can be derived in this manner, this represents a loss of about $7,500 per annum. The quantity of refuse burned at each of these places is about 9,500 tons per year. In Westmount, P. Q., the destructor is an auxiliary enterprise operated in the same building as the municipal electric lighting plant, the steam generated by the cremation of the refuse being used in the production of electrical energy. Messrs. Hallock and Runyon, the engineers who were appointed to report on the disposal of refuse, by the City of Newark, stated that the total revenue in 1910 for electric lighting and destructor plants was $102,149.17, and the total cost of operation of the combined plant was $75,426.38, leaving a net profit of $26,722.79. The operating costs include all capital charges and depreciation. The cost to the health department for the refuse destroyed was $9,449.06 for the year. The population of Westmount is about 16,000.

One incidental result of good steam production is the production of good clinker, which can be used for many purposes. Low temperature cremation results in soft clinker which is not only useless but objectionable, as it cannot be used and it often contains partially consumed organic matter. Clinker should be well fused and vitrified and this can only be produced by maintaining a uniformly high temperature in the furnace.

Good clinker is used for making pavement slabs, for sewage filtering media, brickmaking, crushed into sand and mixed with lime for mortar, etc.

Reference has already been made to steam production. In the earlier plants, the boilers were placed in the furnace directly over the fire, but it was found that the cooling effects of such boiler militated against the successful cremation of the refuse. The next step was to place the boiler between the cells and although better results were obtained, the makers i later installation have located the boilers beyond the combustion chamber. By this means the maximum temperature is secured and the gases adequately combusted before coming into contact with any cooling surfaces. The figures quoted point to the possibility of developing considerable power by the high temperature cremation of refuse. If due attention is paid to the fundamental requisites of a destructor the cost of operating the same can be materially reduced by the sale or utilization of the steam and hard vitrified clinker produced.

Some disappointment has in places been experienced owing to the makers' claims being exaggerated and impossible of realization. It would of course, be folly to decry every new device, arrangement or design. until it has been put to a practical test, for that would be tantamount to placing an embargo on all legitimate developments, but experiments are costly and occasionally disturbing as was recently found to be the case in a large plant in North America, whose designers received due publicity in engineering journals. The achievements that were going to be accomplished fell short and the works are now being improved.

The evolution of the destructor has been slow and expensive and the results of experience in all parts of the world has greatly assisted the makers in deciding upon the arrangements, capacity and construction best suited for the refuse produced in different places in different climes.

New York, Westmount, or other destructors in the East may not be quite suitable for Western refuse, and doubtless this is the case. Each city has its own problem to solve and it, therefore, behooves that the authority contemplating the installation of a destructor or incinerator should take the fullest possible advantage of the experience of others under similar conditions and of the plant best suited to satisfy its own specific needs.

ACKNOWLEDGMENT.

The foregoing matter is from a paper prepared for presentation before the Canadian Public Health Convention, at its scheduled annual meeting, by Mr. R. O. Wynne-Roberts, consulting engineer, Regina, Sask. Owing to unsettled conditions due to the European war the meeting was cancelled and a copy of the paper was sent to this journal for editorial use.

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Results Obtained at Abilene, Kansas, in 16 Months under Commission-Manager Government.

In order to prove the greater economy in transacting city business under the commission-manager form of municipal government as compared with results under the old form of government, Mr. Kenyon Riddle, city engineer-manager of Abilene, Kans., cited certain savings effected in that city, during the past 16 months, in a paper before the annual convention of the Kansas League of Municipalities. The figures cited are here given as are the general results obtained at Abilene during Mr. Riddle's handling of the city's business.

Sewer Work.—In sewer work 5,530 lin. ft. were laid at 40 cts. per ft. The former price was 60.3 cts., so the saving was $1,122.59. The cost of flush tanks installed has dropped from $66 to $51.80, giving a saving of $56.80 on the four installed. Manholes now average $23 as against the old average of $40.80. This gave a saving of $89 on five manholes. The total saving on sewer work was thus estimated to be $1,268.39.

Water Pipe Extension.—The present cost of installing 4 and 2-in. pipes are 10 and 6 cts. lower than the old costs. On 2,500 ft. of 4-in. and 4,200 ft. of 2-in. the saving has amounted to $502.

Street Cleaning.—The unit cost of street cleaning has dropped from 50 cts. per 1,000 sq. yds. to 34.2 cts. In cleaning 180,300 sq. yds. three times during the past 16 months the saving has been $876.95.

WASTE IN WATER WORKS DEPARTMENT.

Listed in Table I is certain work done in the water works department during the past 16 months, the period of time during which the commission-manager plan of government [illegible] at Abilene. The table compares the actual cost of work done in prospecting, connections, repairs, etc., under the existing conditions, to the necessary costs, had the past work of the water works department been under the control and supervision of one equipped to manage that class of work.

This same evidence of waste has shown up in many cases, all a result of no records and a lack of engineering knowledge.

Mr. Riddle also points out a few instances of waste in the street and alley department. These data are given in Table II. This waste is due to the fact that the work was carried on in times past by some ordinary laborer acting as street commissioner, under the supervision of the council.

To save enumeration, Group 1 in Table II was a singular case, there were three others almost identical to those sighted in Group 2, and four similar to those of Group 3. To sum up these blunders, the total waste was $2,056.75.

The intersections mentioned in Table II have been paved within the past two years, and with the necessary cross section notes, and existing evidence of the original earth grade the comparative earth work costs were derived. The grades, which were correctly established by reputable engineers in 1889, and which were totally ignored by the city, made it possible to estimate the cost of the improvements which should have been made, and to compare with the faulty works which existed.

Some of the things accomplished within the past 16 months follow:

A complete re-survey of the town, which included the locating of permanent monuments. A map showing the recorded measurements, together with actual measurements, and record of all monuments with their ties and guides to assist in future surveys.

A system of blanks and maps to aid in recording all new underground work and for obtaining records of existing underground work, not recorded in the past.

A schedule and log sheet for the dragging districts.

A diary or history of all construction work done under guarantee, to serve as evidence of doubtful material or workmanship.

Cost data sheets of all municipal work, to

TABLE I.--WATER WORKS EXPENDITURES NECESSITATED IN 16 MONTHS BY LACK OF RECORDS AT ABILENE, KANS.

Location.	Amount spent.	Necessary amount.	Remarks.	Waste.
Olive and First	$ 4.50	$0.00	Attempting to locate main under street to be paved Found in parking	$ 4.50
Kuney and Eleventh	40.00	4.00	An effort to locate cross and connect an extension	36.00
Buckeye and Third	26.00	2.00	To cut off old line under pavement	24.00
Spruce and Second	19.50	3.00	To connect an extension underneath pavement	16.50
Second street	9.50	2.00	Attempt to locate and cut off abandoned line under pavement	7.50
			Unsuccessful; may burst and cause damage. To $7.50 add further possible cost of prospecting and probable damage from leak.	
Buckeye avenue	12.00	0.00	Necessary excavation to examine line in street to be paved	12.00
			Total	$100.50

TABLE II.--WASTE IN STREET DEPARTMENT OF ABILENE DUE TO IGNORING OF ESTABLISHED GRADES IN PAST YEARS

Location.	Amount spent to bring street to established grade.		Necessary expense, had system of established grades been followed in past		[illegible] be removed.		Waste.
Third and Mulberry	Excavation	$ 70.00	Excavation	$31.50	Two concrete culverts	$250.00	
	Taking out culverts	21.00	Two concrete gutters	24.00	Unnecessary fill	44.00	
	Lowering manhole	5.00					
	Lowering sidewalks	24.00					
		$120.00		[illegible]		[illegible]	[illegible]
First and Buckeye	Excavation	$ 75.25	Excavation	[illegible]	[illegible]	$150.00	
	Disposing of culvert	12.00	One gutter	[illegible]	[illegible]	[illegible]	
	Lowering manhole	7.00					
	Lowering sidewalks	24.00					
		$118.25		[illegible]		[illegible]	$267.25
Sixth and Kuney	Excavation	$ 80.00	Excavation	[illegible]	[illegible]	[illegible]	
	Disposing of culvert	[illegible]	One gutter	12.00	Unnecessary fill	[illegible]	
	Lowering manhole	3.00					
	Lowering sidewalk	[illegible]					
		$104.00		[illegible]		$182.00	[illegible]
Cedar and Fifth	Excavation	[illegible]	Excavation	[illegible]	[illegible]	[illegible]	[illegible]
	Lowering manhole	[illegible]					
	Lowering sidewalks	24.00					
		$63.00		$11.00			$77.00
Cedar and Sixth.	Practically the same as above.						

be used as a guide in future work of a similar nature.

The laying out of the paved area into districts, in order to experiment on modes of clearing and to study results, so as to apply the most efficient methods to particular districts.

A thorough examination of the sewer system, to detect sluggish sewers, and to adjust the flow of water into flush tanks.

An over-hauling of the water pumping plant, and a contract let to a private power company to deliver all water to the city's distribution system for $1,000.00 less than it had cost the city the previous year, making the rate 2¾ cts. per 1,000 gals. meter measure.

Readjustment of rates, based on a sliding scale, cutting the price of 150,000 gals. from $30 to $15.50.

A survey of all existing tree lines in the streets, to aid in the formulation of an ordinance regulating these lines.

The adoption of standard plans and specifications for every class of work coming under the city's control and regulation.

The inspection of all plumbing, sewer and water lines, chimney construction, meters, electrical work, etc.

The re-establishment of a complete system of grades, and the rigid recognition of this system.

Some Faults of Engineers from the Contractor's Viewpoint.

Probably the majority of young engineering graduates go upon their first construction job with a firm belief that contractors are determined to take advantage of engineers at every opportunity. Consequently the young engineer is inclined, oftentimes, to get the best of the contractor before the contractor gets the best of him. Some aspects of this matter and its effect on bidding prices were discussed recently before the Albany Society of Civil Engineers by Mr. Richard W. Sherman, chief engineer of the New York State Conservation Commission. The following paragraphs are from Mr. Sherman's paper:

I have known experienced contractors who, when they read a list of engineering graduates, said, with an air of despair: "See what has been turned loose on us now." Contractors dread the "boy engineer" just from college without, as a rule, any practical experience in construction. Experience as contractors has taught them that the engineer just graduated, with a few exceptions, is prejudiced against contractors. These young engineers are extremely technical. They expect a literal compliance with every provision of the contract obligations by the contractor, concede the contractor few, if any, rights; consider the engineer as infallible and final in all matters under the contract and almost scoff at the contractor's views of his rights and his construction of the meaning of contracts and specifications. They, as a rule, are conceited, opinionated, arbitrary, unjust and unmerciful. This is about the state of development in which a large majority of engineers start out on their professional careers With few exceptions, they immediately begin to improve, to lose much of their conceit, to learn that their opinions are not infallible and to be less arbitrary and more just.

Contractors are largely influenced by their opinions of engineers. The engineer who has a reputation for ability, honesty, fairness and good disposition, will attract bidders for any work of which he has charge and the desire to do work under him would be an incentive to reasonably low prices. On the contrary, if contractors consider an engineer incompetent, dishonest, an inebriate or of a cranky disposition, they will often avoid bidding on the work of which he has charge or add to their bids a sum which they hope will cover the excess cost of the work due to the unfavorable attributes in the makeup of the engineer. It is a feature of contracting to "size up" the engineer with as much accuracy as possible, and many contractors become very expert thereat.

In bidding for work, contractors are almost as sensitive as weather vanes. As a rule, they run quite a risk of loss if they bid too low; and the engineer is at least one of the most important features in the situation, as by reason of the characteristics of the engineer it may be possible to make a profit at a given bid under one engineer and impossible to avoid a loss under some other engineer with all conditions, aside from the engineer himself, precisely similar.

Contractors who do not care for the contract often bid fairly high without any expectation of securing the contract, but merely to avoid a reputation among contractors of being low bidders and with the bare chance of getting the work at good prices. Excessively high bids are usually the result of lack of knowledge of the value of the work, lack of time to become familiar with it, and not infrequently from a variety of reasons by which the engineer has been "sized up" so unfavorably by some contractors that they do not want the work under him, are prejudiced against him and the work and consequently unintentionally or otherwise bid too high. The treatment which bidders think they will receive from the engineer if they secure the work is an important feature of the bidding.

If an engineer's preliminary estimate is believed to be too low it drives away bidders and tends to indifferent high bidding. Some over-anxious contractors may be influenced thereby, to bid too low. They may secure the work, in which event the engineer has an unpleasant task during construction. There is almost sure to be a disposition on the part of the contractor to save himself from loss, and he is thus tempted to slight or cheat in the quality of the work which the engineer tries to prevent (as he should). The engineer, no matter how fair and just he may be to the contractor, will usually end with a feeling of enmity toward him. Both contractor and engineer are in some degree injured by the work having been done at less than cost. It is detrimental to an engineer to make a practice of making his preliminary estimates higher than the prices at which it is later found the work can be let to reliable parties. On the other hand, great annoyance and trouble and, in most cases, detrimental features which in one way and another cause loss to the owner and detriment to the reputation of the engineer result from making preliminary estimates too low.

In the matter of monthly and final estimates, contractors, particularly those ignorant of engineering, are very prone to suspect that engineers have underestimated or cheated them. There never has been any good reason why contractors should not check up measurements and quantities even if it is necessary to employ contractors' engineers to do so. In large contracts this has been the custom for some years and is becoming more so. Many more engineers than formerly are engaged in contracting and this feature adds to the practice of contractors measuring and checking up estimates. It is to be hoped that this practice will become general.

Great injustice has been done engineers, as many intelligent contractors know, by the perpetual suspicion that the engineer is under of having cheated contractors. There is seldom a motive other than spite for an engineer to cheat a contractor and the spite cases I believe to be very rare. In small works, for instance, such as village water works and sewers, some engineers may be tempted to underestimate in order to keep the cost of the work down to the preliminary estimate or appropriations. I trust there are not many such, but I fear there are some.

An indolent, indifferent engineer and also a procrastinator, are torments to contractors and a cause of considerable unnecessary cost in executing the work.

ROADS AND STREETS

Organization for and Methods and Cost of State Aid Road Construction in Alabama.

(Staff Article.)

The state highway commission of Alabama consists of five members: the state geologist, a professor of engineering in the Alabama Polytechnic Institute, and three civilians appointed by the governor, who hold office for four years. The commission serves without pay and has general authority in the interpretation of road laws and outlining the state's policy in road improvements.

The state highway engineer selected by the commission acts as their executive officer. The selection of state aid roads, the determination of the character of improvement and the supervision of construction are duties of the state engineer together with the collection of information and the preparation of a general highway plan for the state.

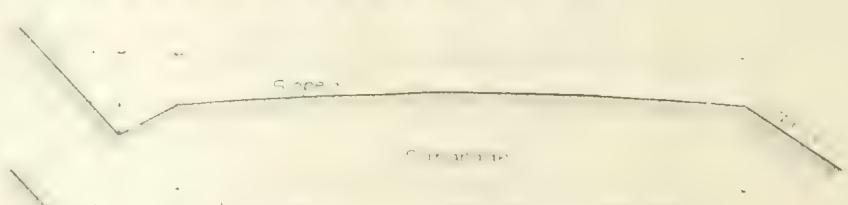


Fig. 1. Typical Cross Sections for Sand-Clay Roads.

In this state the county is the unit of government in road and bridge construction and maintenance, the state highway commission having authority only over state aided roads. County road affairs are in the hands of boards of county commissioners or boards of revenue that apportion funds for county roads and perform other necessary duties. Many counties maintain county convict gangs or employ road forces under the supervision of county road foremen. These forces are constantly employed on work within the various counties. State aid road construction supplements the work of the various counties. ENGINEERING AND CONTRACTING for June 24, 1914, gives further data upon the present organization.

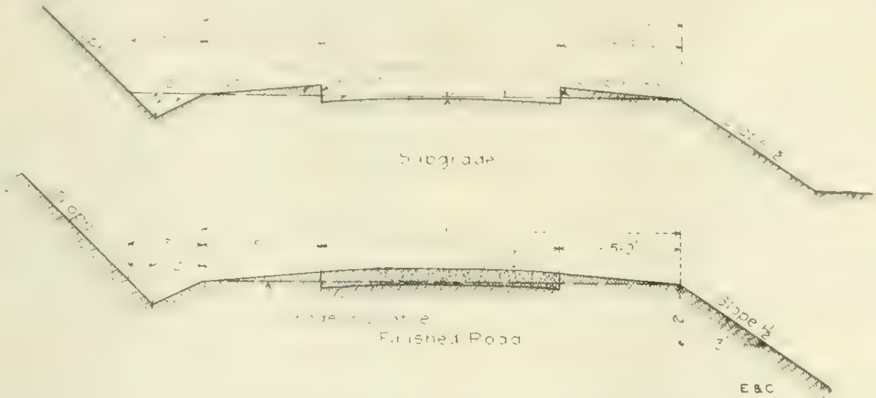


Fig. 2. Typical Cross Sections for Gravel and Chert Roads. Note Profile Grade.

STATE AID FUNDS AND METHOD OF APPORTIONMENT.

A cash appropriation is made by the state legislature which is apportioned equally to the various counties on condition that the counties provide an equal amount. In 1913 the state appropriation was $154,000, less the expenses of the commission, and the amount apportioned was about $2,000 to each county. In some cases $4,000 was apportioned. County aid if not used may accumulate for several years and be used at the termination of a period of years.

A county desiring aid makes application to the state highway commission. The approval of the application, the preparation of plans and specifications and the supervision of the actual construction are under the direct control of the state highway engineer. After the completion of a state aid road maintenance is in the hands of the county, but if such maintenance is not performed as directed by the state the commission has authority to maintain the road, deducting the cost thereof from future state aid appropriations to the county.

TOPOGRAPHY AND SOCIAL CONDITIONS.

The state of Alabama is mountainous in the northern section, hilly in the central section and somewhat rolling in the southern portion. Stone or gravel is well distributed throughout the state and natural sand-clays are abundant in portions of it. The soil for the most part is of a residual type composed of sand and clay, although in the valleys and along the coast alluvial soil is found.

For the most part the state is sparsely settled and the main occupation of the inhabitants is that of farming. In the northern portion are many mines and there is much manufacturing there. The negro population is large and most public works are carried on with this class of labor.

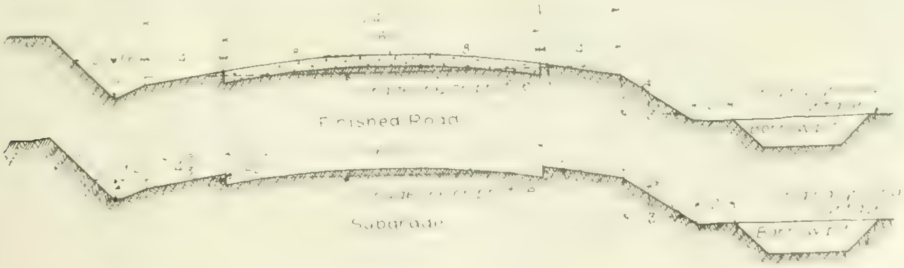


Fig. 4. Typical Cross Sections for Gravel and Chert Roads. Note Profile Grade.

METHOD OF STATE SUPERVISION.

The methods employed by the state in the supervision of construction work, as stated in a recent report of the commission, are given below.

Application for Aid.—No financial aid is given by the state until the county has appropriated and rendered available a sum equal to the amount to be received from the state. Accompanying the application shall be attached the probate judge's certificate or treasurer's receipt showing that the county has set aside a fund to be known as the state aid road fund to be used jointly by the state and county.

Selection of Work to Be Performed.—The commissioners' court or board of revenue designates the section of road to be improved. No financial aid will be given any county by the state unless the contemplated work is to be on a main traveled thoroughfare, culvert or bridge and unless the improvement is to be of a permanent nature and a public utility and convenience. The state highway commission approves the contemplated work upon report of the state engineer or his assistant. In the construction of roads, one year's appropriation is used on a continuous section of road. However, a different road may be selected each year, or if two years' appropriations are being used, half may be used on one road and half on another.

Length of Road.—The construction begins at the point designated and continues as far as possible with the funds in hand.

Right of Way.—Width of right of way of all state aid roads must not be less than 50 ft. and must be acquired by the county, either by donation by the owners of lands through which such road will pass, or by agreement between the owners and the proper authorities in such county, or by the exercise of the proper eminent domain; without expense to the state and exclusive of the $2,000.00 appropriation made by the county. Right of way must be secured before any contract is let or construction work begins.

Rules of Construction.—The width of the travel way of any state aid road must not be less than 20 ft. between side ditches. The space between the improved surfaced part of any road and the ditches, known as the shoulders, shall not be less than 4 ft. each in width and the surfaced part on macadam roads shall be not less than 10 ft. in width. On gravel roads, the surfaced or graveled width shall not be less than 12 ft. Sand-clay may be placed any width exceeding 14 ft., but not less than 14 ft. The depth of surfacing material will vary with the quality of the material and will be left to the judgment of the state highway engineer or his assistant. The grade of any state aid road shall not exceed 5 per cent. The best surfacing material near the work may be used and it must be approved by the state highway engineer or his assistant. Specifications for the construction of any state aid road or bridge shall be prepared by the state highway engineer. Where a county has a competent county engineer, he may prepare plans, profiles and specifications, but such specifications must be submitted to the state highway engineer for his approval or disapproval, or for such changes as he may deem wise.

Change in Location.—In order to secure a good alignment and not exceed the maximum grade or to secure a better foundation, it is often necessary to make changes in the old road. Such changes in location, where necessary for good construction, in the opinion of the state highway engineer, shall be adopted by the county, and such disposition of the old road made as the county sees fit. The state will not be responsible for damage to crops or land caused by change in location of a road or by the change of the flow of water caused by the construction of a road or culvert.

The state highway engineer is the executive officer of the commission, and he has charge of all state aid work. All employes on state aid work are subject to his orders. Upon requisition in writing from the state highway engineer, the county or contractor shall discharge any employe engaged in state aid work, provided no man be discharged except for cause. The following reasons shall be deemed cause for discharge: incompetency, dishonesty, drunkenness, insubordination and misconduct.

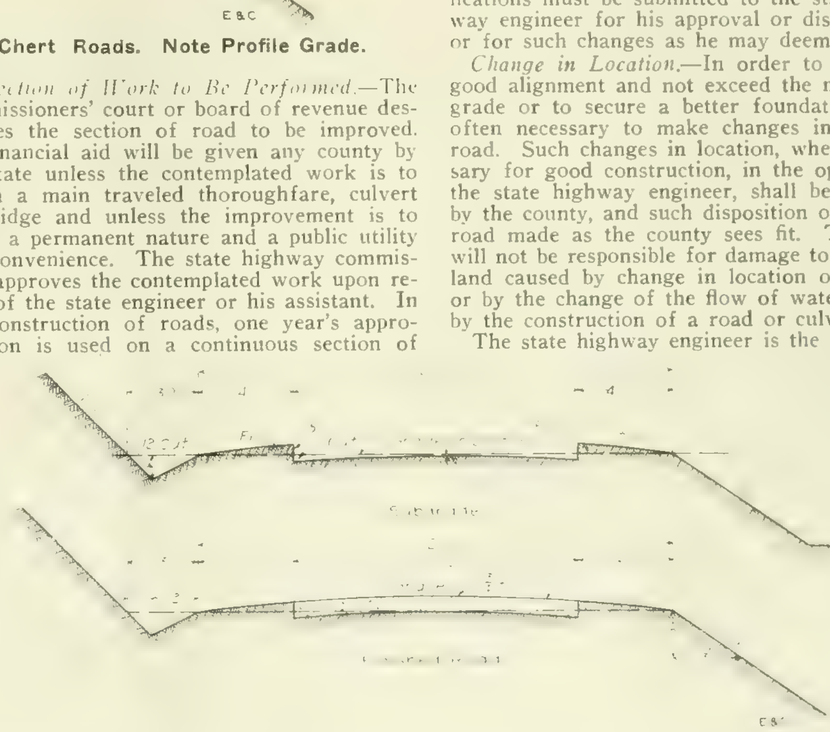


Fig. 3. Typical Cross Sections for Gravel and Chert Roads. Note Profile Grade.

The Survey.—The state makes a survey only long enough to cover the amount of money to be expended, or to secure proper alignment. If any county wishes to continue the survey were the state aid survey stops, or make another survey, the county employs the engineer. The commission allows resident engineers, when not engaged on work for the state, to do this work. The state highway engineer will consult and advise with any county relative to any survey work on any highway other than that upon which the state aid money is expended.

Resident Engineers.—Application being made and approved and the road selected, the county will be furnished an engineer as soon as possible. Resident engineers are required to furnish their own instruments, rods, chains, etc. The commission furnishes profile paper, note books, keel, report blanks, pay rolls, estimate blanks, etc.

Salary of Resident Engineers.—Resident engineers are paid at the rate of $5.00 per day and they pay their own maintenance after [illegible] helpers are allowed the resident engineer and paid for at a reasonable rate. Where possible, an engineer is used in more than one county. When en route to a county, his expenses are allowed, such as railroad fare, board and lodging [illegible]

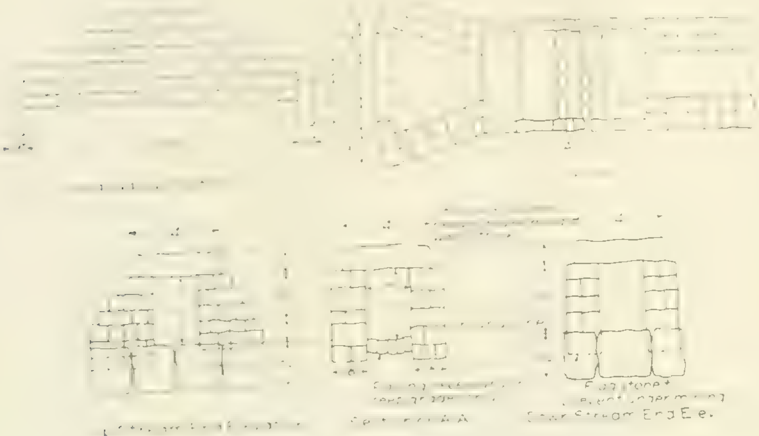


Fig. 5. Standard Rubble Culvert Design.

gage and necessary expenses incurred by the work. In going from one county to another, the county to which he goes pays the expense incurred reaching that county one way only. All salaries of the resident engineer, rodmen and helpers and their traveling expenses are paid out of the joint fund of the state and county and charged to the road. All expenses must be shown in detail and receipts attached for all expenditures over $1.00, except railroad fare. Engineering pay rolls are made out in duplicate at the end of the month and both the original and duplicate sent to the office for approval. The duplicate is retained in the office and the original returned for payment. Probate judges should see that all engineering pay rolls and other vouchers are approved by the state highway engineer before payment.

How the Work Shall Be Done.—The commissioners' court or board of revenue shall say whether the work shall be done by contract, by their own forces or by hired forces.

Contract Work.—If the work is to be done by contract, the law requires that bids be advertised for 30 days before bids are received. A copy of the plans and specifications are on file with the probate judge and in the office of the commissioners for the information of contractors. On the date of receiving bids, the state highway engineer or his assistant is present to open bids and advise the county whether to accept or reject them. If bids are satisfactory, the engineer recommends that the work be awarded to the lowest responsible bidder. The state highway engineer will say when a bidder is responsible. All contracts are made conditional upon the approval of the commission. Contracts will be approved or rejected by the commission at the first regular meeting after they have been awarded. A bond equal to double the amount of the contract will be required on all state aid contracts. Monthly estimates are given the contractors for the work performed.

County Forces.—If a county owns its own equipment, such as wheel and drag scrapers, road machine, roller, plows and small tools, the work can be done with its own forces. The state is not responsible for damage to any machinery, tools or teams engaged in the construction of state aid roads. The state will not buy any machinery, tools, supplies, etc., except those actually consumed in the work, such as cement, pipe, lumber for bridges, or forms, dynamite, nails, etc. Such items as shovels, picks, harness, etc., every outfit should have, and they can no more be paid for than wheel scrapers, road machines and rollers. Small repairs necessary while work is in progress will be paid for.

There are two ways of working county forces. One in which all labor is paid for at the prevailing price of labor in that section and allowing a reasonable amount per day for teams and drivers belonging to the county or hiring teams. If this method is adopted, no supplies are paid for and no labor or teams maintained. The other method—all labor is paid for at the rate paid by the county and teams are maintained, but no allowance per day for teams will be made. When convicts are used both men and teams are maintained and the salaries of foremen and guards are paid out of the joint fund of state and county until it is exhausted.

DETAILS OF CONSTRUCTION.

Cross sections.—Figures 1 to 4 illustrate typical cross-sections used on state aided roads. It will be noted that heavy grading is finished to a flat cross-section shown on the profile. Fine grading and shaping forms the flat surface to the desired cross-section. The relation of the finished cross-section to the flat grading is shown on the plans.

The following abstracts from specifications of the commission describe the the details of construction used on various types of roads.

EARTH ROADS.

Grading and Grubbing.—The completed grade must conform to the grade as established and shown by profile in the office of the State Highway Engineer at Montgomery and stakes set by the County Engineer. Nothing other than earth, gravel or stone will be allowed in any embankment and all brush, logs or matter that will decay must be removed.

All decayed stumps must be grubbed up. Green stumps will be allowed where there is as much as eighteen inches of earth fill over the top of same, when grade calls for less than eighteen inches of fill over top of green stumps, said stumps must be grubbed up. The grade when completed must have a crown or top width of .. feet. This width applies to both grade on embankments and in cuts. Side ditches or gutters must be made in cuts of sufficient width and depth to carry all water reaching the road during the heaviest rainfall. Such ditches where emptying from cuts to embankment must be turned so as not to

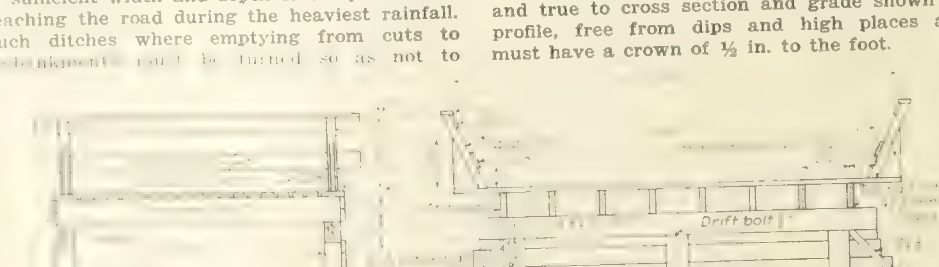


Fig. 6. Standard Wooden Bridge Design.

[illegible] The slope of embankment [illegible] horizontal to 1 ft. vertical and the slope in cuts will be one foot horizontal to one foot vertical. The right to alter the slopes is reserved, but in all cases will be shown by slope stakes, and all changes in earth quantities will be properly shown and accounted for. Material taken from excavations shall be deposited in embankments unless otherwise ordered by the Resident Engineer.

Over-Haul.—The engineer will indicate on profile where the distance of haul exceeds 500 ft. All earth moved more than 500 ft. shall be known as over-haul and shall be paid for at the rate per cubic yard bid by contractor plus —— cent for each additional 100 ft. over 500 ft.

Classification of Material.—Excavations will be classified under the following heads, viz: earth, loose rock, and solid rock.

Earth will include loam, clay, sand, gravel, marl, decomposed rock and slate, stone and boulders containing less than one cubic foot of material, and all other material of any earthy kind, however hard, stiff or compact. Loose rock will include all boulders and detached masses of rock over one cubic foot in bulk, and less than one cubic yard; also all slate, coal, shale, and other rock soft or loose enough to be removed without blasting, although blasting may occasionally be resorted to. Solid rock will include all rock occurring in masses of more than one cubic yard and which in the judgment of the engineer requires blasting. In road alterations, or changes of water courses, the contractor will be entitled to the same compensation as for like material on the same section.

Berms.—Where fills of as much as two (2) feet are made from side ditches or borrow pits, berms 4 ft. in width shall be left between such ditches or borrow pits and the toe of the fill.

Finished Grade.—The finished graded road must have a smooth even surface, free from holes or high places and must be to grade established and shown on profile. The width between ditches must be —— ft. at all points. The fall from center of road to side ditches must be at the ratio of one inch to one foot. On grades this fall must be increased sufficiently to carry water that falls on the road to side ditches instead of down the road. Side ditches or gutters must be parallel to the center line of the road and free from any obstruction.

Grades to Be Surfaced.—Before any State Aid Road is surfaced with gravel chert, stone, sand-clay or other surfacing material, it shall be graded in accordance with the foregoing specifications, with the exception that road bed must be either flat or have a crown not exceeding a rise of ½ inch to the foot.

Payments.—The grading will be paid for in excavation only. If the excavation exceeds the embankment all wasted material is paid for. If the excavation is not sufficient to make the embankment the borrow will be paid for. Payments for the work done shall be made monthly on the estimates furnished by the Resident Engineer upon being approved by the State Highway Engineer; 15 per cent being reserved until the work is completed and accepted by the State Highway Engineer.

SPECIFICATIONS FOR TOP SOIL OR SAND CLAY ROADS.

Before any top soil or sand clay is brought on the road the finished grade must be smooth and true to cross section and grade shown on profile, free from dips and high places and must have a crown of ½ in. to the foot.

Top Soil or Sand Clay. The finished road shall have a crown or slope to the ditches of ...... inch to the foot. The top soil or sand clay shall be ...... inches deep and ...... feet wide with a 4 ft. shoulder on each side. Top soil or sand clay will be used from pits along the right of way and they will be designated by the engineer. No top soil or sand clay shall be placed on the road until it has been inspected and passed by the engineer. The top soil or sand clay shall be free from all trash and foreign matter and shall contain no rocks or boulders that will not pass through a 1½ in. ring. If any such non-road building material is placed on the road it must be removed by the contractor at his own expense.

All top soil or sand clay pits will be furnished by the county and the price bid by the contractor is for loading, hauling, dumping, spreading, shaping and maintaining. The top soil or sand clay will be paid for by the cubic yard measured in wagons before it is dumped. If any other means is used to apply the surfacing as by wheelers or shoveling, the top soil or sand clay will be paid for by road bed measurement and the yardage will be arrived at by multiplying the depth by the width and the length, in feet, of the surface top soiled and divided by twenty-seven.

The pits from which the top soil or sand clay will be taken are located at Station .....

The pits are furnished by the county and the contract price bid by the contractor is for hauling the material from these pits. In the event that the material is found unsatisfactory or the material gives out, necessitating the hauling from another pit, and the length of the haul is not increased, the price bid will still apply to the contract.

The longest average haul shown on the profile is ...... ft., and in the event the haul is increased for every additional 1,000 ft. haul over ...... ft., I agree to haul the material for ...... cts. per yard, additional to the contract price, including spreading, dressing, etc.

Dumping.—The top osil shall be dumped on the road with three or more loads opposite each other; the distance between the loads depending on the width and thickness of the top soil. The loads should be dumped in this way in three or more parallel lines until a hundred or so feet have been dumped. The shaping of the top soil should be commenced before the individual loads begin to pack.

Shaping.—The grader or road machine should first be run over the top of the piles, filling in the space between the piles. Next the outside edges should be gone over with the grader, set so as to pull the sand clay or top soil toward the center. The shoulders should then be pulled up against the sand clay or top soil by the use of the grader and the entire road bed smoothed and trimmed to the cross section shown on the plan.

This shaping and dressing should be kept up from time to time until the road packs firm and smooth. The best time to surface and trim up the road is when the ground is just beginning to dry after a rain.

The contractor must keep the road dragged either with road machine or split log drag until it becomes hard and firm, free from ruts, holes, bumps and irregularities of surface.

Additional Material.—Where it is deemed necessary to place additional surfacing on the road, the engineer will order same placed and see to the spreading of it. The contractor will be paid for such extra material at the price bid for sand clay or top soil.

Mixing Sand and Clay.—If the surfacing used is not a good natural mixture of sand and clay the mixture will have to be made on the road in the following manner on a clay foundation the sub-grade must be plowed up to a depth of 4 ins., all clods being thoroughly pulverized by harrowing or otherwise, and sand spread on to a depth of 6 ins., the mass shall then be mixed and puddled by turning with a plow and using a disc harrow and dressed up with the road machine. On a sandy foundation the sand must be plowed to a depth of 6 ins. and a suitable clay spread to a depth of 4 ins. and mixed as above described. The mixing and puddling process must be kept up from time to time until a good mixture is obtained and the road packs firm and hard and is true to grade and cross section and free from holes and bumps. The mixing will be paid for by the square yard and the material that is hauled whether sand or clay will be paid for at the price bid.

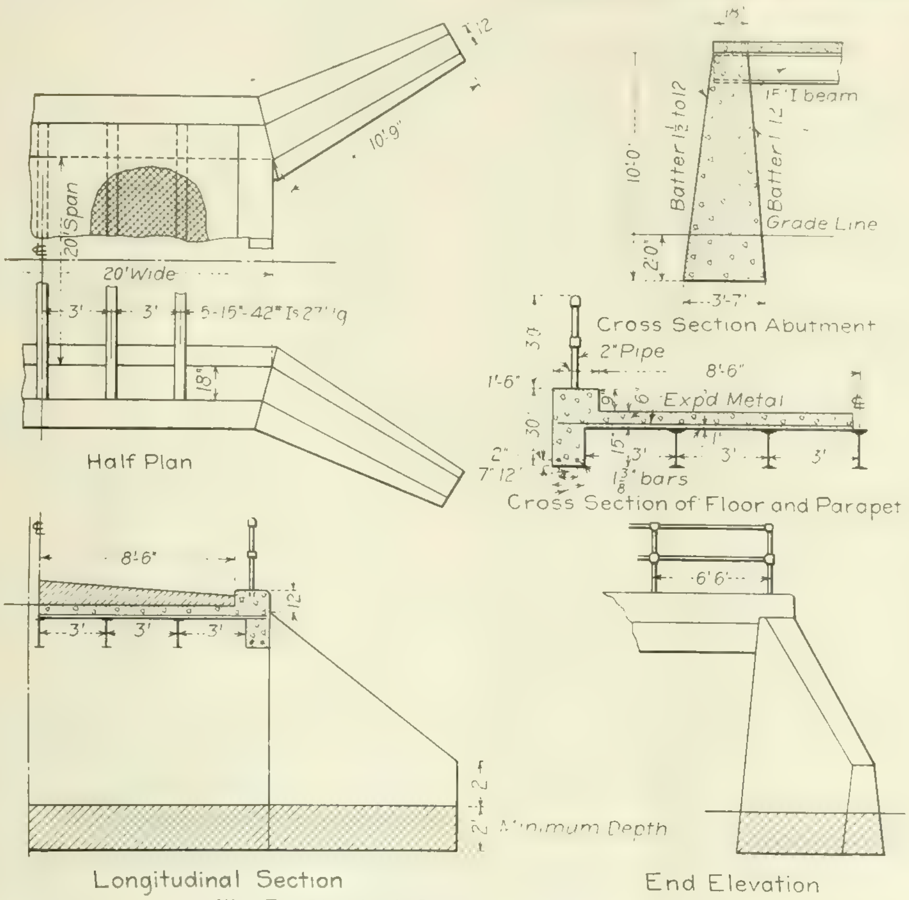


Fig. 7. Standard Concrete Bridge Design.

SPECIFICATIONS FOR GRAVEL ROAD.

Sub-Grading.—After the road has been graded according to specifications herein given and before any gravel is placed on the road, a sub-grade must be prepared. Sub-grading consists of cutting out a trench or pit the width and depth the road is to be surfaced so that the earth surface will have the same shape and fall from center to ditches as the finished road will have with the addition of 4 ft. shoulders on either side to hold the material in place, as well as to provide for additional room for vehicles to pass. This pit must be cut so that when filled with gravel, water falling on the road will flow easily from the gravel surface on to the shoulders into the ditch. After the gravel has been placed according to the specifications herein given, the shoulders must be dressed with grading machine, or by hand until they are smooth and even with uniform width and with a fall of 1½ ins. to the foot.

Just before any gravel is placed the sub-grade must be thoroughly rolled with a roller weighing not less than six tons.

Gravel.—Before any gravel is used in the construction of a State Aid Road, it must be examined and approved by the State Highway Engineer, or the Assistant State Highway Engineer. If it is found advisable to add screened gravel or clay to the gravel it will be so specified in the specifications for the work in hand.

All gravel pits will be furnished by the County and the price bid is for loading, hauling, dumping, spreading and finishing the road to the required cross section.

When gravel is shipped in, the contractor must pay the freight on the gravel, load and unload cars, and any demurrage must be paid by the contractor. The County will not be liable for any damage occasioned by delay in furnishing cars for gravel or the movement of same.

The pits from which the gravel will be taken are located at Station ....; Station ......; Station .... These pits are furnished by the County and the contract price bid by the contractor is for hauling the material from these pits. In the event that the material is found unsatisfactory or gives out, necessitating the hauling from another pit and the length of the haul is not increased, the price bid will still apply to the contract.

The longest average haul shown on the profile is ...... ft., and in the event the haul is increased for every additional 1,000 ft. haul over ...... ft. I agree to haul the material for ...... cts. per cubic yard additional to the contract price including spreading, dressing, rolling, etc.

Fig. 8. Typical Concrete Girder Bridge.

Graveling.—After the road has been graded and sub-graded as specified, a coat of gravel, [illegible] Engineer or the Assistant State Highway Engineer, shall [illegible] depth of [illegible] to a uniform width [illegible] must be dumped and rolled with a 10-ton power roller until the surface is hard and firm and until the gravel ceases to creep in front of the roller with every particle of gravel thoroughly nested. No gravel must be placed on road when the surface of the sub-grade is wet or spongy. The road must present a smooth, even surface, free from dips or high places, and must conform to the grade and cross section shown on the profile and plans. The ditches must be parallel to the center of the road, except where it is possible to turn ditch away from the road.

The contractor must see that no strippings, earth or other inferior or non-road building material is placed on the road. Where such material shows on the road the contractor will be required to move same and place instead material that has been approved by the Engineer or his Assistant.

Clay.—When it is deemed necessary by the Engineer to place sand-gravel on the road, a good quality of mineral clay approved by the Engineer will be required mixed with said sand gravel to make a firm bond of the mass. Specifications heretofore given for quarrying and hauling gravel shall cover the excavating and hauling of clay.

Mixing Clay.—The mixing of the clay and gravel on the road must be done in strict accord with the following directions: Where possible, the wagons must be loaded one-third clay and two-thirds gravel and dumped on the road. Where such is not possible, 4 ins. of clay must be hauled and placed on the road, after which 5 ins. of gravel must be placed on said clay. In both cases after 400 or 500 lin. ft. of the mass has been placed, a thorough plowing must be given the road so that the clay is well broken; after such plowing the entire mass must be harrowed until the gravel and the clay are thoroughly mixed. After the thorough mixing and the road has sufficiently dried out, it must be properly shaped so as to have a fall from center to ditches of 1 in. to the foot and then thoroughly rolled. In the event a lack of mixing is apparent, the contractor will be required to repeat the process until the mass is well mixed.

SPECIFICATIONS FOR CHERT ROADS

Before any chert is used in the construction of a State Aid Road, it must be examined and approved by the State Highway Engineer or the Assistant Highway Engineer. If it is found advisable to screen the chert it will be so [illegible]

When chert is not screened it must be placed on the sub-grade the required depth, which in no case must be less than 6 ins. and must be immmediately spread after dumping to the required form of the road; that is, with fall from center to shoulders of 1 in. to the foot, and must be kept smooth and even. Any solid pieces of chert more than 2 ins. in diameter must be broken with a hammer before the mass is rolled. After as much as 300 lin. ft. has been so spread, it must be rolled with a road roller weighing not less than 6 tons. If possible, a steam or gasoline roller must be used.

The road must be rolled until the chert ceases to creep in front of the roller. If the chert is dry and does not readily bind under the action of the roller, it must be sprinkled before rolling.

Finished Road.—The finished road must be to grade as established and shown on profile, plus the thickness of the surface material. The surface must be smooth and even, free from dips, holes and depressions, bumps and high places; hard and firm and free from spongy places. The alignment of the road must be in strict accord with the alignment as established by the engineer, and side ditches or gutters must be parallel with center line of the road, except where it is possible to turn the water from the roadway. The road must present a neat appearance. All material which has been condemned, all broken tools or machinery, plank or other material, or tools not [illegible] removed from the road right-of-way. At all curves the trees and brush must be cut far enough back from the road so that an aproaching vehicle can be seen from any point for a distance of 100 ft.

Payments.—Payments shall be made monthly for the actual number of cubic yards of chert placed on the road in accordance with the foregoing specifications, by actual count of cubic yards of loose material in wagons as made by an inspector furnished by the county, less 15 per cent (15%) reserved until the entire road is completed.

The pits from which the chert will be taken are located at Station ......; Station ......; Station ......; Station ...... These pits are furnished by the county and the contract price bid by the contractor is for hauling the material from these pits. In the event that the material is found unsatisfactory or gives out necessitating the hauling from another pit and the length of the haul is not increased the price bid will still aply to the contract. The longest average haul shown on the profile is ...... ft., and in the event the haul is increased for every additional 1,000 ft. over ...... ft. I agree to haul the material for ...... cts. per cubic yard additional to the contract price, including spreading, rolling, dressing up, etc.

Where chert is shipped in, the contractor must pay the freight on the chert, load and unload cars, and any demurrage must be paid by him. The county will not be liable for any damage occasioned by delay in furnishing cars for chert or the movement of same.

BRIDGES, CULVERTS AND HEADWALLS.

Concrete.—Concrete shall be composed of Portland cement (subject to engineer's approval), clean, sharp sand or limestone screening, and durable stone, gravel or slag broken to pass through a 2½-in. ring and free from dirt or clay. The proportion of the mix will depend on the material used, but it will be approximately one part of cement to three parts of sand or screening, and five parts of stone gravel or slag. The mass shall be mixed in small batches thoroughly dry, then sufficient water added to make it consistent with placing and then mixed again after being wet. The concrete shall be placed without delay and thoroughly tamped in place. The forms shall be so carefully placed that when removed the concrete will present a smooth surface. All facings that are exposed to view shall be finished by spading. All reinforcement shall be to size and length and placed as shown on plan and tied so as not to be displaced by concreting. All reinforcement shall be embedded to a depth of at least one inch.

Concrete will be paid for by the cubic yard and the price bid includes all excavations unless in solid rock. Then the price bid for solid rock shall be paid.

Bridging.—The timber composing the bridges shall be long leaf yellow pine or white oak and must be 90 per cent heart or better, free from shales, loose knots or other imperfections.

Drain Pipe.—In localities where but small quantities of water pass, drain pipe shall be used for culverts. Contractors will bid furnishing and laying best double strength vitrified stone sewer pipe, of sizes as shown on [illegible]

Before laying the pipe the bottom of the trench shall be rounded out to fit the pipe from the lower surface to the horizontal center line. Depressions shall be cut in the trench to fit the sockets, so that when the pipe is laid its entire lower surface from end to end shall have a solid bearing. All joints of the pipe shall be cemented with mortar, composed of clean, sharp sand and Louisville cement, mixed one to one. The ends of the pipe shall be protected by headwalls as shown by detail drawings.

Box Culvert Masonry. Box culvert masonry shall be laid dry. The side walls shall be built of good size and well shaped stones properly laid and bonded together in each course by stones extending entirely through the wall at least every 6 ft. in the length of the wall.

Headers and stretches shall not be less than 15 ins. wide and at least as wide as high. The back of the wall is to be the same as front, with the exception of the facing. The upper course to have at least one-half of the stones, headers and stretchers in no case less than 18 ins. wide; no stone in the course to be less than 8 ins. thick.

The covering stones to be sound and strong and of such shape to form suitable joints; to be of approved thickness according to width of opening but in no case less than 12 ins. thick, and to lie with their whole width not less than 12 ins. on each side of the wall. Care will be taken to show a neat finish at the ends of the culvert.

COST OF WORK ACCOMPLISHED.

The cost of work accomplished during the past year, the extent and type of which is shown in Table I, is given in Table II. About one-half of this work was accomplished by contract and one-half by day labor or convict forces.

TABLE I. — EXTENT OF ROAD WORK IN ALABAMA FROM MARCH 1, 1913, TO APRIL 1, 1914.

Kind of road.	Miles built. By state aid.	By counties.	Total.
Graded	31.81	531.70	563.51
Top soil	44.72	170.00	214.72
Gravel	24.02	153.30	177.32
Chert	21.53	101.75	123.28
Macadam	7.00	44.75	51.75

The total expenditure for road work from March, 1913, to April, 1914, on work supervised by the highway commission and other work performed independently by the counties was as follows:

Expended by counties:	
For road work	$2,003,146.00
For bridge work	529,638.00
Expended by state and counties jointly:	
For road work	256,795.41
For bridge work	44,168.26
Total	$2,833,747.67

TABLE II.—UNIT COSTS OF STATE AID ROAD WORK IN ALABAMA FROM MARCH, 1913, TO APRIL, 1914.

Item.	Maximum.	Average.	Minimum.
Earth excavation, cu. yd	$ 0.38	$ 0.23	$ 0.13
Loose rock excavation, cu. yd	.73	.42½	.30
Solid rock excavation, cu. yd	1.50	.90⅓	.50
Lumber in bridges, M ft.	50.00	33.26*	22.00
Box culvert dry masonry, cu. yd	4.50	3.56	1.50
Retaining wall dry masonry, cu. yd	6.00	3.37†	1.18
Riprap, cu. yd	3.50	2.05	.50
Plain concrete, cu. yd	12.00	8.78‡	5.00
Reinforced concrete, cu. yd	14.00	11.74	7.00
Clearing and grubbing, per acre	92.00	52.93§	15.00
Clearing and grubbing, per mile	225.00	......	20.00
Sand-clay or top soil in place, maintained 30 days, cu. yd	......	.44¼	......
Gravel	......	.86	......
Chert	......	.80**	......
Macadam	......	1.46***	......
Vitrified culvert pipe in place, per lin. ft.—			
12-in.	.85	.62	.25
15-in.	1.30	.83½	.55
18-in.	1.50	1.07	.50
[illegible]	2.00	1.40	.90
[illegible]	2.50	1.60	.65

[illegible] jobs. §13 jobs. **7 jobs. ***5 jobs.

The cost of operating the state highway department in addition to the above was as follows:

Item.	
Salaries	$ 7,[illegible]86.54
Traveling expense	1,288.97
Miscellaneous	1,279.50
Total	$10,155.01

PERSONNEL

The personnel of the present highway commission is as follows: Robert E. Spragins, chairman, G. H. Hitcham, E. A. Smith, V. B. Atkins and John Craft. W. S. Keller is state highway engineer and secretary of the commission, and R. P. Boyd, assistant engineer.

Instructions Governing the Care of Steam Rollers.—A book of instructions to employes recently issued by the Pennsylvania Highway Department contains instructions for the care of steam rollers. It is of especial importance, the instructions state, never to blow out the boiler below normal water level while under steam pressure. The following directions are also given:

1st. Thoroughly clean the boiler as follows: Remove hand-hole plates in firebox legs, also in smoke-box, and over crown sheet. Use a hose with good pressure and wash out all dirt and scale. If there is anything which cannot be washed out with the hose, use a small scraper or the hand and pull it out of hand-holes. Then put hand-hole plates with a gasket in position and fill boiler with water until all flues and crown sheets are covered. Put a quart of common oil in the boiler and allow the water to run out. Then take out one hand-hole plate in the lowest point in the legs, and sponge or soak out any water that may remain.

2d. Scrape off all scale or corrosion inside of firebox and flue sheet. Sweep clean with a steel brush or broom and then apply a coat of boiled oil. Do the same with the smokebox. Scrape clean the outside of firebox and also all parts of the boiler that are not jacketed and paint with a good locomotive black (linseed oil and lamp black will answer if locomotive black cannot be obtained). Smokestack also should be painted.

3d. Remove the injectors and see that they are perfectly dry. Drain all pipes, both steam and water, dry.

4th. Remove manhole plates in bottom of tanks and drain them dry and paint inside with white lead and boiled linseed oil (never use boiler paint). If this is done injector troubles from tank scale will be avoided. Clean all grease and dirt from all parts of the machine, as it will come off more easily now than it will next spring.

5th. Remove the old packing in piston and value steam glands. Take off the cylinder heads and steam chest bonnets.

6th. Especially prepared grease or heavy cylinder oil should be applied with a brush to all parts of the engine, inside the cylinders and valve seats, piston rods, crossheads, slides, connecting rods, cranks, lever handles; also Russia iron jacket and every place that is liable to rust.

Under no consideration should the coal be left in coal bunker when roller is not in use. The corners of the coal bunker and the foot plate should be thoroughly cleaned and accumulation of coal dust removed to prevent rusting. A coat of paint (same as used on the boiler) will greatly add to the life of the tank.

If the roller is exposed to weather, cover the top of smokestack to keep out rain and snow. Examine the roller carefully, especially links, link blocks, eccentric rods, slide bushing, connecting rods, crosshead shoes, gears and all parts which may have become worn and in need of repair.

CONSTRUCTION PLANT

MACHINES DEVICES MATERIALS

A Pneumatic Railroad Tie Tamper.

The two views shown herewith illustrate the pneumatic tie tamper now in use for tamping ballast under railroad ties and for surfacing track on the New York Central lines. Fig. 1 is a close view of the tamper, and Fig. 2 is a view of the application of the tool, working in pairs, on the River Division of the New York Central and Hudson River railroad. It is claimed that the tool here described effects a saving over hand methods in tie tamping, is more readily used in points difficult of access, as under frogs and switches, and does better and more permanent work than is possible by hand tools. It is claimed that two men, equipped with the pneumatic tamper, can tamp more ties than 8 or 10 men using picks and bars. Twelve of these pneumatic tamping plants are now in use on the railroad named. Their adoption was the result of a careful test of their worth.

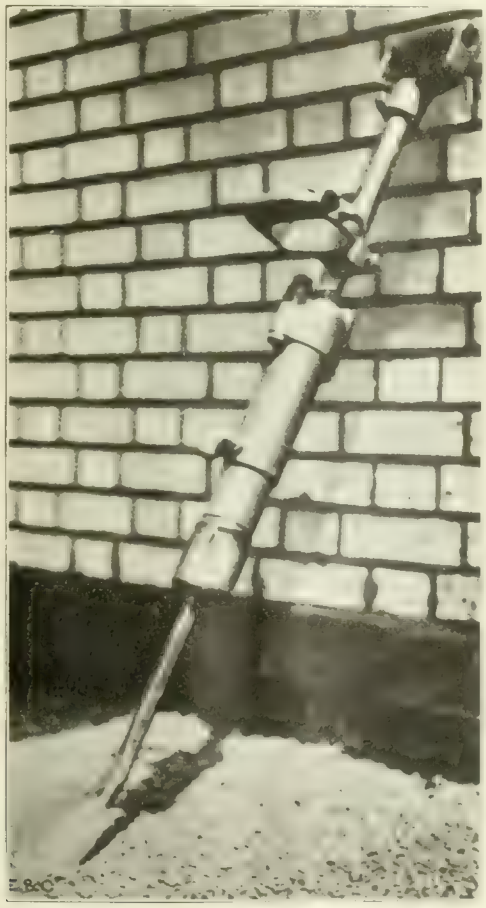


Fig. 1. View of Imperial Pneumatic Tie Tamper.

A stability test was made on a stretch of track 1,600 ft. long across the Hackensack Meadows in 1913. Half of the test section was tamped by the usual hand methods and half by the pneumatic tamper here described. At this point, owing to yielding foundation, it is difficult to maintain the track in proper surface. After six months of service under heavy traffic the maximum settlement of hand tamped ties was .116 ft. and of the machine tamped was .063. The corresponding minimum figures were .018 and .004, and the figures for average settlement were .067 for hand and .033 for machine tamped ties respectively. A year's cost record for one pneumatic tamper showed an average cost per tie tamped of $0.026.

The complete pneumatic tie tamping outfit consists of two tie tamping machines, an air compressor, direct connected with a gasoline engine which is mounted on a car fitted with flanged wheels, standard gage. The car is self-propelled, geared for a speed of 12 to 15 miles per hour; has a capacity of carrying 12 men or for supplying sufficient air at the proper pressure for running two of the tie tamping machines continuously. The car is also fitted with cross trucks having double flanged wheels so that it can be easily removed from or placed on the track by four men. Each side of the car is raised in turn 2 or 3 ins. and temporary tracks made of 2x4-in. scantlings are slipped under the derailing wheels at either end of the car and the car is rolled to the outside of the main track. The time required to remove the car in this manner with four men is about 45 seconds. The car, fully equipped weighs about 1,740 lbs. The car is also equipped with a fuel tank, water cooling radiator, air receiver, tool box, and with a housing which covers the entire mechanism so that it can be locked up when the plant is not in use. The weight of the tamper, including the tamping bar is 37½ lbs. A 600-ft. length of ¾-in. air supply hose has been satisfactorily used.

The tie tamper here described is known as the Imperial Pneumatic Tie Tamper and is manufactured and sold by the Ingersoll-Rand Co., 11 Broadway, New York City.

A Concrete Mixer and Atomizer.

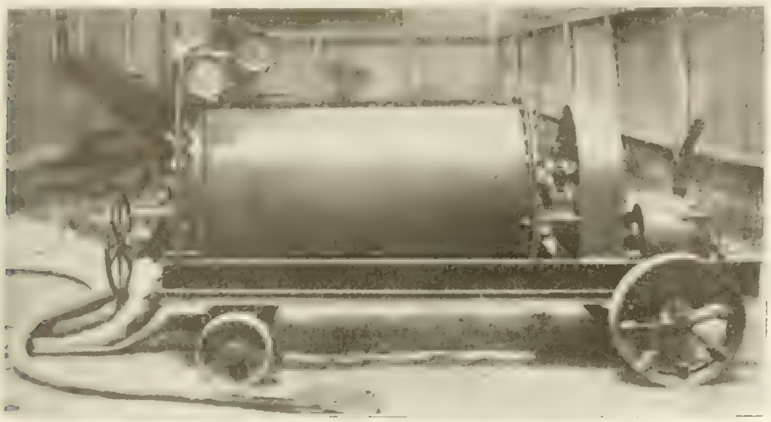
(Contributed.)

The concrete atomizer will mix cement, sand, broken stone or pebbles and water in any desired proportion and project the wet concrete upon the work. For this purpose compressed air is used at from 40 to 50 pounds pressure. The concrete adheres strongly to old concrete, masonry and brickwork or to structural steel or wooden buildings covered with coarse wire netting. The aggregate used should be suitable broken stone or pebbles of a size which will pass through a screen of 1-inch mesh with No. 12 wire. Special machines can be supplied to handle stone up to 1½ ins.

When aggregate is used and the stream of concrete is first sprayed upon the walls or floors, the pebbles will bound away and can be collected and used a second time. But after from ¼ in. to ½ in. of cement mortar has been deposited on the work, the pebbles will imbed themselves and thereafter, no part of the material is wasted, provided the proper amount of water is used and the hose nozzle is handled in accordance with directions. The amount of water can be accurately proportioned to the work and the materials thoroughly mixed; the product is therefore uniform. The use of pebbles or broken stone gives very great strength and density to the work since the blows delivered by these projectiles thoroughly tamp the concrete and fill all voids.

Fig. 2. View of Pair of Pneumatic Tie Tampers in Operation.

The No. 2 size of machine, shown in Fig. 1, [illegible] chute closed by a sliding door. A central shaft carrying mixing blades and scraping devices is driven by a motor or air engine. After a batch is thoroughly mixed the air is admitted through the valve P. The air valve A is opened and a stream of compressed air released through the hose. The delivery valve



Machine for Mixing, Atomizing and Placing Concrete.

P is then opened and the concrete is forced out in an atomized stream which adheres to the work wherever it strikes. When the mixing cylinder is emptied, the air valve A is closed and the fall of pressure releases the door for a fresh charge. The delivery valve is self-clearing and the mixer shell and paddles are kept free from clogging by special devices.

The same machine can be used with dry sand or pebbles as a sand blast to clean paint, rust or dirt from the surface of old work. It can also be used with plaster or cement mortar, whitewash or paint, since in each case with the proper adjustment, the sprayed material will adhere to the work and can be evenly applied. The patent expansible nozzle gives the operator complete control of discharge rate and prevents clogging. Special hose connections are furnished which do not cause trouble by reducing the inner diameter

Portable Device for Loading Wagons from Stock Piles.

[illegible] grout ejector under high pressure to fill [illegible] using grout mixed to the consistency of cream piped into the crack.

The machine is very strongly made, has but few moving parts and is very easily handled. Mounted on a metal truck, it weighs [illegible] lbs. and [illegible] with one level-headed man at the nozzle, one at the machine and two or three laborers for handling the material. Where a troweled or floated surface is desired, one or two good masons will be needed, but a skillful nozzleman can produce a very smooth surface by his control of the jet. It requires about 250 cu. ft. of free air per minute at 40 to 50 lbs. pressure when operated at full capacity. For out-of-door work with the proper apparatus, steam instead of compressed air can be used to atomize the concrete. The concrete atomizer can be purchased outright or can be leased with privilege of applying rental on price. The special concrete hose used with the machine is not rented as it can be easily injured by careless handling; the wear upon it is very slight. It is marketed by Harold P. Brown, 124 Liberty Street, New York.

Device for Loading Wagons From Stock Piles.

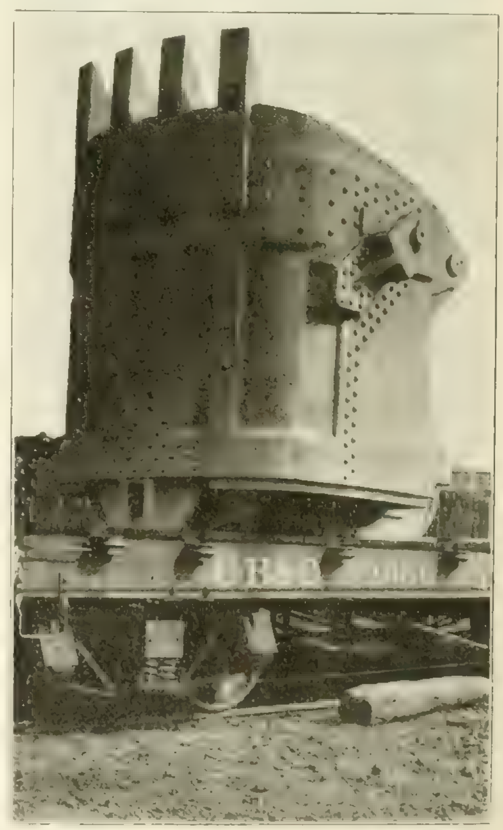
A wagon loader which weighs only 1,000 lbs. and can be moved about by one man is illustrated here. The operation of the loader is clearly indicated by the illustration. A noteworthy feature is the provision when desired of a chute with a screen bottom, which enables the material loading to be screened at the same time. In handling gravel for concrete this is a practice that is frequently desirable. When screening is not desired an adjustable plate is provided which fits over the screen and transforms the screen chute into a plain chute. The chute mouth is 10 ft. above the ground. The illustration shows the original device which has been improved by substituting the top construction indicated by dotted lines instead of the square top shown. This loader is made by the Jersey Wagon Loader Co., 95 Liberty St., New York City.

Ten Cubic Yard Manganese Steel Dippers for Panama Canal Dredges.

(Contributed.)

Two manganese steel dippers of 10 cu. yds. capacity and weighing approximately 37,600 lbs. each, were recently purchased by the government to be used on the new dipper dredges "Paraiso" and "Gamboa" on the Panama Canal. These are known as the "Missabe" type dippers; are patented, and were manufactured by the Edgar Allen American Manganese Steel Co. for the Bucyrus Company. They are the largest manganese steel dippers ever made. The dippers are of manganese steel construction throughout with the exception, of course, of the bolts, nuts and rivets. The overall dimensions are 10½x9x9 ft. The lips of the dippers are 3¼ ins. thick; the fronts are 3¼ ins. thick underneath the teeth; 1½ ins. thick between the teeth and 3¼ ins. thick at the bottom band; the backs are 1¼ ins. thick at the sides and 3¼ ins. thick at the bottom bands.

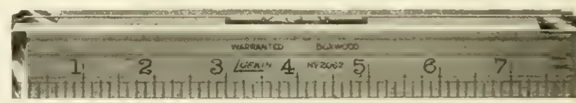
It will be noted from the illustration that there are few rivets as compared with a dipper of the built-up type. The body of the dipper consists of but two pieces, the front and back castings. The lap-joints at the sides make the dipper very rigid and relieve the rivets from strain. The bail brackets are attached to the front casting at an angle conforming to the line of pull on the bail, which throws the strain of the pull directly on the front casting. Shoulders or offsets to relieve the strain on rivets are provided throughout the dipper.



Manganese Steel Dipper for New Dredge on Panama Canal.

A Pocket Rule and Level.

The illustration shows a combination folding pocket rule and level; the rule is 2 ft. of boxwood. The three 8-in. rule-sections are each 1 in. wide, 3/16 in. thick, and are united by substantial brass joints. The rule is fully graduated its entire length on both sides, one side inches to eighths, and the other side to sixteenths. The spirit level is neatly set into and flush with the upper edge of the middle



Combination Folding Rule and Level.

section of the rule where most convenient for use and where it is securely protected by the two outer sections of the rule, which fold against it on either side when closed for carrying in the pocket. The combination rule and level being light and but 8 ins. long when folded, is conveniently carried in the pocket. Closing pins especially designed for the purpose hold the sections of the rule in proper alignment, insuring a perfect bearing surface. The accuracy of the rule and level is guaranteed. The article is new in every sense of the word and both as rule and level is a practical tool. The Lufkin Rule Co. of Saginaw, Mich., are its manufacturers and patents are pending.

The St. Louis public school system is being extended to give night courses for men employed in the mechanical trades, covering patternmaking, foundry practice, cabinetmaking, mechanical drawing, etc.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design and to Methods and Cost of Construction

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Increasing the Length of Life of Wooden Stave Pipe.

In this issue we publish the second article in the series pertaining to wooden stave pipe. The article is entitled: The Durability of Wooden Stave Pipe. In discussing durability our contributor deals with both the staves and bands. The durability of staves is discussed primarily with respect to the kinds of wood of which they commonly are made, to their design, to the agencies of decay and to the depth and nature of the covering over the completed pipe line. The durability of the bands is considered with special reference to the material of which they are made, to their coating and to their design. The decay of staves is discussed further with respect to the influence on durability of the location of the pipe line, to the characteristics of the soil in which the pipe is embedded, to the care with which the line is constructed and to the degree of saturation of the staves.

Mr. Swickard enumerates and fully discusses the factors which tend to increase the length of life of wooden pipe lines and their component parts and also considers, in a very concise manner, the causes which contribute to the failure of pipe of this type. He shows how the former may be enlisted by the engineer who seeks durability and how the latter may, in large measure, be avoided by competent selection of material, design and construction.

Numerous illuminating examples of the failure of engineers to take advantage of the conditions which are known to inhibit the decaying of wooden staves and the rusting of the bands are cited. The article is of major importance to all who are charged with the responsibility of selecting a material of construction for water pipe lines and of designing and building such lines.

Influence of Highway Commissions on Bridge Work.

The preparation of standard designs for steel and concrete highway bridges, together with the supervision of the construction of a considerable number of such bridges, by the various state highway commissions, have already exerted a marked influence in raising the standards for highway bridge construction. The designs of several of the commissions are especially commendable in that they are sufficiently complete in all their details to serve as models to practicing engineers. In the old types of light highway bridges poor details were common, but the increased loadings and the appreciation of the need of better structures along our improved highways have resulted in better designed details. In making use of these standard designs engineers should bear in mind that they are not to be followed blindly, but are intended to be representative of good practice.

For culvert work it is possible to prepare standards which can be followed closely in the construction of like structures, provided the engineer exercises ordinary judgment in determining the size of opening required and also takes precautions to prevent scour. For bridges of considerable span, however, the substructure design is always a special problem, and it is in the foundation work that standard designs are likely to be of little value. There is too great a tendency on the part of engineers to select a type of bridge and then make it conform to existing conditions, instead of first investigating topographic and subsoil conditions and then selecting the type of bridge which will meet these conditions. Although it requires time and patience to determine accurately the subsoil conditions and the volume of water for which provision must be made, a careful study of bridge failures will show conclusively that an ultimate saving will be made by increased attention to this part of the work. Shallow spread footings are often used in soils where even a casual examination would show the probability of danger from rapid scour; in other cases pile foundations are used where rip-rapping or a small expenditure in straightening the channel would render the use of piles unnecessary. Due to the tendency to make a particular type and span of bridge fit the site, piers and abutments are often located without reference either to economy or to permanence of construction. Although it is the experience of most engineers that the majority of highway bridge failures is due to the undermining of piers and abutments there are still numerous evidences of neglect to exercise ordinary judgment in designing and constructing foundations. The decision as to the depth to which foundations are to be carried too often is left to the inspector, instead of to the engineer who designed the bridge and should superintend its construction.

Some of the highway commissions are now issuing their bridge plans on a good quality of white paper in the form of small sheets. This practice is to be commended, as it enables the plans and specifications to be bound in a single volume, and thus furnishes an incentive to the contractor to study both more carefully in their relation to each other. This is an advance in the right direction, and when engineers will practice writing specifications which apply only to the particular job at hand there will be less criticism of contractors for not following more closely the plans and specifications. It is gratifying to note that the latest specifications of some of the more progressive highway commissions show a decided tendency to break away from the old stereotyped form of specification and to omit non-essentials. The carefully prepared instructions to employees which some commissions are now issuing contain data which will prove of considerable value to practicing engineers and to contractors. More attention to the work of these commissions by engineers and contractors in general will not only result in direct benefit to them, but will also act as an incentive to the commissions to set higher standards for future work.

Some Difficulties Involved in the Development of an Economic Theory of Highways.

From the earliest periods the use of the public highway has been considered an inherent right of the citizen. Conversely, the right of use implies that each individual should be supplied with a road to travel and the provision of this road is a public work to be performed by the state. But from the standpoint of economic location, construction and maintenance of highways only those roads should be provided that produce a profitable return on the investment.

There exists now and always has existed a contention as to relation of the individual to the state. On the one hand, it is thought that the welfare of the state is supreme and that individual injury or benefit should be subordinated to the well being of the majority. On the other hand, while the importance of the state is admitted, it is believed that each individual should share equally in the benefits of the government and works undertaken by the state for the welfare of its citizens. Under the latter premise it is almost impossible to develop an economic theory of highways without making it hopelessly involved and therefore of little practical value. Under the former premise, when a sufficient number of data have been obtained, it should be possible to formulate certain principles the observance of which in the location, construction and maintenance of roads, will benefit the majority of the people.

The proposition is axiomatic that only those public works should be undertaken that will yield a profitable return on the investment. But what constitutes a profitable return on the investment? Can it be measured in dollars and cents; if so, under what conditions? It is self-evident that each individual cannot share alike in the benefits of road improvement and, therefore, it would seem the return must be measured by the increased welfare of the people as a whole. Granting this fact, what conditions must be investigated and what data secured for use in measuring "profitable return"?

Several thoughts occur to the writer. Social conditions of wealth, type of industry, intensity of settlement and urgent needs of public welfare vary so widely in different sections of the nation, state and even county that any attempt to generalize, or average the problems of various localities, can lead to nothing but fallacious conclusions. The problem of each locality must be the unit of analysis. The present status of the returns from public investments must be determined in each locality, i. e., the cost of hauling farm produce and goods distributed from the cities and the value of time and convenience of travel estimated. The last-named element must be determined indirectly from, say, the economic efficiency of the people as determined from the returns on investments in manufacturing enterprizes, health and educational statistics, and other factors that might furnish a means of comparison with other communities and the improvement in welfare of the community in question. Also, a study of the effect upon costs of hauling, the radius of hauling, changes in types and distance of profitable hauling and the effect upon public intercourse and communication should be undertaken for various improved roads. With these data the fundamentals of an economic theory may possibly be established, the principles of which may be applied to any problem to determine approximately the "profitable return" to be expected from any investment in highways—as measured by the yardstick money.

It is believed an economic theory of pavements will be developed before many years have past. The elements involved are less in number and admit of more accurate determination than those of the general problem of road economics. Excluding the cost of hauling and the value of increased speed and convenience of travel an economic pavement is determined by first cost, maintenance cost and the effect of traffic. Reliable data concerning these factors are being obtained and results of some accuracy will undoubtedly be derived from them.

The difficulties mentioned are a few of those confronting the road engineer of the present day in the study of road economics. The field for original research is vast and the opportunities for accomplishing work of exceeding value to society are inspiring.

BUILDINGS

Formulas and Data for Designing Eccentric Riveted Connections.

The trial method of determining the number of rivets required in an eccentric riveted connection often results in several solutions before a satisfactory connection is designed. On account of the variables involved the deduction of a general formula is a long and tedious process, but where many connections are to be designed a formula is desirable. In a paper published in the Columbia School of Mines Quarterly, Joseph Di Stasio has derived a general formula, together with some formulas for special cases, and has given some data to facilitate the designing of eccentric riveted connection, from which we have abstracted the following:

FORMULAS.

General Formula.—The safe load on any eccentric riveted connection in which the rivets are arranged in horizontal and vertical rows may be found from the formula,

$$W = \frac{Rn}{\sqrt{1 + 2ek + e^2(k^2 + q^2)}} \quad \ldots (1),$$

in which W is dependent upon the variables e, n, d, r, and p. In this formula,

R = maximum safe value of one rivet in either shear or bearing,

n = total number of rivets,

e = eccentricity of load from center of the group of rivets,

$$k = \frac{6dn(r-1)}{d^2r(r^2-1) + p^2(n^2-r^2)},$$

$$q = \frac{6rp(n-r)}{d^2r(r^2-1) + p^2(n^2-r^2)},$$

d = distance between vertical rows of rivets,

r = number of vertical rows of rivets, and

p = pitch of rivets in any vertical row.

When $q = 0$.—In some special cases, when $q = 0$, i. e., when $n = r$, formula (1) becomes,

$$W = \frac{Rn}{\sqrt{(ek+1)^2 + e^2q^2}} \quad \ldots (2).$$

Single Horizontal Row of Rivets.—When all the rivets are placed in a single horizontal row, then $n = r$ and $p = 0$. Therefore $k = \frac{6}{d(n+1)}$ and $q = 0$.

Substituting these values in equation (1) and squaring, we have,

$$W^2 = \frac{R^2n^2}{\left[\frac{6e}{d(n+1)} + 1\right]^2}$$

from which,

$$W = \frac{Rnd(n+1)}{6e + d(n+1)} \quad .(3).$$

Single Vertical Row of Rivets.—When all the rivets are placed in a single vertical row, $d = 0$ and $r = 1$. Therefore $k = 0$ and $q = \frac{6}{p(n+1)}$.

Substituting these values in equation (1) and squaring, we have,

$$W^2 = \frac{R^2n^2}{\frac{36e^2}{p^2(n+1)^2} + 1}$$

from which,

$$W = \frac{Rn}{\sqrt{\frac{36e^2}{p^2(n+1)^2} + 1}} \quad ..(4).$$

Beam Fixed at Both Ends—Uniformly Loaded.—If we have a rigid connection, such as exists when a beam is rigidly fixed at both ends by riveting, even though the reaction may be assumed to act at the center of gravity of the groups of rivets, still the rivets will be subject to a bending stress in addition to the direct stress, the intensity of this bending stress depending upon the degree of fixedness of the connection and the subsequent deflection of the beam under loading. The safe method of design when the ends are rigidly fixed and the beam is uniformly loaded is indicated by the following:

Negative bending moment at end of beam $= \frac{1}{12} wL^2 = \frac{PL}{12}$, where w = load per linear foot of beam, L = length of beam, and P = total load on beam. In the general formula, the eccentric moment is indicated by We.

$We = 12\,(PL \div 12)$ in.-lbs.

But $W = \frac{P}{2}$, therefore $We = 2WL$, or e, in inches, $= 2L$, in feet.

Substituting for e in formula (1) its value $2L$, we have,

$$W = \frac{Rn}{\sqrt{1 + 4Lk + 4L^2(k^2 + q^2)}} \quad \ldots (5).$$

For any other kind of loading a similar procedure can be used.

TABULAR DATA.

Table I (see page 513) gives the safe eccentric loads, for various eccentricities, for ¾-in. rivets spaced 3 ins. apart vertically and for various distances between the vertical rows of rivets; the value of R is taken at 4,400, which is the allowable single shear on a ¾-in. rivet at 10,000 lbs. per square inch. In this table the safe eccentric loads are given in thousand pounds.

The Use of Reinforced Cinder Concrete Floors in Buildings.

The extensive use of cinder concrete in the floors of high commercial buildings, particularly in our Eastern cities, has led to a demand for definite data on the value of this material for reinforced floor slabs. In New York, where about 90 per cent of the fireproof floors are cinder concrete, actual tests of cinder concrete floor slabs have been required by the building department as a basis for the acceptance or rejection of such construction. The attempt to place the design of such floors on a more rational basis by adopting certain allowable stresses has been made by some engineers. In our issue of May 27, 1914, we gave some pertinent data on this subject, and in this article extracts from the discussions of several engineers and architects are given, the data being taken from the discussion of a paper "Cinder Concrete Floors," by Guy B. Waite, in Proceedings, American Society of Civil Engineers, vol XL, p. 1963.

DISCUSSION BY A. W. BUELL.

When safe working stresses for cinder concrete have been correctly determined for the various specified conditions encountered in practice it will, without much doubt, become clearly apparent that there is little, if any, economy in its use for structural elements subject to stress, notwithstanding its lower weight per unit of volume as compared with stone concrete.

The writer regrets that Mr. Waite has not taken up the question of the corrosion of embedded steel, as it is of vital importance in a permanent structure, especially as the records of some experiments and considerable experience have shown bad results.

Although a satisfactory series of experiments from which to deduce safe unit stresses for cinder concrete would be a valuable addition to our knowledge of the subject, it would seem advisable first to determine by experiment and experience whether the material is suitable for permanent structures and whether the reinforcing metal is reasonably safe from corrosion. Until this is done it will hardly be safe to specify cinder concrete for reinforced members or to adopt it as an acceptable material of engineering or of construction—except as a filler—no matter how satisfactory the experimental determinations of its strength may be.

In 1907 it was stated by William H. Fox that the results of experience and experiment show serious corrosion in almost all cases, and recommends that with cinder concrete no mixture less rich than 1:1:3, or one still richer in cement, should be used. In a report of a committee read before the Structural Association of San Francisco it was stated that metal encased in cinder concrete had been corroded, and the committee advised a revision of the code, to exclude the use of cinder concrete. Mr. Fox reports a set of experiments at the Thayer School, with the following conclusion:

> With but one exception one or more of the three steel pieces in each specimen showed unmistakable signs of corrosion. Apparently it made no difference how the concrete was mixed—wet or dry, tamped or untamped; whether the steam or water treatment was used, the result was the same—rust streaks and spots were found, the difference in the amount of corrosion being imperceptible.

These records seem to make a case against cinders for reinforced concrete that will require something more than explanation, and a good deal of substantial evidence to the contrary, before conservative engineers and architects can safely adopt this material. The advocates of cinder concrete should accept the task of making the necessary demonstrations.

DISCUSSION BY CHARLES C. HURLBUT.

Good cinders are sometimes hard to obtain, even with the best intentions, and on many occasions the writer has rejected cinders on the job, which otherwise would have been used in the floor construction. It may be noted in passing that the floor construction in question had been duly "passed" by the Building Department, but it is safe to say that the test on which the approval was based was made with very different cinders from those rejected, illustrating Mr. Waite's objection that the Building Department tests have small relation to what may be expected of any particular floor. The cinders rejected in some cases were full of unburned coal and in other cases contained little but fine ashes. Wherever cinders are used they should be subject to rigid inspection. In the matter of unburned coal in cinders, although it is the writer's practice not to accept cinders containing more than the small necessary proportion, it is his opinion that the danger from unburned coal in concrete is over-estimated. This opinion is based on a rather crude test made on a partition built of 2-in. cinder concrete blocks. The blocks were literally filled with unburned coal. A small covered enclosure was built and subjected for more than an hour to a very hot wood fire. The coal on the surface was reduced to cinder, but all below the surface seemed to be entirely unaffected.

The objection has been raised, in designing cinder concrete by formula instead of by test, that the material is too variable. This argument does not seem to be well founded. Given a certain system of reinforcement the strength of any floor slab will vary from that developed by a full-sized test sample in just the same degree that the materials in it vary from those in the slab tested. It is just as proper, therefore, to designate a safe unit compressive strength and other properties for cinder concrete as it is to assume a safe slab strength based on a test, and far more rational. As a matter of fact it is a far safer method, for the average strength of cinder concrete can be determined with some approximation to the

TABLE I.—SAFE ECCENTRIC LOADS, IN THOUSAND POUNDS, FOR ¾-IN. RIVETS.

(Values are for p = 2 ins. and R = 4.4.)

Notation:
n = total number of rivets.
r = number of vertical rows of rivets.
p = pitch of rivets in vertical rows, in inches.
d = distance between vertical row of rivets, in inches.

Notation:
e = eccentricity of load from center of group of rivets, in inches.
R = maximum safe value of one rivet in either shear or bearing, in thousand pounds.
W = safe eccentric load, in thousand pounds.

For these values of e, in inches—

TWO VERTICAL ROWS OF RIVETS.

For these values of d, in inches—

n	e = 0	1½					3					6					9				
d =	3, 6, 9, 12, 15.	3	6	9	12	15	3	6	9	12	15	3	6	9	12	15	3	6	9	12	15
2	8.8	4.4	5.8	6.6	7.1	7.3	2.9	4.4	5.2	5.8	6.3	1.7	2.9	3.7	4.4	4.9	1.2	2.2	2.9	3.5	4.0
4	17	11	12	13	14	14	7.9	9.5	10	11	12	4.9	6.5	7.8	9.1	9.9	3.5	4.8	6.1	7.2	8.1
6	26	19	19	20	21	22	13	15	16	18	19	8.6	10	12	14	15	6.2	8.0	9.5	10	12
8	35	27	27	28	29	29	20	22	23	24	25	13	15	17	19	20	9.6	11	13	15	16
10	44	37	36	36	37	37	29	29	30	31	32	19	20	22	24	26	14	15	17	19	21
12	52	46	44	44	44	45	38	37	38	39	40	25	26	28	30	32	19	20	22	24	26
14	61	55	53	53	54	54	46	45	46	46	47	32	33	34	36	38	24	25	27	29	31
16	70	64	61	61	62	62	56	54	54	55	56	39	40	41	42	44	30	31	32	35	37
18	79	73	71	71	71	71	65	63	63	63	63	48	48	49	50	51	37	37	39	41	43
20	88	82	80	80	80	80	74	72	72	72	72	57	56	56	57	59	44	44	45	47	49
22	96	92	88	88	88	88	83	80	80	80	81	65	64	64	65	66	52	51	52	53	55
24	105	101	97	97	97	97	93	89	89	89	89	75	73	73	73	74	60	59	59	60	62

n	e = 12					15					18					21					24				
d =	3	6	9	12	15	3	6	9	12	15	3	6	9	12	15	3	6	9	12	15	3	6	9	12	15
2	1.0	1.7	2.4	2.9	3.4	0.8	1.4	2.0	2.5	2.9	0.7	1.2	1.7	2.2	2.6	0.6	1.1	1.5	1.9	2.3	0.5	1.0	1.4	1.7	2.1
4	2.7	3.9	5.0	6.0	6.9	2.2	3.2	4.3	5.2	5.9	1.9	2.8	3.7	4.5	5.3	1.6	2.4	3.3	4.0	4.7	1.4	2.1	2.9	3.6	4.2
6	4.8	6.3	7.9	9.2	10	3.9	5.3	6.6	7.9	9.1	3.3	4.5	5.7	6.9	8.1	2.9	3.9	5.9	6.2	7.2	2.5	3.5	4.5	5.5	6.5
8	7.4	9.1	10	12	14	6.0	7.6	9.2	10	12	5.1	6.5	8.0	9.4	10	4.4	5.6	7.0	8.3	9.7	3.9	5.0	6.3	7.5	8.8
10	11	12	14	16	18	9.0	10	12	14	16	7.6	8.8	10	12	14	6.6	7.6	9.1	10	12	5.8	6.8	8.1	9.7	11
12	15	16	18	20	22	12	13	15	17	19	10	11	13	15	17	9.1	10	11	13	15	8.0	8.9	10	11	13
14	19	20	22	24	26	15	17	18	20	23	13	14	16	18	20	11	12	14	16	18	10	11	12	14	16
16	24	25	27	29	31	20	21	23	25	27	17	18	19	21	23	14	15	17	19	21	13	13	15	17	19
18	30	30	32	34	36	24	25	27	29	31	21	21	23	25	27	18	18	20	22	24	16	16	18	20	22
20	36	36	37	39	42	29	30	31	33	36	25	26	27	29	31	22	23	24	26	28	20	20	21	23	26
22	42	42	43	45	47	35	35	37	39	41	30	30	32	34	36	26	26	28	30	32	23	23	25	27	29
24	49	49	50	51	53	41	41	42	44	46	35	35	36	38	40	31	31	32	34	36	27	27	28	30	33

THREE VERTICAL ROWS OF RIVETS.

For these values of d, in inches.

n	e = 0	1½				3				6				9				12				15				18				21				24			
d =	3, 4, 5, 6.	3	4	5	6	3	4	5	6	3	4	5	6	3	4	5	6	3	4	5	6	3	4	5	6	3	4	5	6	3	4	5	6	3	4	5	6
3	13	7.5	8.4	9.1	9.6	5.7	6.2	6.9	7.5	3.3	4.0	4.7	5.3	2.4	3.1	3.5	4.0	1.9	2.4	2.8	3.3	1.5	2.0	2.4	2.8	1.3	1.7	2.0	2.4	1.1	1.5	1.8	2.1	1.0	1.3	1.6	1.9
6	26	16	17	18	19	12	13	14	15	7.8	9.0	10	11	5.8	6.7	7.6	8.5	4.5	5.4	6.1	6.9	3.7	4.5	5.2	5.9	3.2	3.8	4.4	5.0	2.8	3.3	3.9	4.4	2.5	3.0	3.4	3.9
9	39	28	28	29	30	20	21	23	24	13	14	16	17	10	11	12	13	7.9	8.9	9.9	11	6.6	7.4	8.4	9.4	5.6	6.3	7.2	7.9	4.9	5.5	6.3	7.0	4.3	4.9	5.6	6.2
12	52	39	40	40	41	30	31	32	33	20	21	23	24	15	16	17	19	12	13	14	15	10	11	12	13	8.4	9.3	10	11	7.7	8.1	9.0	10	6.5	7.2	8.0	8.9
15	66	52	52	53	53	41	42	43	44	28	29	30	32	21	22	23	25	16	17	19	20	14	15	16	17	11	12	13	14	10	11	12	13	9.1	9.9	10	11
18	79	66	66	66	66	53	53	54	55	37	38	39	40	28	29	30	32	22	23	24	26	18	19	20	22	16	17	18	19	14	14	15	16	12	13	14	15
21	92	79	79	79	79	66	66	67	67	47	48	49	50	36	37	38	39	29	30	31	32	24	25	26	27	20	21	22	23	17	18	19	20	15	16	17	18
24	105	92	92	92	92	79	79	79	79	58	58	59	60	45	45	46	47	36	37	38	39	30	31	32	33	25	26	27	28	22	23	24	25	19	20	21	22
27	118	106	106	106	106	92	92	92	92	69	69	70	71	55	55	56	56	44	44	45	46	36	37	38	39	31	32	33	34	27	28	29	30	24	25	26	27
30	132	119	119	119	119	106	106	106	106	82	82	82	83	65	65	65	66	53	53	54	55	44	45	46	47	38	38	39	40	33	34	35	36	29	30	31	32
33	145	132	132	132	132	119	119	119	119	94	94	94	95	76	76	76	77	62	62	63	64	52	52	53	54	45	45	46	47	39	39	40	41	35	35	36	37
36	158	146	146	146	146	133	133	133	133	107	107	107	107	87	87	87	87	72	72	72	73	60	61	62	62	52	53	54	54	46	46	47	48	41	41	42	42

FOUR VERTICAL ROWS OF RIVETS.

For these values of d, in inches.

n	e = 0	1½				3				6				9				12				15				18				21				24			
d =	3, 4, 5, 6.	3	4	5	6	3	4	5	6	3	4	5	6	3	4	5	6	3	4	5	6	3	4	5	6	3	4	5	6	3	4	5	6	3	4	5	6
4	17	11	12	13	13	8.0	9.2	10	11	5.2	6.3	7.2	8.0	3.8	4.7	5.5	6.3	3.0	3.8	4.5	5.2	2.5	3.2	3.8	4.9	2.1	2.7	3.3	3.8	1.9	2.4	2.9	3.4	1.7	2.1	2.6	3.0
8	[illegible]	[illegible]	25	26	27	17	19	21	22	11	13	15	16	8.5	10	11	12	6.7	8.2	9.4	10	5.6	6.9	8.0	9.0	4.8	5.9	6.9	7.9	4.2	5.2	6.1	7.0	3.7	4.6	5.4	6.2
12	52	37	38	40	43	28	30	32	34	18	21	23	25	14	16	18	20	11	13	14	16	9.4	11	12	14	8.0	9.1	10	12	7.0	8.3	9.6	10	6.2	7.4	8.5	9.7
16	[illegible]	[illegible]	54	55	56	41	43	45	46	28	30	32	34	21	23	25	27	17	19	21	23	14	15	17	19	12	13	15	17	10	12	13	15	9.4	10	12	13
20	88	[illegible]	70	71	72	55	57	58	60	38	40	42	45	29	31	33	35	23	25	27	29	19	21	23	25	16	18	20	22	14	16	17	19	13	14	16	17
24	105	86	86	87	88	71	72	73	74	50	52	54	56	38	40	42	44	31	33	35	37	25	27	29	31	22	24	25	27	19	20	22	24	17	18	20	22
28	123	104	104	104	105	87	87	88	89	63	65	66	68	48	50	52	55	39	41	43	45	32	34	36	39	28	30	32	34	24	26	28	30	21	23	25	27
32	[illegible]	[illegible]	[illegible]	122	122	103	104	105	105	77	78	79	80	60	62	63	65	48	50	52	54	40	42	44	46	34	36	38	40	30	32	34	36	27	29	30	32
36	[illegible]	139	139	139	139	122	122	122	122	92	93	94	95	70	73	75	77	58	60	62	65	49	51	53	55	42	44	46	48	37	39	41	43	33	35	36	38
40	176	157	157	157	157	139	139	139	139	108	108	109	110	86	86	87	88	70	71	73	75	59	60	62	64	51	52	54	56	44	46	48	50	40	41	43	45
44	193	[illegible]	175	[illegible]	175	156	156	156	156	124	124	121	125	100	100	101	102	82	83	84	86	69	70	71	73	60	61	62	64	51	53	55	57	48	49	50	52
48	211	193	193	193	193	174	174	174	174	140	140	110	141	111	114	115	116	94	95	96	98	80	81	83	85	69	71	72	71	61	63	64	66	55	56	57	59

SINGLE VERTICAL ROW OF RIVETS

n	e = 0	1½	3	6	9	12	15	18	21	24
2	8.8	6.2	3.9	2.1	1.4	1.1	0.9	0.7	0.6	0.5
3	13	10	7.3	4.1	2.8	2.1	1.7	1.4	1.2	1.1
4	17	15	11	6.7	4.7	3.6	2.9	2.4	2.1	1.8
5	22	19	15	9.8	6.9	5.3	4.3	3.6	3.1	2.7
6	26	24	20	13	9.5	7.4	6.1	5.0	4.3	3.8
7	[illegible]	28	24	17	12	9.8	7.9	6.7	5.8	5.1
8	[illegible]	33	29	21	15	12	10	8.5	7.5	6.4
9	39	37	34	25	19	15	12	10	9.2	8.1
10	44	42	38	29	23	18	15	12	11	9.8
11	48	47	43	34	27	21	18	15	13	11
12	52	51	48	38	31	25	21	18	15	13
13	57	56	53	43	35	29	24	21	18	16
14	61	60	57	48	39	32	27	21	21	18
15	66	65	62	53	44	36	31	27	24	21

truth by taking a large number of tests on varying grades of cinders. On the other hand a test made on a slab proves little beyond the slab tested. Only one test is made on each "system," and it is safe to say that the cinders entering into these tests are the best that can be obtained. The writer has witnessed a number of these tests; he has seen some of the test slabs built, and knows that the cinders were better than the average used throughout the city. The charge is not made that any of the tests have been "faked," or even that the materials used were not such as can be obtained in the open market, but merely that the tests of which the speaker has knowledge were made with exceptionally clean cinders, which are not always used in actual construction. No proof as to the accuracy of an engineering formula would be deemed conclusive if based on a single test, yet a single test with an admittedly variable material is here used as a pattern to be followed without variation. It would be possible to define a standard cinder aggregate, the properties of which would not vary beyond reasonable limits, no more, for instance, than timber varies, or some other materials the use of which is based on rational formulas. The proportion of fine ash can readily be determined with a sieve of some standard mesh, and the percentage of unburned material can be determined with sufficient accuracy by means available almost anywhere. These are the two principal uncertain and variable elements in cinders as used in concrete.

A serious objection that has been raised to the use of cinders is their alleged tendency to corrode metal with which they come in contact, and it is to be regretted that Mr. Waite did not take up this phase of the question. It has been the writer's opinion that a cinder concrete, if mixed not leaner than 1:2½:5 and placed rather wet, would not injure pipes or steel reinforcement embedded in it. No cases of corrosion clearly attributable to the cinders have come to his attention under these conditions. There are many cases, however, where pipes embedded in a weak cinder fill have been destroyed, and the presumption has been that the cinders were to blame. It is the writer's practice to specify and enforce rigidly the requirement that all pipes embedded in the floor construction or fill, where cinders are involved, are to be solidly encased in cement mortar made with 1 part cement to 2 parts sand. The Lackawanna Railroad has experienced considerable difficulty with the corrosion of pipes in its stations which, in some cases at least, the engineers have attributed to the effects of contact with cinder concrete. Owing to this belief, they are now prohibiting cinder concrete in every form. In this case at least some of the trouble was subsequently found to be the result of electrolysis. How much of the corrosion, if any, was actually due to cinders, it would be hard to say. It is quite possible that in other cases of damage to pipes cinder concrete has been made to bear the blame which should have been attributed to stray currents of electricity.

[illegible]

About sixteen or seventeen years ago, cinder concrete, having been officially tested and approved by the New York building department, was introduced. That it has been a success in its limited field of application time has shown conclusively. The method of testing originally devised, and still followed, does not give accurate results, but it was the best that knowledge then available afforded.

For several years after its introduction the cinder concrete industry was dominated by reputable individuals and companies of financial responsibility who generally controlled a patented method of reinforcing which was not available to others in the trade. It was their custom, as a rule, to arrange for a steady supply of cinders from fairly reliable sources. The patents referred to cover the forms of reinforcing bars and rods, either plain or deformed, with spacers, clips, and hangers for insuring the proper placing of the reinforcement and holding the concrete protecting the bottom flanges of the steel floor-beams and girders. There is also a rectangular wire mesh with welded intersections, which is one of the best types of reinforcement.

Each concern conducted a series of tests under the direction of the Building Department, and obtained approvals on forms of construction which gave the architects a range of choice sufficiently wide to cover generally all the requirements of practice. The conditions in the trade were fairly stable, with reasonable competition, a good grade of cinders, and responsible concerns to deal with.

Five or six years ago a subsidiary of the Steel Corporation began the manufacture of a triangular-mesh, wire reinforcement, which, having been tested and approved in slabs of various spans and for various quantities of steel, was placed on the market. This is sold to any one having the means to pay for it, and it is so cheap that, in many instances, as a result of the ensuing sharp competition, the proprietors are obliged to use it instead of their own methods of reinforcement. No criticism of this form of reinforcement is here implied, for it is one of the best in use today.

The trade is not now in such a satisfactory condition as formerly, for the reason that an irresponsible, unskilled, and, too frequently, careless element has appeared, which, through ignorance or willingness to assume risks, underbids its more conservative and responsible competitors. It is practically impossible for the architect to induce the average owner (especially the speculator) to discriminate between the two elements in the trade. Cinder concrete is cinder concrete to him, and it is difficult for him to see beyond the dollars apparently saved by the acceptance of an unduly low estimate.

In times of normal building activity, it is difficult and sometimes impossible for the new element in the trade to obtain cinders of good quality; consequently, they are often taken from household refuse at the dumps, and contain all sorts of foreign matter adversely affecting their quality and the strength of the concrete.

The foregoing has been written for the purpose of showing clearly the influences which have resulted in conditions as they are today.

For some time the method of arbitrary tests imposed by the Bureau of Buildings has caused dissatisfaction, not only in the trade, but among architects, because they require much time and are expensive, and because the approval is limited to the exact form tested, no variation whatever being allowed. This means that a firm desiring to use a number of forms of construction must take the time for, and incur the expense of, a test on each one, no matter how slight the difference may be.

This condition has naturally led to the desire, on the part of the cinder concrete interests, for a more rational and simple method of determining the strength of slabs, and the proposal to calculate their stability on the basis of unit stresses is the result. This proposal is logical and highly desirable, if it can be done with safety, but there are points involved which would seem to make it unwise.

Until recently all slabs were tested under uniformly distributed loads, developing unbelievable carrying capacities. The only way to account for this is that the load arches itself, a large and indeterminate portion being transferred to the haunches. In fact, it would require great skill in placing the load to prevent it. We are in the dark, therefore, as to the true strength of the slabs we have used. Within the last two or three years slabs have been tested under concentrated loads at the middle.

As has been intimated, it is difficult to obtain cinders of anywhere near a standard quality.

The proper inspection of cinder concrete requires a superintendent continually on the work to see that the materials are of the proper kind, and that they are mixed and placed as they should be. It is practically impossible to persuade an owner to incur the expense of a clerk of the works, especially in commercial work. This is another valid reason for a strict law safeguarding the use of cinders.

The most uncertain of all the elements entering into the composition of any concrete, but more especially cinder concrete, is the so-called human element. Carelessness in or indifference to any one of the many precautions which must be observed in the mixing and placing of concrete may lead to failure.

If unit stresses are used in the calculation of cinder concrete slabs, they should be low enough to discount faulty cinders, carelessness, possible dishonesty, indifference, and lack of inspection. To accomplish this, the unit stresses must be so low that the cinder concrete interests themselves would object on the ground that the quantity of material required would be increased to such an extent that they could not compete successfully with other forms of floor construction. If unit stresses are used the cinders should be standardized; the law should require that they be washed and screened.

The fire-resisting qualities of cinder concrete have been developed so many times in tests and during actual fires that it would seem that the time has come when they may be accepted as proved and fire tests eliminated in the future.

The following recommendations are made:

(1) That the Bureau of Buildings conduct a series of tests, approximating as closely as possible commercial conditions in the mixing and placing.

(2) That the slabs be tested under concentrated loads at the middle of the spans, with various combinations of span lengths, thicknesses of slab, and areas of reinforcement.

(3) That the results of these tests be divided by a conservative factor of safety and the remainders be tabulated for use as the basis of slab design. For instance, if it is desired to use a slab 4 ins. thick, with a 6-ft. span, carrying an applied load of 150 lbs. per square foot, a given area of reinforcing steel per foot of width should be required.

(4) That a ruling be made requiring the washing and screening of all cinders used in all future buildings.

(5) That all existing approvals be revoked, after the results of the new tests are available.

The adoption of these recommendations would place the use of cinder concrete on a rational basis and enable those interested to proportion the slabs with a degree of freedom and safety hitherto unknown.

Several years ago A. N. Talbot embedded steel in cinder concrete and concluded that the corrosion was negligible, because, with the wet mixtures used, a film of cement was deposited around the steel which protected it against the action of any chemicals in the cinders tending to cause corrosion.

It is not safe to place unprotected pipes in the cinder fill over slabs. This can probably be explained by the fact that the mixture is usually 1:7 or more, and is placed quite dry, the consequence being that there is not sufficient water present to carry in suspension enough cement to coat the pipes. Piping thus placed should always be given a heavy coat of asphaltic or similar acid-resisting paint.

About the only knowledge we have of the effect of cinder concrete on reinforcing steel in actual construction is derived from the Pabst Building, which was removed (to make way for the Times Building) after having been occupied for seven years. Corrosion of the steel was found in a few places, but not to a degree affecting its strength. Pipes embedded in the cinder fill were badly corroded.

DISCUSSION BY A. L. A. HIMMELWRIGHT.

The great difficulty in evolving a rational theory of design is that the strength is not the only important feature of fire-proof construction; fire resistance and durability are even more important. Conditions which will produce the greatest strength generally result in the poorest fire resistance; that is to say, the farther away the reinforcing metal is from the neutral axis and the nearer it approaches the underside of the slab, the greater will be the strength; on the other hand, the nearer the reinforcing metal approaches the under side of the slab, the thinner will be the protective covering and the more quickly the reinforcing metal will be heated in a fire so as to become weak.

The question of durability is most important. Exposure to the elements, which is a necessary condition in practical building construction, always causes initial rusting. When the reinforcing metal is in the form of bars or rods of considerable size and section, the rusting is not important, and does not materially weaken the metal. When, however, the reinforcement is in the form of wire or small mesh made from light sheet-metal or similar material, the rusting affects the strength very materially by reducing the effective section, and when embedded in wet concrete it continues to rust in rainy weather and until the building is roofed over. In extreme cases such reinforcing metal under normal conditions becomes weakened by oxidation so that it retains only a fraction of the strength shown by the formula.

Density is another factor that has large importance in cinder concrete fireproofing. When stone aggregates are used and the concrete is thoroughly rammed so as to eliminate all voids the action of the concrete approaches more nearly to that of the solid stone of which the aggregates are composed, and it cracks much more readily when exposed to heat and sudden cooling than when the concrete is full of voids. This, in fact, is the principal reason that cinder concrete develops higher fire resistance than any concrete with stone aggregates. It is impossible to eliminate all the voids in the cinder aggregate by ramming, as such voids are always present in the aggregates themselves, to a greater or less extent, even if the cementing material is free from them.

Tests made by the writer conclusively demonstrated that the more porous the concrete the less likely it is to crack and fail under sudden and violent changes of temperature. Cinders crushed so as to pass a ¾-in. screen, made into concrete mixed in the proportion of 1:2:5, and simply spread over the forms and leveled and patted down with shovels in segmental arch construction, will develop ample strength to carry ordinary loads up to, say, 200 lbs., with a safety factor of 5. The material manipulated in this way will be extremely porous, light in weight, and will exceed in fire resistance any other material known to the writer, except possibly a concrete made from "tetzlonti," a porous lava rock which the writer has used in a number of buildings in the City of Mexico.

It is well known, of course, that density is desirable, as tending to prevent oxidation of the steel reinforcement, since oxidation takes place more rapidly where voids occur in the surrounding concrete; but this oxidation practically ceases when the concrete becomes perfectly dry after a building is enclosed and finished. This phase of the problem, therefore, is of minor importance, especially when the reinforcing metal is in the preferred form of rods or bars of considerable section.

The economic advantage of voids in securing lightness is much more important than the initial oxidation, but it has never received the attention it deserves. A concrete made and deposited as above described will weigh only from 70 to 75 lbs. per cubic foot, as against 95 to 100 lbs. when thoroughly rammed. This is a gain of 25 per cent in the dead load, in favor of economy, and, as above stated, ample strength is also obtained.

From what precedes it will be apparent that certain qualities and conditions which result in improved fire resistance, both in the form of the reinforcing metal and in the concrete, tend toward weakness, so that we have two conflicting elements in the design of cinder concrete floors which will forever prohibit the development of a practical rational formula of general application.

An important feature of this whole problem is the fact that it is a mistake to legislate or inaugurate regulations that are restrictive and tend toward the development and improvement of only a few forms of construction. The error of this, perhaps, is more plainly illustrated in the regulations governing theater construction than in anything else. All the building laws at the present time specify refinements of the conventional design for theaters, and absolutely prohibit other designs which might provide greater safety and result in larger seating capacity. It is always better, in preparing building codes and city ordinances, to specify results to be obtained rather than detailed specifications.

The best that can be done with the problem is to develop a theory and formula in accordance with the results of the usual and more common forms of cinder concrete floors, as proposed by Mr. Waite, and, in addition, make a separate provision for the determination by test of the strength of those special forms of construction to which the formula and rational theory do not apply. This course has recently been followed in the most up-to-date building codes, in order not to discourage or prohibit methods which aim at superior fire resistance, lightness, and other economies.

Cinders with any considerable quantity of soluble sulphates are entirely unsuited for concrete aggregates, and should be prohibited. A good quality is obtained from large steam boiler plants in which the cinders are cooled by spraying. They contain practically no soluble sulphates or silicates, and from 95 to 98 per cent of the fine material is grit fully equal to sand of good quality for the purpose of making concrete. When a building law requires screening, it can readily be understood that excellent material (the fine grit) is thus prohibited and wasted, which involves useless expense and a large loss in economy of construction. Locomotive cinders—on account of the coating of soot—and cinders from gas plants are also unsuitable for concrete. The extremely coarse clinker from bituminous coal is objectionable, as is, in fact, any coarse material.

To get uniform and satisfactory results, all cinders should be passed through rollers or crushers which will reduce them to a size that will pass a ¾-in. square mesh screen. If sand is added to the fine material, so as to make a mixture in the proportion of 1 part of high-grade Portland cement to 2 parts of fine material and sand, and 5 parts of crushed cinder aggregate, a most excellent fire-resisting concrete will result.

A good definition for a suitable quality of cinders in a building code is the following:

Cinders, when referred to in this code, shall be construed to mean the residue of coal consumed in large steam boiler plants, free from soluble sulphates and silicates, and in which the fine material is composed wholly of grit. (Note: —Ashes or cinders from ordinary coal stoves, ranges, hot-air furnaces, small steam boilers, or locomotive or gas-plant cinders, shall in no case be used for aggregates for cinder concrete.)

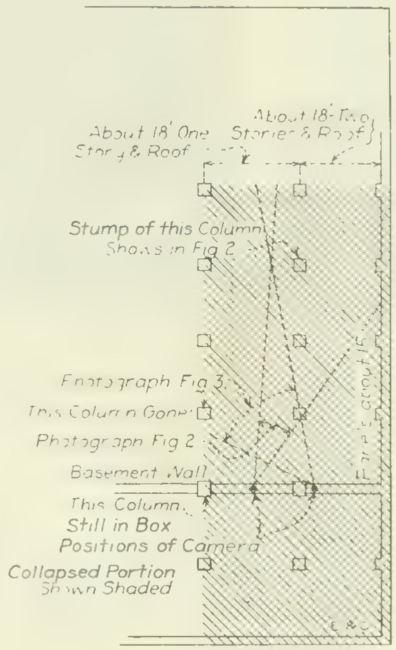


Fig. 1. Sketch Showing Extent of Failure of Theater and Arcade Building, Youngstown, Ohio, and Positions of Camera in Taking Views Shown in Figs. 2 and 3.

Comment on the Failure of the Theater and Arcade Building, Youngstown, Ohio.

To the Editors: On the afternoon of Oct. 26, 1914, there occurred, at Youngstown, Ohio, a partial collapse of the reinforced concrete portion of the new Theater and Arcade Building which was under construction for the Youngstown Hippodrome Co. The sketch in Fig. 1 shows roughly the extent of the wreck, as well as the positions from which the views shown in Figs. 2 and 3 were taken. These views were taken at noon on Oct. 27, and they show the condition of the building at that time.

The floors were ribbed construction using steel "Floretyles," the spans being about 15 ft. They were carried by reinforced concrete girders, the girders being reinforced with "Kahn" bars. The columns were square concrete columns having a small rod in each corner. Brick pilasters were provided where the girders rested in the tile-and-brick wall.

I am informed that the concrete of the roof was poured one week before the wreck. This would make the concrete in the first and second floors several weeks old. The collapse was sudden and without warning. Three men were killed and three were injured. Newspaper accounts are very indefinite as to where and how the wreck started, as might be expected, since the collapse was so sudden and so complete.

Of course there will be the usual investigation and the usual stereotyped conclusions. Several things are certain, and these can be written now as well as later. One of these is that cold weather had absolutely nothing to do with the failure, in spite of an editorial in a Pittsburgh newspaper the day after the occurrence calling loudly for reinforced concrete work to be stopped when cold weather arrives. Cold weather did arrive the day after the wreck, but for a long period before it the weather was ideal for concrete work. There were not even nightly frosts.

Wind had nothing to do with it, for the building is completely shielded from wind.

We Pennsylvania engineers are worked up over a proposed law to license engineers on the pretext that excluding the incompetent will diminish failures. There is apparently no ground for finding fault in this case with the competence of either architects, consulting engineer, or contractors. As I stated before the State Commission, which was responsible for this law, this failure would not change engineering designs one iota.

I do not hesitate to say that the one big cause behind this wreck was bad design, and I do not hesitate to place the blame on the structural engineering profession at large. This is just another of the countless wrecks due to abominable standards of design, standards that I have been publicly condemning for eight years, standards that no engineer has dared to defend publicly for a large portion of that time; nevertheless the "Joint Committee Report" has in that time twice sanctioned those standards.

A man on the job told me that they were pulling forms when the collapse took place. I surmise that if any forms were being pulled it was in the cellar. It is scarcely probable that they would be pulling forms from under week-old concrete girders. The photographs (Figs. 2 and 3) show that there are still remaining the props under that portion of the roof which did not collapse, as well as the props under the second-story girders, and a few of those under the first-story girders. But suppose they were pulling forms from under the ribs in the roof prematurely, why did not the second floor catch and hold the falling roof, at least to the extent of delaying a complete collapse? Why did not the girders and columns, relieved of their load, stand up?

There are visible two props under the first floor girder shown in Fig. 2, whereas four are to be seen under the roof girder. This indicates that the props had been quite generally drawn out in the basement story. It means that the girders and columns of the first story were doing a large part of their work.

There was a lot of "unripe" concrete about the place, as would be expected when much of it was only a week old. But there were evidences much more sinister than unhardened concrete, for the building might have stood up until it was occupied, when a much more serious wreck might have taken place.

One of these evidences is in the columns, these columns being conspicuous by their absence. Of the several three-story columns in the wrecked portion, one lone stump remains. This may be seen in the lower left portion of Fig. 2. Note the "reinforcing" rods. These are curled up, illustrating how they expired in a vain effort to do their work as four little columns. The other columns were broken up into short chunks. These were standard columns, columns approved by the Joint Committee from the great engineering societies. That committee's report will allow plain concrete columns; it also will allow concrete columns with a little rod in each corner held in place with a little wire "during the pouring of the concrete." Some of these columns were about 10 ins. square and about 13 or 14 ft. high. And these foolish little shafts of concrete pass as structural members equivalent to a steel column, or a cast-iron column, or a hooped column. They have been present in nearly all of the large wrecks, and they invariably break up into short pieces and fail utterly to carry their intended loads or even to stand upright.

In one wreck a column of this kind—a seasoned column standing nearly 100 ft. from the wrecked portion of the building—broke under a load of 200 lbs. per square inch, simply from the jar of the nearby collapse.

This style of so-called reinforced concrete column has not, and never had, and never could have, any excuse for its existence. Tests and wrecks have demonstrated again and again that it is unreliable and dangerous. The Joint Committee ought to be called in special session to revoke their recommendations concerning it, and the innumerable building codes based on those recommendations ought to be amended.

Another bad feature of design in this building is in the girders, as is evidenced in the wreck. These girders sheared squarely from the columns and walls, showing that the steel did not penetrate the supporting columns and walls. There were shear members, web reinforcement, of course, and that web reinforcement fulfilled the Joint Committee Report. That report says, "It is important that adequate bond strength or anchorage be provided to develop fully the assumed strength of all web reinforcement." This is commonly interpreted to mean that shear members should be attached to the longitudinal steel rods, and it was probably written for the benefit of the systems that have such details. It certainly does not say that web reinforcement, nor does it say that longitudinal reinforcement, should be anchored into the support. The Joint Committee seemed to be satisfied if a beam held together; the little matter of its load being carried to the support did not concern it. How any intelligent man can place any dependence on stirrups or short shear members is a mystery to me. If there had been three times as many shear members in these girders they would have fallen away from their supports with exactly the same facility.

As I have said before these wrecks and their cause (bad design) are a burning shame on the engineering profession.

Very truly yours,

EDWARD GODFREY.

Monongahela Bank Building,

Pittsburgh, Pa., Nov. 9, 1914.

(It will be noted that no criticism of individuals is intended by the writer of this letter, as it is specifically stated that even though "the one big cause behind this wreck was bad design," the blame is placed on the "structural engineering profession at large." It will also be noted that the writer definitely points out the features which he criticises as bad design.—Editors.)

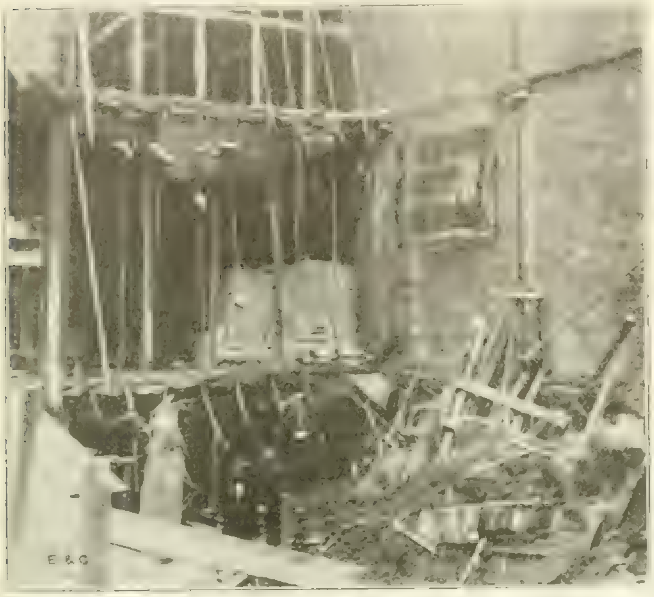


Fig. 2. View of Wrecked Portion of Theater and Arcade Building—Note Props for Floor and Stump of Column—See Fig. 1 for Location of Camera.

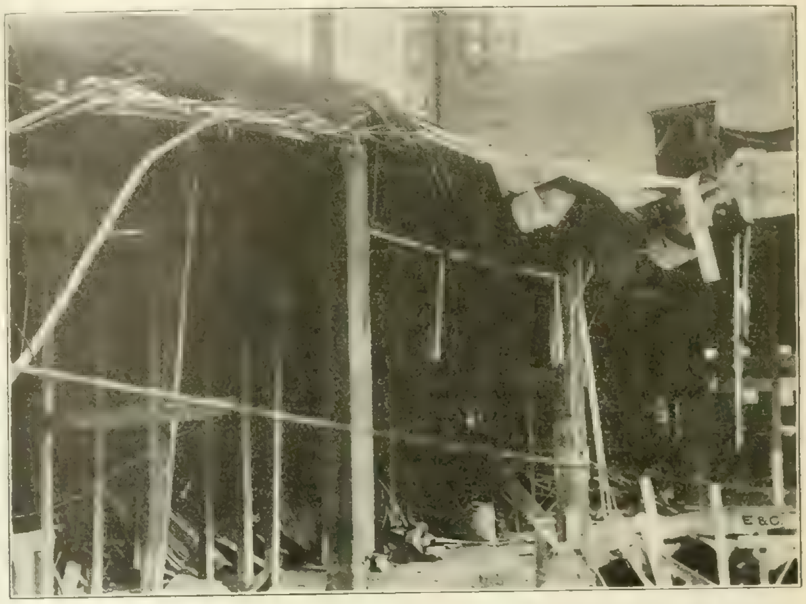


Fig. 3. View of Wrecked Portion of Theater and Arcade Building—Note Squareness of Girder Fracture—See Fig. 1 for Location of Camera.

WATER WORKS

The Use, Design, Construction, Cost and Durability of Wooden Stave Pipe.

II.

The Durability of Wooden Stave Pipe.

[illegible] gineer, Palo Alto, Calif.

The materials used in a wooden pipe are, more or less, subject to deteriorating influences; the wood decays and the iron rusts. When these natural phenomena have proceeded far enough a pipe becomes useless, and must be renewed. The length of time between installation and renewal, may be long or short, depending on: (a) the general and local physical conditions of the region through which the pipe is built; (b) the kind and character of the material used; (c) the design; (d) the care used in the manufacture of the material, and the location, construction, and operation of the pipe, and (e) the measures taken to prevent or retard the action of decay and rust.

If decay destroyed wood, and rust corroded iron with the same avidity in all localities, under all physical conditions, of soil and climate, then a wooden pipe would last as long in one place as in another. Then the useful life of a pipe, as determined by an actual case, would determine the useful life of any other pipe; but this is not the case.

Wooden pipe has been in use now long enough to furnish examples that indicate conclusively that pipes built of equally good material, and with equal care, and under similar operating conditions, will last much longer in one locality than in another, and that some parts of a pipe line will last longer than other parts.

The same thing is noticeable in the case of iron pipe. The metal rusts gradually in the presence of moisture and oxygen. These two elements being everywhere on the earth, rust is equally prevalent. But rust is greatly accelerated by the presence of acid, common salt, or weak alkaline solutions. There may be in the water, carbonic acid, or in the soil humus acids, common salt, or alkalis. The presence or absence of these accelerating agents would make a vast difference in the life of an iron pipe; they are prevalent in some localities and wanting in others.

The same agency, rust, attacks the bands of a wooden pipe. Here the rapidity of its action is but little influenced by any acids or alkalis carried by the water because it is separated from the bands by the staves, but elements in the soil or the air will have an accelerating effect, which will vary largely with localities in proportion to the presence or absence of such elements.

The only serious destructive agency, affecting the staves of a wooden pipe is decay, and the life of the staves will depend on the rapidity of growth of the decay fungus. This, in turn, will depend for its prevalence and rapid growth on propitious climatic conditions.

Decay is purely a vegetable growth, that spreads its small thread-like branches in a network through the walls of the wood cells, converting the wood substance and cell contents into the food on which the fungus lives, until eventually nothing remains except a punky mass. In order to thrive, the fungus must have, besides the food obtained from the wood, some water, air, and heat. New decay growths start from seed, which is furnished in abundance by the toadstool growth which appears on the outside of a decayed piece of timber. Extreme cold only retards decay but heat exceeding 150° F. will kill it. A temperature around 75° and 80° F. is most suitable for a rapid decay growth.

It is an easy matter to conclude from these facts that a timber structure will decay more rapidly in a uniform, temperate climate of a region subject to a large annual rainfall that is distributed over a good part of the year. Such a region would be timbered, and decaying logs and branches would be scattered in profusion among the trees, supplying the fungus seed or spores in clouds. Staves taken into a region of this sort would be literally covered with seed or spores before being built into the pipe. While the wood remains dry the spores will not grow, but after being built into the pipe will start if the conditions of growth are favorable, or are permitted to become favorable. Nor is it difficult, considering the same facts, to conclude that decay growth will be at a disadvantage in a semi-arid region of high altitude. Here the low temperature for the most part of the year prevents or retards decay, and when the temperature becomes favorable, other conditions are likely to be generally unfavorable. It is in regions of this sort that wooden pipe will have a long life, and be much in demand.

Twenty-two years ago the Butte Water Co. of Butte, Mont., built a long line of wooden pipe that is today in excellent condition. The maintenance charges on this line have been merely nominal from the beginning to date. It would be safe to say that this pipe line is good for another 20 years. The Garfield Water Co. of Garfield, Utah, installed several miles of large pipe eight years ago. This pipe is above ground and shows absolutely no signs of deterioration. The maintenance charges have to date consisted of a patrolman's salary. There are many other pipe lines in this region, which extends from the Rocky to the Sierra Nevada Mountains and from up in British Columbia down into Mexico, that are giving extraordinarily long service.

The wooden pipe built by the city of Astoria, Ore., 18 years ago, affords a good example of what might be expected in a region as first described above. In article I this pipe was referred to as an example of the results of poor location. Extensive repairs were necessary after ten years, due largely to locating parts of the line close to the hydraulic grade line. The unfavorable location was the more reprehensible when the unfavorable climatic and local conditions are taken into consideration. An examination of the portions of the pipe, more favorably located, made at the time of these renewals, was the basis of an opinion that the pipe would not last more than 10 years more, making 20 years the total life of the pipe line. It is very probable that this pipe has nearly reached the end of its usefulness. The climatic conditions around Astoria are almost ideal for rapid decay. The rainfall is generally distributed throughout the year, the temperature is mild and even and rotten wood is scattered everywhere through the forests.

The climatic influence of a large area of country cannot be avoided, but should be given due consideration in determining whether or not wooden pipe will be the economical kind to use.

Local conditions and influences embrace more particularly the character of the soil, and of the vegetation along the pipe line. Different conditions of soils have decided differences in effect upon the life of both bands and staves. Soils impregnated with alkalis or common salt hasten the corrosion of the bands; so also will the presence of any kind of an acid. Carbonic acid is even present in the air, and can be partly avoided by covering the pipe with a heavy soil that will largely exclude the air.

One of the products of vegetable decay, by *oxidation*, is humus acid, which is an active agent in the corrosion of iron. This acid is formed in all damp ground when the decay of vegetable matter is going on. The backfilling over a pipe should contain no vegetable matter. Leaf mold through wooded areas should be excluded from backfilling, but even then the infiltration of water from above will carry down humus and carbonic acid, but in lesser amounts as the depth to the pipe increases, and especially if the backfilling is a dense soil.

Destructive agencies that cannot be avoided can be inhibited in their action on the iron bands by painting or galvanizing. The action of alkalis and common salt can best be avoided by building the pipe above ground when soils impregnated with these salts are encountered. The outfall sewer of the city of Palo Alto is a 12 in. wooden pipe, laid through a salt marsh. After seven years' service the bands were nearly rusted away, while the wood was as sound as when the pipe was installed. At this time the City Engineer reported to the Council on the condition of the pipe, in part, as follows:

In one place, about 600 ft. from the outfall, there was evidence of sewage on the surface above the pipe, so I uncovered the pipe for a distance of 15 ft. in that location, and found a ¼-in. slit between two wood staves through which the leakage occurred. The only remains that I could find of iron bands at this point was a small piece about 6 ins. long. In my opinion the pipe is entirely without support of bands, or at best only feebly supported, throughout its whole length, and is held together merely by the pressure of the mire around it.

The elements making up the soil are not important as far as the life of the staves is concerned. In order to decay, the destroying fungus must have water, air, food, and heat. If the air is cut off the fungus will not grow. A loose covering of soil will not cut off the air supply, but a covering of a dense soil about 4 ft. in thickness, well packed, will largely exclude the air, and thus inhibit decay growth. The dense covering also prevents the evaporation of the water that percolates through the staves, thereby insuring a more nearly complete saturation of the staves and contiguous soil with water, thus forming a medium which aids in if not entirely excluding the air from the surface of the staves, and thus further inhibiting or preventing the growth of decay. While this fungus growth must have water, a saturated piece of wood (not immersed) either contains too much moisture, or the wood cells being filled with water prevent the growth getting the proper supply of oxygen from the air, for the degree of saturation of the wood seems to affect, inversely, the growth of decay.

The superintendent of the Butte Water Co. of Butte, Mont., in writing of the condition of their pipe in 1906, wrote in part, as follows:

In the first pipe we built, in 1892, for a short distance through a rocky cut, we were not particular about backfilling, and considerable of the covering was broken stone. In a short time after using it we discovered that the staves were beginning to show indications of decay along this portion of the line. Believing decay was caused by the rock filling allowing the air to get to the outside of the pipe, thereby causing decay, I had the filling entirely removed and replaced with good dirt on top of the pipe. Since then we have had no trouble and have never found a stave rotted sufficiently to require its removal. Where the pipe is properly backfilled with good dirt all around the pipe the seepage from the pipe keeps the earth damp, and practically waterlogs the wood, so that I see no reason why we should ever have trouble with rot.

Up to about the year 1875 the city of Detroit had installed in all about 200 miles of log pipe in their water system. These logs were of tamarack with about a 2½-in. hole bored through them. They were laid for the most part in blue clay, with the exception that an occasional pocket of sand or gravel was encountered, and the pipe was laid through it. It was found that logs laid in the sand or gravel would rot out within five or six years, while specimens have been taken from the blue clay after about 50 years' service in good condition. Here the clay covering excluded the air from the pipe surface, thereby cutting off the supply of oxygen required by the decay fungus, while in the sand or gravel pockets the air reached the pipe, supplying the fungus with oxygen and also allowing the water that filtered through the logs to drain away from the surface, and also to evaporate to some extent, thereby reducing the degree of saturation of the log surface to that more nearly suitable for a rapid growth.

Brush or trees that grow rapidly and require much water, if permitted near a pipe, will, as mentioned in Article I of this series, send great masses of roots that will spread over the surface of the pipe. Under these circumstances the pipe is ready to be renewed after from two to ten years' service. It is known that the staves will decay exceedingly rapidly under this condition, but the reason for the rapidity of the deterioration is not apparent. The moisture in the surface of the staves is reduced to a state suitable for decay growth, but it seems as though the roots exert some influence that is not understood. In Article I reference was made to the destruction of part of a pipe line at Long Beach, Calif., from this cause.

In 1896 and 1897 the West Los Angeles Water Co. of Los Angeles, Calif., built quite a long stretch of 30-in. wooden pipe that after about ten years gave considerable trouble, making it necessary to do considerable repair work each year. The main trouble was that portions of this pipe were laid in ground that was overgrown with willow brush, and the soil was sandy and porous, thus permitting air to reach the pipe. Moreover, the winter of 1897-1898 was very dry and during the summer of 1898 the pipe was not kept full of water. During this period decay started and grew very rapidly, encompassing the almost total destruction of staves at places, making repairs necessary during the first year.

The Southern California Mountain Water Co., at San Diego, Calif., had a similar experience with a pipe that was only partly full at places during a hot, dry summer. Some staves decayed in one season to such an extent that renewals were necessary. When wood is once attacked by the fungus it becomes predisposed to further decay; therefore, it is desirable to inhibit the commencement as far as is practicable.

Under certain conditions the bands of a pipe give way before the staves, but generally the staves give out first. As long as the pipe must wear out it is desirable to have all parts go at about the same time. If precautions be taken to keep the air away from the surface of the pipe, by putting the pipe a good depth below the surface, the depth of cover varying inversely as the density of the soil, and backfilling with clean, dense earth so that the water that percolates through the staves will keep them thoroughly saturated, a combination is secured that insures a long life to the staves, regardless of the natural durability of the wood. If the conditions are such that a pipe cannot be so covered, and the soil is light and porous, then the available wood that is naturally the most resistant to decay should be selected. However, the selection of any one variety does not mean that all staves obtained will have equal resistant qualities.

Under the most favorable conditions of heat, light, soil and moisture hardwoods produce the most durable woods, but in the case of coniferous trees it is the opposite. Pipe staves are made exclusively from coniferous woods, and come largely from the regions where the trees grew most luxuriantly. If, for any particular project, arrangements could be made for securing staves from mills located in regions where the conditions are less favorable for rapid growth, staves of a more durable quality would be obtained. The redwood belt in California is along the coast, extending from the Oregon line to Monterey Bay. The most favorable conditions for growth are in the North, decreasing to the South. It is a well known fact that the Santa Cruz lumber, coming from the lower end of this belt, is more durable than the timber from the North. But, get staves where you will, and they will not be uniform in quality since trees in the same locality produce lumber of different quality, and this is even so in the case of individual trees. Lumber from the lower cuts of a tree are much more durable than those from the upper cuts. It is not practical to get all butt-cut lumber, but it is practical to exclude the light lumber from the upper cuts.

About eight or nine years ago a city near the Atlantic Sea Coast had installed about 20 miles of wooden pipe 30 ins. in diameter. The staves were made from redwood lumber, and were manufactured on the Pacific Coast. In order to effect a saving in freight the manufacturer picked from the yard all light weight lumber. Some cars that were shipped averaged as low as 1,800 lbs. per 1,000 ft. board measure; which means, of course, that some staves were even lighter than this. This light material was built into the pipe, and some of it stood nearly two years before water was turned on. Then it was found that many staves had punky places in them and, where the pressure was sufficient, the water would break through. A large number of cast plates were made that fitted the surface of the pipe, and were put on at these places and fastened down with the bands. It is probable that it will not be long before we hear of this pipe being cited as an example of the short life that might be expected of wooden pipe.

The bands of a pipe should be made from mild, homogeneous steel. Segregations in the structure of the metal stimulates corrosion. Steel that has been carelessly manufactured or that contains a high percentage of impurities should be avoided. It is contended by some that a certain percentage of impurities inhibits corrosion, but the adherents to this theory seem to be in the minority. If heterogeneousness cannot be avoided with the presence of impurities, they should be kept at a minimum. Basic open-hearth steel is now being manufactured that is almost pure. However, most of the bands used on "continuous" stave has been Bessemer steel, and it has given good results.

Most machine banded pipe is wrapped with cold drawn open hearth steel wire that has been heat treated and galvanized. The quality and continuity of the galvanizing is very [illegible] a continuous coating of pure zinc of uniform thickness that adheres firmly to the steel, all lots should be tested.

The American Telephone and Telegraph Co. uses the following test:

[illegible] and wiped dry with waste.

The solution shall consist of commercial copper sulphate crystals dissolved in cold water in the proportion of 36 parts, by weight, of crystals to 100 parts of water. Then neutralize the solution [illegible] cupric oxide. The sediment of this reagent at the bottom will indicate an excess.

Then filter through filter paper, and the filtered solution shall have a specific gravity of 1.186 at 65° F.

The temperature of the solution should be maintained at about 65° F. during the following test:

Immerse samples four times, of one minute duration to each immersion. After each immersion the samples should be washed in clean water having a temperature of about 65° F. and wiped dry with cotton waste.

If there should be bright metallic copper deposited upon the samples after the immersions the lot represented by the samples should be rejected.

However, copper deposits on the zinc or within 1 in. of the end shall not be a cause for rejection.

This test will disclose the thickness of the galvanizing, but not the purity of the coating.

The shoes or band couplings on continuous pipe should be made of malleable cast iron in order to avoid breakage and to secure a lighter shoe. Malleable iron resists corrosion exceedingly well. The clips used to fasten the ends of the wire on machine banded pipe would better be made of malleable iron rather than from steel plate, because it is more resistant to rust.

There are two features of the design of pipe that particularly affects durability.

If the rods or wire used for banding are of unusually small size, the thickness of the ring of metal that provides a factor of safety would be very thin, and would be rusted away in a comparatively short time. For all sizes of pipe, where the bearing strength of the wood under the band controls the spacing the factor of safety against the water pressure for any given size of rod or wire increases as the size of pipe decreases. It is customary to use smaller wire on the smaller sizes of pipe in order to avoid the use of an excessive quantity of metal, but where competition is keen this is likely to be carried to extremes.

The weight of the metal and the consequent increase in the cost of the pipe, do not increase proportionately with the factor of safety. For example, a 6-in. pipe banded with a No. 6 wire would have a factor of safety of 5.3, and if branded with a No. 3 wire would have a factor of safety of 7.5. In the first case, if banded for a 100-ft. head, the wire would weigh 1.42 lbs. per foot of pipe and in the latter case, 1.85 lbs. This means an increase in the cost of the pipe per foot of about one cent; a cent well spent.

In cases where the bearing strength of the strip of wood under the bands controls the spacing, if the strength of the wood has been assumed too large, the staves will move outward when the pressure comes on until the increasing width of the indentation provides sufficient area to hold, which it will do if the discrepancy has not been too great, but the wood fibre along the edges of the indentation will be broken and decay will begin first and will proceed rapidly in these broken places. If the width of the indentation does not exceed one-half the diameter of the rod the wood fibre will be bent but not broken.

The rods and wire used in banding must be of careful manufacture. For continuous pipe the heads on the rods should be of full dimensions and must be uniform; if not of exact size and uniform they will not fit properly in the shoe. When the shoes are being manufactured care must be taken that the receptacle for the bolt head is of the right dimensions and is of sufficient smoothness to allow the bolt head to enter easily. The thread on the rod should be cut or rolled so accurately that the nuts will fit close, but still turn easily and smoothly.

The staves should be milled very accurately to the required dimensions. A steel templet should be at hand so that tests can be made frequently in order to detect any change that might occur in the setting of the cutters on the machine. The first stave run after the machine has been set up should be sawed into short pieces, and the number necessary to complete the ring of the pipe be bound together, thus forming a short section of the pipe. Then it can be noted whether or not the diameter is of the requisite dimension and whether or not the radial joints fit accurately at both the outside and inside of the pipe. Seams that gap open, especially on the outside, form excellent places for decay to begin.

It is especially important that the saw kerfs across the ends of staves of continuous pipe be of uniform position and of accurate and uniform depth. If not of uniform position then the staves do not butt together so as to form smooth outside and inside surfaces. Unevenness on the outside provides favorable spots for decay to begin, and on the inside interferes with the flow of the water. If the kerf is cut too deep the tongue does not jam into the bottom sufficiently to make a watertight joint, and if not deep enough the tongue will split the staves when they are driven home. The tongues must be cut from band iron of uniform width or the same trouble will result.

In machine banded pipe the surface of the tenons should be concentric with the inside surface of the pipe, otherwise an unevenness in the inside surface occurs that reduces the capacity of the pipe. The lengths of the tenons should be such that a space will not be left inside the pipe when driven home in the connecting collar.

In banding continuous pipe the rods should not be hammered so hard in an endeavor to give them the proper seating that the matter is overdone and the pipe of the wood broken. The same thing applies to the machine banded pipe; if the tension in the wire is too great when being wrapped the wood will be broken. Injuring the wood in any way makes spots where decay will start first by striking them with a hammer hard enough to break the fibre.

It is not probable that natural conditions will often be found under working circumstances that will absolutely prevent decay and often injured, so as to produce this result, and will be particularly active. Staves are rust, but advantage can be taken of favorable conditions to such an extent as to largely inhibit destruction from these sources. In the past the coating of the staves with some decay preventive has been suggested and tried, but nothing is so effectual as keeping the staves saturated with water if the pipe is buried. A thorough saturation is not desired or obtainable if above ground.

As stated before, the fungus growth causing decay must have food, heat, moisture and air. If any one of these requisites is cut off entirely then there is no decay. The foodstuff is the wood itself, so that will always be on hand and cannot be eliminated. Under all practical conditions there will be some degree of heat present. High temperature, 150° F. and up, will kill the fungus, but we cannot keep the pipe hot. With the absence of moisture wood is dry. We all know that dry wood will not decay. Wood immersed in water will not decay because the supply of air is cut off.

In a pipe line we can have wood containing moisture as far as practical results are concerned in all degrees from dryness to immersion. The rate of evaporation from the surface of a pipe built entirely above the ground will be so rapid that a skin of dry wood, varying in thickness indirectly as the pressure, will surround the entire pipe. Decay will take place only at poorly fitted joints where the wood has been injured as by a hammer blow, or where the fiber has been broken by the band being indented too deeply. With a covering of dense, heavy soil of sufficient thickness to prevent the air readily reaching the pipe and of a character that will hold the moisture that percolates through the staves, there is obtained a moisture condition surrounding the pipe that will exclude any air that might come through the covering, and thus prevent decay almost as readily as will complete immersion in water.

If a pipe line along which pressures vary from that due to a few feet of head to that due to 200 ft. is covered with ordinary soil porous enough to allow air to reach the pipe, but that will retard evaporation and prevent rapid drainage, the parts under the higher heads will be surrounded by a moisture condition that will inhibit decay. As the parts under low pressure are approached the moisture condition of the staves and surrounding the staves will not exclude the air from the stave surface, and decay will become more rapid. Under ordinary conditions durability alone would demand that wooden pipe be used for heads from about 40 ft. up to the limit. For lower heads than 40 ft. the pipe should be covered by a denser covering and to a greater depth, or located entirely above the ground.

It requires moisture in the presence of oxygen to rust iron. The corrosion will be greatly accelerated by the presence of even a weak acid. Alkalis are opposed to acid, but weak alkali solutions aid corrosion, while concentrated solutions will inhibit. If it were practicable to cover the pipe entirely with slacked lime the iron would not rust. Under all ordinary conditions attending a pipe line the bands will rust, and some good quality of elastic paint will help in prolonging their usefulness. The paint must form a coat that will not fly when struck by a hammer, but will simply mash to a thin film. The galvanized banding of machine banded pipe is very likely to have the galvanizing cracked during the process of winding it on the pipe, and unless protected is subject to further injury during transportation and handling. Running the wire through a paint that will form an elastic coat, while the pipe is being wrapped will afford some protection, but this coating would be badly injured during transportation. Coating the entire pipe with a hot asphaltum mixture and rolling the pipe in sawdust if carried far enough to make a covering about ⅜ or ½ in. thick will give good protection, as this forms a very tough and elastic covering. This covering prevents the deposit of decay spores on the wood, thus putting off the time when decay starts.

Under all ordinary conditions for which wooden pipe is built decay and rust take place, and it therefore behooves one to make these agencies as ineffectual as circumstances and economy will permit.

Any references to the design of pipe in the present article will be taken up in detail in the article to follow, entitled "The Design of Wooden Stave Pipe."

British Practice in the Design of Reinforced Concrete Reservoirs.

The following notes on British practice in the design of reinforced concrete water supply reservoirs are from an address at University College, London, England, by Prof. E. R. Matthews of the Department of Municipal Engineering of the University of London.

Some of the advantages of reinforced concrete as the material of construction for water works reservoirs are: There is a saving in first cost; a reservoir built of this material occupies less space than one constructed in masonry or brickwork, or a combination of both; it can more easily be made watertight; it increases in strength with age; there is less likely to be vegetation upon the face of the walls; practically no cost in maintenance.

Circular vs. Rectangular Reservoirs.—In the United States most of the modern reservoirs are circular in plan. Reservoirs of this shape are seldom constructed in England, British engineers preferring a rectangular structure. I recommend the circular shape, the chief advantage being that when of this shape no angles occur, so that the reservoir wall can be built without a joint. This is a great advantage, for one often finds that when leakage occurs in a reservoir it can be traced to the point where the walls join. Circular reservoirs, because of their circular form, are also better adapted to withstand external earth pressure; they are cheaper than rectangular ones.

The Twin Peaks reservoir at San Francisco is a good illustration of the curvilinear type of reservoir. This may be briefly described as follows:

The reservoir was completed about two years ago, and is a good example of what may be done in reinforced concrete. It is oval in plan, and divided into two parts by a reinforced-concrete dam. The reservoir is situated 750 ft. above the business section of the city, and is part of the high-pressure system for fire-extinguishing purposes established in consequence of the great fire of 1906. The depth of water allowed for is 27 ft., and the capacity is 11,000,000 gals. The buttresses of the dam are spaced at 9-ft. centers on both sides of the wall. They are 1 ft. in thickness, and 13 ft. wide at the base.

Watertight Reservoirs.—Referring to the leakage of reservoirs, we would say that this is a problem which has engaged the serious attention of engineers for many years past. Reservoirs have been constructed in what appeared to be a substantial manner, on a good foundation, and at a cost of many thousands of dollars, but they have never ceased to leak, and while many attempts at remedying the leakage have been made, these constructions are looked upon, from an engineering point of view, as a failure.

I know of no type of reservoir, however, which is more likely to be watertight than that constructed of reinforced concrete, provided the reservoir is properly constructed and expansion joints are inserted in suitable positions both in the walls and floor of the reservoir, and if suitable material is used for the concrete, and the walls are not less than a minimum thickness of 5 ins. and are well reinforced, and that the depth of water in the reservoir does not exceed, say, 30 ft.

The thickness of the floor will depend entirely upon the sub-foundation. This floor may be either of plain or reinforced concrete; if the latter and the sub-foundation is good, the thickness of floor should not be less than 6 ins. for a depth of water up to 15 ft., and 9 ins. for a depth ranging from 15 ft. to 25 ft., and 12 ins. from 25 ft. to, say, 35 ft. In every case it is assumed that the floor is well reinforced. In order that the reservoir may be watertight, in addition to the before-named precautions, great care must be taken as to the properties used in the concrete. I would suggest that never should a weaker mixture be used than 1:2:4, and where the depth of water to be stored is not less than, say, 20 ft., I would recommend a mixture of 1:1½:3. The materials should be very carefully graded, as most of the trouble in reservoir work occurs where this has been imperfectly done.

I also recommend for reservoir work that a wet mixture shall always be used, and while a dry mixture may show a better result as regards its strength in compression, and a plastic mixture may also give better results in this way, still a wet mixture more perfectly prevents leakage, and on that account should always be used.

Not only so, but with a wet mixture the reinforcement is better covered. I have seen reinforced-concrete work where a fairly dry mixture had been used, and where occasion had arisen for the breaking up of the concrete, where the reinforcement in places had not been covered in the least degree by the concrete. This had occurred, first, because the mixture was too dry, and, secondly, because the aggregate used was too large. In practice the concrete should be of such a consistency that every portion of the reinforcement will be well covered, and every crevice filled.

Not only is this obtained by using a wet mixture, but with such a mixture when the forms are removed there is a smooth face to the concrete, and on this account also it is wise that the mixture should be fairly wet. Care, however, must be taken to see that it is not too wet. It is possible for too large a quantity of water to be added so that the grout will run through the joints in the forms, and escape from the wall; on no account should this occur. If reservoir walls are constructed with a mixture of 1:2:4, the ultimate strength of this concrete, assuming well-graded stone and sharp sand is used as the aggregate, will be about 2,000 lbs. per square inch at the termination of one month, and 2,700 lbs. per square inch at the end of six months. The usual working stress for good concrete is taken at 600 lbs. per square inch. Under exceptional circumstances, however, this may be reduced to, say, 500 lbs., but I would recommend, generally speaking, for reservoir work, 600 lbs. per square inch.

Reservoir Piers.—These are sometimes built circular in plan, and there is a slight saving in the space occupied by so doing. It is, however, more costly to construct the forms in this shape, and the advantage of saving in space does not really compensate for the increased cost. The usual practice in this country is to build rectangular piers, and to chamfer the edges, say, with a 1½-in. chamfer to a 10-in. or 12-in. square pier. The square pier has the advantage also that it affords a better seat for the reinforced-concrete beams carrying the roof to rest upon.

Reservoir Roofs.—In reinforced-concrete reservoirs this usually consists of a reinforced-concrete slab 5 ins. or 6 ins. in thickness, carried by reinforced-concrete beams which rest on the piers just referred to. Some engineers prefer a lighter slab, and to ensure this they introduce a system of secondary beams. For my own part I do not care for the roof slab to be of greater thickness than 6 ins.; 5 ins. is preferable, and the beams can be designed accordingly. In calculating the weight of the roof it is necessary to include the weight of concrete and reinforcement forming the beams and slabs, also to allow for any soil which may be spread on the roof. This is usually about 1 ft. or 18 ins. in thickness, also to assume a thickness of snow of, say, 2 ft. It is necessary to construct two or three openings in the roof for the purpose of getting to the inside of the reservoir, and iron ladders are usually built in at these points. The openings need not be larger than 6 ft. by 3 ft.

Cement Grouting.—It is always advisable after short sections of the work have been completed to wash the walls over with two coats of cement grout, 1 to 1 (sand and cement). This fills in any slight cracks which may have occurred on the surface of the concrete. A fortnight should elapse before the grout is applied. No further cracks on the surface of the concrete will be likely to occur.

Wetting Concrete.—If the reservoir is being built during the summer months, when the sun is likely to dry the surface of the concrete very rapidly, the latter should be frequently wetted so as to retard the drying action of the sun. It is also advisable to do this in high winds.

Size of Aggregate.—In reservoir work the aggregate used in the construction of the walls, floor, piers and beams of the reservoir should be capable of passing through a ¾-in. ring; for the roof slabs it should be capable of passing through a ½-in. ring.

Where the thickness of the concrete floor exceeds 9 ins. the aggregate should be capable of passing through a 1-in. ring, the upper 4 ins. of the concrete in the floor should consist of concrete, the aggregate of which is capable of passing through a ¾-in. ring.

Reinforcement.—It is not my intention to advocate any particular system of reinforcement. There are many in the market, and most of them are good, and I say this after having used several different systems myself. It is quite possible, however, and I strongly advise you to adopt this course, for the municipal engineer to work out his own stresses and design his own system of reinforcement.

Thickness of Walls.—For heads up to 10 ft. a 5-in. wall will generally be found to be sufficient; a 4-in. will answer, but I prefer a minimum thickness of wall of 5 ins. It has, however, been suggested that the walls should be increased in thickness for every additional 5 ft. increase in head pressure. I agree with this suggestion, but prefer that the top 6 ft. of the reservoir wall shall be 5 ins. in thickness, and that the wall shall be increased in thickness by 2 ins. for every additional 6 ft. of its height.

It is usually desirable to build counterforts or buttresses at the back of the wall.

There is a saving of from 20 to 25 per cent by using reinforced concrete in the construction of reservoirs. Some engineers recommend that the water face or reservoir walls shall be plastered with 1 to 1 cement mortar to a thickness of ½ in. or ¾ in., but I do not favor this; I think it is unnecessary.

Stresses.—The elongation of steel being so much greater than that of concrete, it is advisable, in order to ensure of the reservoir being watertight, to limit the stress to 12,000 lbs. per square inch, and I think it is advisable in reservoir work to use bars which give a mechanical bond, such as the indented bar, twisted bar, lug bar, or one of the other various types of bars, rather than plain bars, but this only applies to reservoir work, as plain bars are admirable for all other reinforced concrete work.

It is necessarv that an adequate percentage of steel be inserted in the concrete in both directions, even when the stresses due to water pressure occur in one direction only, or otherwise slight cracks may occur.

BACKING OF RESERVOIR WALLS

Reservoirs may be: (a) Entirely sunk below ground; (b) partly sunk below ground; (c) out of ground.

(a) Reservoirs Sunk Below Ground.—On designing the first type we must remember that the reservoir mav be full or empty. When empty the walls will be acting as retaining walls with all the pressure at the back; when the reservoir is full there is still the pressure of the soil at the back of the walls, but counteracting this is the water pressure from within. It is therefore desirable that walls of this class shall have reinforcement on both sides, and not in the center of the wall, as had the reservoir which failed at Mittagong, N. S. W.

(b) Reservoir Partly Out of Ground.—These are known as the cantilever type. In determining the width of the heel we must remember that in an ordinary retaining wall the weight of the earth at the back of the wall would help to keep the wall from overturning, but with a reservoir the weight of the water should be disregarded. The heel should therefore be made of sufficient width and thickness to be stable when the weight of water above it is neglected. One of the most important things to be careful with in designing a reinforced concrete reservoir is the arrangement of the bars at the joint between the vertical wall and the footing.

(c) Reservoirs Out of Ground.—These will be similar in design, but buttresses are usually added. The wall acts as a dam to withstand water pressure.

RESERVOIR ROOF.

The reservoir roof often acts as a tie, and holds the reservoir walls together; the reinforcement is therefore arranged accordingly. When the span is large, however, the elongation may be considerable, and it is then not advisable to tie the walls in this way, but to give the roof free sides for expansion.

Steel Dumping Pier.—At Newport News, Va., an all-steel dumping pier has been constructed, one side of which was completed and put into operation June 1, 1914. This pier is electrically operated and equipped with every modern appliance for the expeditious handling of coal into cargo vessels and bunkers. The cost of this pier, with its accompanying facilities, is about $1,630,000. The pier has a rated capacity of 5,000 tons per hour when both sides are in operation, the road cars being dumped into conveyor cars which are lifted to the top of the pier by electric elevators.

Standard Form of Correspondence Used in the Department of Water, San Diego, Calif.

In order to standardize the correspondence between the various bureaus of the Department of Water of the city of San Diego, California, and other offices in connection with the city government a standard form of correspondence has been adopted. We here give a sample letter, which was written originally, of course, on the department letter head, of Mr. Herbert R. Fay, Superintendent, and a copy of the rules governing departmental correspondence. It has been found that this method not only shortens the labor of correspondence but keeps the letter files in first class shape.

SAMPLE LETTER.

The sample letter follows:

November 18, 1914.

FROM the Superintendent
TO the Chief Clerk.
SUBJECT: Account of John Smith.
1. You will please furnish this office with data regarding the above account at your earliest convenience.

H. R. FAY.

1st. Ind.

Chief Clerk to Sup. 11/18/14. I attach herewith all papers pertaining to the above account.

H. L. WORTHEN.

(Rubber stamped.) Filed, Nov. 19, 1914. H. R. F.

RULES GOVERNING CORRESPONDENCE.

The rules mentioned, entitled: Instructions for the Conducting of Official Correspondence of the Department of Water, City of San Diego, California, follow:

Heading and Subject of Letter.—The letter will begin with the place and date, written as at present; below this, beginning at the left margin, will come the word "From," followed by the official designation of the writer, or, in the absence of any official designation, the name of the writer with his rank in the department; below this, also beginning at the left margin, will come the word "To," followed by the official designation or name of the person addressed. Next will come the subject of the communication, indicated as briefly as possible and not to exceed 10 words. The words "From," "To" and "Subject" will begin on the same vertical line.

Example.

Superintendent's Office,
San Diego, Calif., March 1, 1913.

From the Superintendent
To the Chief Clerk.
Subject: Account of John Smith.

1. You will please furnish this office with data regarding the above account at your earliest convenience.

H. R. FAY.

In case of letter paper, the upper third, and in case of foolscap, the upper fourth of the sheet, will be devoted solely to the matter described in this paragraph. (See Par. 7.)

II.

Body.—Then will come the body of the letter, which, when typewritten, will be written single-spaced, with a double space between paragraphs, which will be numbered consecutively.

III

Signature.—The body of the letter will be followed by the signature. If the official title of the writer appears at the beginning of the letter, as it should, it will not appear after his name. (See example in paragraph I.)

IV.

Omission of Ceremonial Forms.—All ceremonial forms at the beginning and end of letters, such as "Sir," "I have the honor," "I would respectfully," "Very respectfully," etc., will be omitted.

V.

Use of Only One Side of Sheet.—Only one side of the paper will be used, the writing beginning about 1 in. from the top except in cases where the department letter head is used, and then the date line should begin two spaces below the head.

VI.

Office Marks.—All office marks will be placed immediately under the body of the letter, and in the case of indorsements, immediately after the proper indorsement.

VII.

Brief.—The matter described in paragraph I will constitute the brief of the letter.

VIII.

Folding.—Letter paper will be folded in three and foolscap in four, equal folds, parallel with the writing; the top fold will be folded toward the back of the letter and the lower fold over the face of the letter. In three-fold letters the brief will be on the outside, as will be the case also with four-fold letters.

IX.

Enclosures.—Enclosures will be fastened securely to the communication by means of paper fasteners which do not cut holes in or tear the paper. Pins will not be used. The number of enclosures will be noted just below the body of the letter at the left-hand margin. If others are added when an indorsement is made their number will also be noted under the proper indorsement, as: 2d indorsement, 2 enclosures.

INDORSEMENTS.

X.

Note.—The indorsement will be used in all cases where a reply to a communication is required.

Form.—The writing widths of indorsement will be the same as that of letters. The first indorsement will begin about ½ in. below the signature of the writer of the letter, and succeeding indorsements will follow one another serially, with a space of about ½ in. between indorsements.

Each indorsement will be numbered, and will give the title of the writer, to whom sent and the date.

When typewritten, indorsements will be written single-spaced with a double space between paragraphs. The paragraphs will be numbered consecutively.

Should one or more additional sheets be necessary for indorsements, sheets of the same size as the letter will be used.

In referring, transmitting, forwarding and returning papers, the expressions "Respectfully transmitted," "Respectfully forwarded," and "Respectfully returned," will be omitted.

Indorsements of a routine nature referring, transmitting, forwarding, and returning papers, will not be signed with the full name, but with the initials, as:

1st Ind.

Chief Clerk to Superintendent, March 2, 1913. —Attached data regarding above account.

H. L. W.

Indorsements will not be used where the Superintendent or the Assistant Superintendent requests a separate communication.

LETTERS AND INDORSEMENTS.

XI.

Numbering of Pages. — Communications requiring more than one page will have the pages numbered, beginning with the second at the top of the page and on the right-hand side, the number of the page to be preceded by the words "To the ....," with the official designation of the recipient of the letter, as:

To the Chief Clerk—2

In referring to an indorsement by number the number of the page will also be given. Thus: "5th Ind., page 3."

XII.

In cases where communications which require replies do not originate in the Superintendent's office a carbon copy of all such communications will be forwarded to that office for file. This applies in all cases whether the letter be sent to the Superintendent's office or to an official outside of that office.

XIII.

Channels of Communications.—Except in cases where communications originate in the Superintendent's office, all correspondence will be carried on between employees and their immediate superior, who, in turn, will address his superior.

ACKNOWLEDGMENT.

We are indebted to Mr. H. A. Whitney, Hydraulic Engineer of the San Diego Department of Water, for the foregoing information.

Novel Method of Measuring Leakage in Gravity Pipe Line.

To the Editors:—In 1889 the writer was constructing engineer on a mountain reservoir which supplied several towns by means of a gravity line some 15 miles long. On completion the line was found to leak materially, in the neighborhood of 500,000 gals. per day. Men were immediately put to work to hunt the points of leakage and to repair the leaks. In order to determine the rate of progress of the repair gang it was desirable to find some means of measuring the amount of the leakage from time to time. The water surface in the reservoir covered some 30 acres, so it was impracticable to use the principle of volumetric measurement. The following simple device proved thoroughly satisfactory:

On the discharge line near the reservoir was a 6-in. branch and valve. Both above and below this branch were gate valves on the main pipe. A 1-in. tap was made on the top of the main pipe between the two gate valves, and a 1-in. pipe and valve were screwed into this tap. Upon opening this 1-in. valve the water spouted up to a height of some 10 ft., whereupon the upstream main valve was throttled until the stream just barely trickled over the top of the 1-in. pipe. Then the downstream gate valve was closed and the water again spouted up. The 6-in. branch valve was then opened gradually until the water in the 1-in. pipe again trickled over the top, and having previously put in a box weir and baffles just in front of the 6-in. branch, the amount of water going through the 6-in. pipe was readily measured, ten minutes being sufficient for the operation. During this time only about 3,000 ft. of the upper part of the line was emptied and only a short time sufficed to fill it up again. This kind of measurement was made as frequently as necessary, sometimes half a dozen times a day until the leakage was all repaired.

It is hardly necessary to make any further explanation to prove that the amount of water going through the 6-in. branch pipe during the test under the conditions stated was equal to the leakage, the point being that the discharge through the upstream gate valve was equivalent to that through an orifice produced by the gate and whose piezometric head was equal to the difference in level between the water upstream from the gate valve and at the top of the 1-in. pipe.

This method has since been used in a number of cases always with satisfactory results.

Very truly yours,

J. W. Ledoux.

Chief Engineer, American Pipe and Construction Co.

Philadelphia, Pa., Nov. 20, 1914.

ROADS AND STREETS

Road Legislation and Economics.

Highway economics both with regard to legislation and economics of construction is a subject of vital importance at the present stage of our highway development. In a paper before the American Road Congress J. E. Pennybacker stated some of the fundamentals of the subject which paper is given here.

ROAD ECONOMICS.

Road economics may be defined as that branch of economic science which treats of the cost and use of a road as a public utility. Cost and public utility, in a comprehensive interpretation, are the determining factors with reference to the amount of money to be expended, the method of its procurement, the liquidation of any indebtedness incurred in connection therewith, the location of the improvement, the character of the work, economy in the management of the project, and the utilization of the completed road for the economic benefit of the public.

The subject is logically comprised in two divisions, the first of which deals with those larger questions of legislation, finance, organization, road classification or selection, the utilization of collateral agencies, and the management of the road as a completed project. The second division of the subject although more limited in scope than the first division is important from the standpoint of economy and efficiency, as it relates to the various activities in connection with the actual work of construction. Examples under this division would be the lowering of cost by the intelligent use of labor-saving machinery; the keeping of adequate and efficient cost records so as to detect extravagance, incompetence or dishonesty; the systematic purchase of materials, and the use of such other measures as would serve to produce a satisfactory road at the lowest practicable outlay.

Legislation, to be effective, must be economically sound, and it is necessary to the intelligent framing of road laws that the economic considerations applicable to the subject should be known and accepted by the legislators. A system of financing road improvement is largely the outcome of legislation, but is often modified by the exercise of administrative discretion. Organization, like finance, is to a great extent prescribed by statute, but here again the personal equation enters largely in the determination of efficiency or inefficiency.

The utilization of collateral facilities of the State, such as convict labor and the aid of State institutions for investigative and educational work is largely determined by law but here again administrative discretion and the personal equation play an important part. The classification and selection of roads for improvement, although resting upon legislative enactment, are much more largely an administrative question than those to which I have already referred, and the same holds true with reference to the use of the road after completion so as to best serve its purpose as a public utility.

FUNDAMENTAL CONSIDERATIONS.

It is thus evident that these basic factors should be correlated and that the undertaking as a whole should conform to those economic considerations which may be regarded as fundamentally sound. I have, therefore, formulated ten fundamental proposition which I hold to be incontrovertible and so self-evident as to be axiomatic. I shall, therefore, first submit these ten propositions, and then endeavor to explain their practical application.

1. That all who share in the benefits of road improvement should share proportionately in the burdens.
2. That the degree of improvement should be proportionate to the traffic important of the road improved.
3. That the rate of payment or the rate of accumulation of the sinking fund on any public debt contracted for road improvement should approximately equal the deterioration of the improvement.
4. That road building and maintenance comprise work requiring special qualifications on the part of those who direct it.
5. That responsibilities should be definite as to persons.
6. That continuous employment is more conducive to efficient service than intermittent and temporary employment.
7. That the specialists who direct road work should be appointed instead of elected; and that they should hold office during efficiency instead of for a fixed term.
8. That no road is wholly permanent and that it requires continuous upkeep, for which financial and supervisory provisions must be made.
9. That cash is a much more satisfactory form of tax than is labor.
10. That all agencies at the disposal of the State, capable of use in works of public improvement, should be so used, rather than in such commercial production as would conflict with private enterprises.

COSTS AND BENEFITS.

The practical application of these ten proppositions does not involve intricate or impracticable procedure. Under the first proposition, that burdens and benefits should be shared proportionately, I would call attention to the fact that the country road is no longer a more local utility. The product of the farm is absolutely essential to the existence of the city population, while, conversely, the product of the city factories finds its way to the most remote country districts. There is an interdependence which should carry with it a cooperative sharing of the burdens incident to improving the facilities of transportation between country and city. Legislation should, therefore, be framed so as to provide for city taxation in aid of country road improvement. Automobile owners should individually pay a material portion of the cost of our public roads, and they are already cheerfully doing so in many of the States. Last year the state revenues derived from automobiles amounted to about eight million dollars applicable to roads, out of a total from all sources State and local, of about two hundred and five million dollars. The exact method of apportioning the road taxes is a detail which can readily be worked out by each individual State.

TRAFFIC.

The second proposition, which calls for the improvement of roads in proportion to their traffic importance, strikes at the very root of our present method of apportioning road improvement. Too often have we seen examples of costly improvements distributed according to the dictates of a few influential citizens or according to some arbitrary arrangement of political units or for sentimental reasons, or through a cheerful, haphazard indifference. It is now generally believed that four-fifths of the traffic of this country is carried on one-fifth of the road mileage. It should be manifest that the most heavily traveled roads should first receive attention and should be improved in the most substantial manner. It is entirely feasible to make an expert study of a county road system and indicate graphically the traffic areas for each important road, much as you would show drainage areas for waterways. The yield and the probable traffic in ton miles for these traffic areas can be readily determined so as to establish with reasonable exactness the amount of outlay which the traffic would justify. The relative cost of such a determination would be almost negligible if incurred as a preliminary to a large outlay for actual construction.

INCURRING DEBTS.

The third proposition, that debts should be liquidated in proportion to the deterioration of the road, is intended to prevent the incurring of a debt which will outlive the utility which it was designed to create. There are two extremes in the controversy which rages over this question of public debt. There is the one faction which either opposes debt in any degree, or contends for an indebtedness of such short term as to make it almost a cash transaction, and asserts that the road is entirely destroyed long before the debt becomes due. The other extreme faction contends for long-term indebtedness, on the theory that as posterity will reap the benefits it should bear the burdens, and that a road well maintained never wears out. As a matter of fact, location, if intelligently made, should be permanent; likewise all reduction of grades. The drainage features, if honestly and efficiently constructed, should be reasonably permanent. The road, except under extraordinary conditions, should, therefore, be considered reasonably permanent as to these features. As a general rule, the foundation of a road should not require renewal if the road is subjected to adequate and continuous maintenance. Avoiding any detailed consideration of the exact proportion of the total cost of a road represented by these features, I should say that in general the permanent features would average at least 50 per cent of the total cost. So that, if the other 50 per cent must be figured as perishable and subject to renewal, the debt should not cover a period longer than twice the length of this perishable portion. For example, if a macadam road is constructed at a cost of $6,000 per mile and has an estimated life of ten years, the bonds could run twenty years, because, at the end of ten years the depreciation is $3,000 and the actual value is $3,000. Another expenditure of $3,000 is made and at the end of the twenty years when the bonds become due, there has been a total outlay of $9,000, against which should be credited the permanent value of the road at $3,000, making the net outlay $6,000, or the face amount of the bonds. This is merely an example and a generalization. It would be desirable to ascertain the permanent and perishable portions in each undertaking.

SUPERVISION.

The fourth proposition, which calls for the employment of specialists in road work, is so nearly self-evident in its application as to require very little explanation. I should say, however, that if the laws of the State would require that all persons selected to have immediate direction of road or bridge construction and maintenance must possess practical knowledge and experience, and if this fitness should be tested by some sort of competitive examination to be prescribed by a State highway department, acting either directly or through a civil service commission, the net result would undoubtedly be the saving of many millions of dollars of road revenue and a wonderfully increased efficiency in our road system.

EXECUTIVE AUTHORITY.

The fifth proposition, that responsibilities should be definite as to persons, is aimed at the elimination of our present complex and cumbercome system of road management. If all of this antiquated organization could be swept aside and in its stead one or a few officials endowed with authority and charged with responsibility in each county, the beneficial effects could not fail to be most marked. If the people, individually or in a representative capacity, could immediately place their finger, so to speak, upon the man responsible for the discharge of public duties we should have no more political juggling and the passing of responsibilities and duties onward in an endless chain.

EMPLOYES.

The sixth proposition, that continuous employment is more conducive to efficiency than temporary employment, finds its antithesis in our present annual or semi-annual junket which we call "working the roads." It is so evident that a minor defect in a road can be repaired at its inception with little effort, and that if allowed to go on it may require the entire reconstruction of the road surface, that it seems scarcely necessary to urge the soundness of this proposition. If a small force of laborers with necessary tools and teams were employed throughout the year on the roads it would not cost any more money than to call out a small-sized army of road hands twice a year, and would not only result in quick repairs where needed but would also insure that the most work would be done at the places where it was most needed. The force would be small, mobile, trained, interested, subject to effective discipline and altogether infinitely more efficient than the unwieldy forces now employed.

APPOINTMENT OF OFFICIALS.

The seventh proposition, which calls for appointment rather than election and for the holding of office during efficiency instead of for fixed terms, is designed to attract to the work men who look upon road-building as a life profession or occupation. A good engineer may be a very poor politician and a good politician may be a very poor engineer, but in a contest in which votes are essential the good politician will usually defeat the good engineer, although the position requires engineering ability rather than political ability. Do not spoil a good highway engineer or superintendent by making him cater to the popular fancy. If he is the right man in the right place, it is absurd to limit him to a fixed term, for his position is not a reward. The county is purchasing his services and is supposed to get value received, and it should continue to purchase so long as he delivers the goods.

MAINTENANCE.

The eighth proposition, that no road is wholly permanent and that it requires continuous upkeep, is intended to impress upon legislators and administrative officials the necessity for making adequate financial provision to care for roads, no matter how costly or efficient their construction. A house is not permanent without repair, a railroad track is not permanent without repair, then why should public funds in a large amount be expended in road construction which, without adequate maintenance, may deteriorate to the extent of 50 per cent in a few years. It would seem almost a reflection upon your intelligence that I should urge upon you these conclusions which are so generally understood and accepted, were it not for the fact that their acceptance is very largely in theory and not in actual practice.

CASH TAXES.

The ninth proposition, that cash is a much more satisfactory form of tax than labor, is put forward as a protest against the continued cherishing of that old heirloom known as "statute labor." If A owes B $10 and B has the option of collecting that $10 in cash or taking the amount out in labor which A shall select and which is totally unfamiliar with the character of work which B requires and which would be semi-independent of any control by B, we should consider it very unsound business judgment if B were to accept the payment in labor instead of cash. If you provide an efficient highway engineer or county superintendent with a modest amount of cash and let him select competent, efficient laborers, he can quadruple the effective results obtained by the same number of laborers under the old statute system. I know that there are sections of country where it is almost impossible to collect a cash tax. A certain amount of discretion might in such cases be entrusted to the county authorities to accept payment in labor.

The tenth proposition, that state agencies which nay be used in works of public improvement should be so used instead of in commercial undertakings, is directed partially toward the convict labor question, and is based upon the assumption that offenders against society owe a debt to society which should be paid in such form as will most benefit society, and the further assumption that honest labor should not be discriminated against through the sale or disposal of products created by criminal labor. The practical application of this proposition would mean the employment of convicts in road-building, the preparation of road materials, or in other works of public improvement so far as practicable. This proposition is intended also to emphasize the necessity for correlation of the States' various agencies in the interest of road improvement. For example, a State geologist should be helpful in the selection and location of road materials, the laboratories of state universities should be useful in the testing of materials, the university staff should be helpful in the giving of theoretical instruction and in many cases in practical extension work, state bureaus of statistics and agriculture should be helpful in accumulating essential data for the road improvement work in the State, and state civil service commissions should be of very great use in the inauguration and conduct of the merit system in the filling of positions requiring technical or practical qualifications and experience.

SCOPE OF SUBJECT.

The subject of road economics is entirely too far reaching to be adequately treated in one paper, and I consider it more advisable to present to you these fundamental considerations than to attempt a hurried and general treatment of the whole subject. You can readily see that under the first division of the subject as I have outlined it, there yet remains a great field for analysis and discussion in the detailed application of systems of finance and taxation and in the organization and working policies of highway departments for state and local work.

The second division of the subject briefly referred to in the opening paragraphs and which relates to the efficient and economical management of the actual work of construction is important enough for a separate paper. I have pointed out a few examples to show you what this division of the subject comprises, but it is manifestly impossible in the space allotted, to take up the second division even in a general way. The time is fast coming, however, when only those contractors and those officials and engineers in charge of force account work who devote attention to the economics of actual construction can obtain material success.

The Location and Width of Highways and the Securing of Rights-of-Way.

One of the most important and at the same time vexing works with which the road engineer has to deal is the securing of rights-of-way. In a paper before the American Road Congress Austin B. Fletcher outlined the difficulties involved, with some methods of overcoming them, and his paper is given here.

LOCATION.

The highway location is the one really permanent feature of the road work. The time to secure proper locations for the roads, and widths sufficient to serve all purposes for long years to come, is now. If we wait until some future day to correct improper locations and to secure suitable widths of rights of way, when we have more leisure, we will have wasted much money in pavements constructed and the land needed will cost much more and will be more difficult to acquire. It goes without saying that all land owners are more complacent in giving up portions of their property to the public before the improvements are begun than at any time afterward.

In some of the older States the people came long before "sectionalization" by the government was thought of but in the Middle West and on the Pacific Coast, most of the land was divided years ago "checkerboard" fashion by the government surveyors. The highways in the older States were laid out, or in most cases, simply grew where the travel wanted to go but in the flat prairie land of the west, and even in the Pacific Coast valleys, the roads were often, if not generally, laid out straddling the section lines, the center of the right of way being usually coincident with the section line. This plan had the merit of lessening the area of land deducted for road purposes from the holding of an owner by making his adjoining neighbor provide one-half of the land required for the roadway.

This method of road location often proves to be an embarrassment to the present-day road builder since this time-honored rectilinear scheme does not fit the present needs. Centers of population often times have not occurred in conformity to such a plan; often the railroads have determined the location of the towns. In such cases it is desirable, considering the volume of "through travel" in motor cars and trucks, to construct the roads in the most direct lines possible. This often entails rights of way running diagonally across the sections, "cut up" land holdings and makes trouble generally for the right-of-way department.

But when the rectilinear plan has been carried still farther and the land owners, to conserve particularly good areas for agricultural purposes, have had in times past enough influence to cause the county authorities to discontinue or vacate portions of ways along the section lines and have introduced right angled turns into the half or even quarter section lines, then the engineer has a task worthy of his mettle to secure a proper location for his improved road. And if the road be in an orange grove section, his joy is indeed complete.

The writer knows of a main paved road in one of the California counties which has at least ten right angled turns in it in a distance of about 20 miles and this road passes through no town or city and is practically level. In planning their new highway system several years ago, that county gave up as hopeless the task of securing a direct route in the locality referred to, so for many years to come all through travel over those 20 miles of beautifully paved highway must be subjected to the dangerous right angled turns and to the unnecessarily increased length.

There is reason in cities and other centers of population for ways laid out in rectilinear fashion. In the open country, there is no excuse for planning a new highway system along such lines. Land should be condemned if the owners will not donate it. There should be as direct a line between important centers as the topographical conditions will permit.

WIDTH.

Assuming that the best alignment for the highway has been adopted taking into consideration the factors of topography, climate and traffic needs, present and prospective, the next question confronting the highway engineer is the width of right of way.

It is certainly desirable that in any highway system the right of way be of uniform width but as a practical matter, each link in the system must be considered by itself. Near the centers of population it is obvious that the pavement and the rights of way must be wider than in remote rural communities, sparsely settled.

It is the writer's opinion, however, that for a minimum width of right of way 50 ft. is none too much and that wherever possible, a width of 60 ft. is the least that should be secured, even in sparsely settled localities.

It is inevitable that street railway, electric light and power, gas, telephone, and telegraph companies will at some time clamor for locations in the highway, and although too little attention has thus far been paid to the matter, tree planting and other landscape treatment of our country highways will have to be provided for.

In many of the older sections of the country right of way problems are not serious affairs. Ways have been established there, well defined and traveled, for many years, and right of way improvements consist chiefly in rectifying the side lines of locations where abutting land owners have encroached successfully under the "open adverse possession" statutes which apply in some of the States.

But in many localities, the acquisition of necessary easements of way becomes as important a factor in the plan and progress of highway work as the road work itself.

In the more sparsely settled communities, roads have been built following lines of least resistance, in the valleys the "sectionalized" land lines, and in the hills wherever the ranchers could best spare it. Accordingly, when modern road building methods are invoked, it becomes necessary to alter meandering and precipitous roads by strainghtening, widening, and improving the gradients. The needed rights of way for these purposes must be acquired.

LITIGATION.

This feature of the work is particularly annoying to the highway engineer. His desire is to press forward the best line in the best way in the best time. When he is confronted by a hostile, reluctant or indifferent land owner, the engineer usually loses his patience.

It is not alone in cases of new rights of way that there is litigation, but frequently old surveys do not exactly coincide with existing ways, many of which in course of usage have become winding and irregular, and consequently additional land has to be acquired to widen, straighten or alter them.

Owners often build fences or cultivate up to the used portion of the ways and resist the shifting of the lines and delay the progress of the work. In many cases much time is lost where owners who have allowed people to pass and re-pass in vehicles without objection for years, assert adverse claims and work must be delayed to avoid complications.

One has also the experience of attempting to use dedicated rights of way shown on plats recorded in times past but which have been entirely unused or allowed to fall into disuse, and then being confronted by claimants, with their attorneys, who contest the rights of the public therein.

There are many unavoidable delays in obtaining rights of way, arising outside of the disputed rights of way mentioned. Even when the owners intend to be liberal they exact a great deal of information before signing the deeds of easement. The records have to be searched to ascertain the true owners of the lands affected; owners must be notified or corresponded with; draftsmen are asked to furnish sketches to many owners defining the rights of way desired; visits to the lands must be made and surveys inspected; minor adjustments of lines and fences must be settled upon; vacation proceedings arranged and prepared, abandoning the old roads or portions of roads over property so as to leave no incumbrance on the same when the new road is located and built; co-owners must consult among themselves before executing deeds of easement; ownerships involved in probate proceedings or title litigation must be searched and a good title to the roads acquired out of the confusion, and there are other details ad infinitum.

These many difficulties have led, in the writer's western experience in highway work, to the employment of the subtle right of way man, who needs be a psychologist as well as a philosopher. His chief duty consists in attempting to wheedle the often-times contrary land owners into signing the needed conveyances and to convince them, usually, that their duty to the public lies in giving their property gratis. Such an employe becomes a very important member of the organization. His troubles are many.

In addition to the "right of way man" and his assistants in the California work, the help and advice of an attorney learned in eminent domain practice has been had who devotes all of his time to the highway work and whose principal activities are in right of way matters.

In many jurisdictions, if the deeds cannot be acquired by diplomatic methods, war must be declared in the courts, and the highway board must desist from its efforts to promptly furnish the community with necessary thoroughfares until the courts finally determine that the litigious land owners' holdings may be entered upon.

METHODS OF SECURING RIGHT-OF-WAY.

There is a great lack of uniformity in the different States in the methods of paying or securing the payment of damages in taking property for public highway purposes. Such methods are of course regulated entirely by the constitutions and statutes of the respective commonwealths.

In some States it is not necessary for the authorities to pay for private property taken for public use in advance of the actual taking of possession. The property owner has been provided with a method of making his claim and with a tribunal constituted so that he may enforce his claim and obtain his damages therein.

In such jurisdictions, highway work may speedily progress and the laying out of routes followed by immediate construction. The property owner, if he is dissatisfied with the original offer of payment of the award made to him by the public authorities, may pursue his remedy in the appropriate court, even though his land has already been occupied by the public.

The public has the advantage of celerity in the progress of its enterprise; the land owner is protected by ultimate and adequate compensation for his injuries, and in one State, at least, he may wait until after the State highway is completed before he must file his petition for jury trial, it then being evident to all interested parties just what damage has been done, not only by reason of the land taken but by the road construction as well.

But some States are so unfortunate as to be harassed in their public work by constitutions and statutes expressly requiring prepayment before entry upon the land required for public use.

The writer has had to do with highway activities in two States which have operated under each of these methods, the one having the right to take land necessary for public use in advance of satisfying the owner; the other requiring that if the owner is not pleased with the offer made to him by the public authorities, he may stand back on his property with a shotgun and compel public officers to initiate proceedings in the court and remain off his property until after judgment has been obtained and the assessed damages paid into court for his use and benefit.

In the first mentioned commonwealth, the welfare and progress of the people as a whole are superior to the notions and eccentricities of an individual land owner.

In the other States, the recalcitrant land owner may oppose and delay the vital needs of a city, county or State as the case may be, and his immediate rights predominate over the requirements of the community at large.

No rights of way, in States having regulations similar to the latter, can arbitrarily be taken by the people before the same, after a vast amount of red tape, have been acquired by donation, purchase or condemnation; that is, a taking cannot be made and compensation and damages adjusted afterwards.

Consequently obstinate land owners are able to "hold up" the community at large until it either pays the demands or contests the question of compensation and damages in trials, the latter usually requiring considerable time, particularly in the case of the belligerent or indifferent land owners residing in other States or foreign countries when long publications of summons are necessary before the suits may be commenced. The western States appear to be particularly oppressed by such roundabout methods of entering upon private property and installing improvements for the benefit and welfare of millions of people.

For illustration, under such a system a large western land owner owning an area equal in size to an entire eastern State may be luxuriously traveling abroad. A county has voted and issued bonds for a large amount to construct important highways. Before the great ranch can be entered upon, except for surveys, a correspondence must ensue between the public authorities and the land magnate. The owner declines to sign a conveyance and the people are compelled to commence proceedings in eminent domain against the absent owner. Before a trial can be had, summons must be published for sixty days, and then follow the tedious court proceedings.

It usually happens that pugnacious land owners demand some exorbitant sum. The court may upon trial only allow a small percentage of their original claim but during the pendency of the action an important artery of travel may be debarred.

Such a system is absolutely hostile to progress; the people should be greater than the individual.

The writer submits that at this time, when modern highway construction is becoming so active throughout the nation, it is apparent that there should be simplification in the constitutions and statutes relating to the subject of eminent domain, and that this Congress may render invaluable service in assisting to bring about so desirable a result.

ABSTRACTS OF TITLE

Too much attention can be given to the title technicalities of right of way activities. It has been an almost universal practice for public boards performing road work to obtain at great expense exhaustive abstracts of title to ascertain land ownerships.

The writer has had under his supervision the acquisition of hundreds of miles of highway right of way in California where the securing of rights of way could not be made much more difficult, complex or annoying, yet the purchase of expensive abstracts of title has been dispensed with. Out of hundreds of ownerships affected, not one serious complication has resulted from the following plan:

When the field parties are making the original surveys, the chiefs of party usually inquire from the occupants of the land surveyed who the owners or those interested in the property may be. This gives a clue to the ownership. Thereafter, one of the staff visits the proper county offices and ascertains from the assessment rolls or the records who purport to be the owners. Deeds or agreements are then prepared, containing the proper descriptions, and it is very rare, indeed, that any objection has been made to the accuracy of the instrument submitted.

By thus performing its own title searches, even though they may not have always been the most exact from a title lawyer's standpoint, the authorities have saved thousands of dollars and have never had an injunction or ejectment proceeding instituted against them by objecting land owners.

By taking a few remote chances of complaints, work, which would otherwise be hopelessly harassed and delayed in the performance of a highway project, may proceed. Furthermore, in most States, title may be obtained to ways by user or implied dedication by the passage of time. It has been the custom in California where the present traveled roads are wide enough for use and properly located, to place the monuments and build the pavements and assert jurisdiction thereover, the theory being that if the owner objects, the authority's title being fundamentally weak, the State can "condemn" as rapidly as the alleged owner can "oust."

CONTROL OF RIGHT OF WAY

The so-called State highways in the several States may be divided into at least two classes with regard to the control by the State of the roads after they are built, namely, those which are maintained by the State and over which the State assumes complete charge from property line to property line with the possible exception of the policing of the way, and those sometimes called State-Aid roads where the commonwealth has little or nothing to do with the maintenance of the roads and the burden is placed by law upon some subdivision of the State, usually the county.

The writer has had to do only with the class of State highways first mentioned and he believes that the State ought to have as complete control as possible over its highways, State or otherwise. Such control, however, places a considerable burden upon the authority which administers the law.

More is expected of a State organization, and rightly so, than of a county board. Its work must be done carefully and accurately. The surveys and plans of the State highways must be well made and no small part of the engineering costs is chargeable to the careful work needed in running out and establishing the right of way lines.

In trying to establish old right of way lines in anticipation of highway improvements, much difficulty is often experienced in finding any landmarks to indicate what the right of way really is, and the old surveys and plans often prove to be of little assistance. Often the roads to be taken over and built as State highways were laid out when the land was of little value and the surveys were carelessly made or the descriptions carelessly recorded. With the lapse of time buildings, trees, and other similar features, which formerly marked the location of the road, have entirely disappeared, and the traveled ways have shifted from place to place as the action of the elements or the whims of the travelers have directed. Fences, if they exist, have been so moved about that they in no way indicate the original line of the road.

In all State work with which the writer has had to do it has been the policy to fix the right of way lines on the ground by setting proper monuments into the soil to such a depth that they serve as markers for all time to come. In planning a new system of highways, careful plans should be made and permanent monuments set. Future generations will surely appreciate such records and the additional cost of this kind of work should not forbid.

Unit Costs of Bituminous Carpet Treatment and Bituminous Macadam by the Penetration Method on Two Roads in Illinois.

The unit cost of surfacing an asphaltic painted macadam road with a bituminous carpet and the cost of constructing a bituminous macadam pavement by the penetration method are given in a recent issue of the official publication of the Illinois state highway department.

BITUMINOUS CARPET TREATMENT.

The Fieldon road in Jersey County, Illinois, was first built as a waterbound macadam road in 1909 and 1910. In 1912 it was resurfaced, using a light application of asphalt, 0.6 gal. per sq. yd., and sand. The binder was of such a nature that it bled considerably during hot weather. The result was that the road soon became pitted and full of ruts. Mr. Warren, county superintendent of highways, influenced interested parties to furnish the necessary money to resurface part of this road.

The state highway department furnished the necessary equipment for surface oiling the road. On account of shortage in funds the old road surface was not scarified or disturbed, but the road was swept clean and a light application of asphalt applied to the surface and the road sanded with ⅛ to ⅜ inch gravel. The ruts were then filled with gravel and asphalt in successive layers until the same was brought level with the surface of the road. The entire surface was again painted lightly with the asphalt and sanded. The smooth surface required about ½ gal. per sq. yd., while the ruts required from ¾ to ½ gal. per sq. yd., making an average of about 0.8 gal. per square yard.

TABLE I.—COST PER SQUARE YARD FOR LABOR AND SUPPLIES.

Superintendence	[illegible]
Unloading, hauling and spreading gravel	[illegible]
Hauling, heating and spreading asphalt	.0176
Asphalt, f. o. b. siding	[illegible]
Sweeping and cleaning road	[illegible]
Gravel, f. o. b. siding	.0563
Freight on equipment	[illegible]
Roller operation	[illegible]
Total cost per sq. yd.	$0.3172*
Cost to local people	$0.2245

*Indicates paid by state highway department.

Conditions—

Excluding contractors' profit and overhead charges.

Amount of road repaired, 2,850 ft., 2,535 sq. yd.

Width of old road, 12 ft. Average width repaired, 8 ft.

Average haul for materials, 1½ miles.

Work began, Aug. 25, 1914. Work completed, Sept. 20, 1914.

Rate of pay for men, 20 cts. per hour; teams, 40 cts. per hour.

BITUMINOUS MACADAM BY THE PENETRATION METHOD.

The final unit costs of constructing the Daysville road in Ogle County, Illinois, are given in Table II.

TABLE II.—COST PER SQUARE YARD FOR LABOR AND SUPPLIES.

Superintendence	$0.0168*
Excavation	.0136
Stone, f. o. b. crusher	.0874
Miscellaneous hauling	.0072
Hauling stone, gravel and bonding material, including spreading	.1060
Hauling binder	.0060
Binder gravel, hauling, screening and placing	.0315
Binder, 35,000 gals., f. o. b. siding	.1680
Shaping and rolling sub-grade and side roads	.0101
Heating and applying bituminous binder	.0305
Rolling and sprinkling	.0068*
Bridge repairs	.0020
Freight on equipment	.....
Depreciation rental on equipment	.....*
Total cost per sq. yd.	$0.4859
Cost, excluding excavation, culverts and bridges	.4703

*Indicates furnished or paid by the state highway department.

Conditions—

Excluding contractors' profit and overhead charges.

Amount of road laid, 8,842 ft., 17,684 sq. yds.

Amount of road treated with bitumen, 17,684 sq. yds. Width of road, 18 ft. Thickness, 3 to 6 ins.

Average length of haul, 1¼ miles.

Work began, Aug. 18; work completed, Oct. 23, 1914.

Rate of pay for men, 25 cts., 30 cts. and 40 cts. per hour; teams, 50 cts. per hour.

Course in Road Building.—The University of Michigan will give a short course in highway engineering for the benefit of township, county and state highway engineers and officials. The course will be free, one week in length, and will consist of lecture and demonstrations. The regular highway engineering faculty will give the lectures, and the following also will address the class: F. F. Rogers, of the state highway commission; Prof. T. H. McDonald, head of the highway work in Iowa; Prevost Hubbard, head of the board of Industrial research at Washington; W. W. Crosby, consulting engineer of Baltimore; Dean C. H. Strahan of Athens, Ga.; Prof. Ira Baker, head of the department of civil engineering of the University of Illinois. No manufacturer of or agent for roadbuilding materials will be allowed to enroll for the purpose of getting his wares before the engineers attending. The intention of the university is to take up the problems of Michigan road building and Michigan material, and to go deeply into the question of economic road construction under the various conditions existing in this state. The course will be part of the extension work undertaken some years ago by the engineering department. The date will be announced soon.

Large Dredge in Toronto Harbor.—The Cyclone, the first of two large dredges to be built for harbor improvement work at Toronto, was recently launched from the Polson Iron Works of that city.

Although this dredge is surpassed in size by some of the dredges used in constructing the Panama Canal, it is said to be the most powerful dredge in the world. It has been built for the Canadian Stewart Co., which has the contract for improving the harbor of Toronto, a work requiring the removal of some 35,000,000 cu. yds. of sand from the bottom of the harbor to the Ashbridge Bay district, which is being reclaimed for an industrial area. It is understood that the Cyclone has the capacity for handling 15,000 cu. yds. a day.

The length of this dredge is 170 ft. and the width 42 ft. Steam is furnished by 4 Babcock boilers of a semi-marine type. At the end of the ladder, which is 100 ft. long, is a boring machine that weighs 50 tons. By means of a long stick of Pacific coast fir swung at the opposite end of the dredge, the suction pipe can operate in a channel 500 ft. in width. The boring mechanism is operated at the end of the suction line by a 500-HP. engine. The pump in the middle of the dredge, which is worked by an engine of 1,750 HP., can handle very large boulders. The dredge is equipped with a pipe line more than a mile in length.

SEWERAGE

New Sewage Pumping, Screening and Sterilizing Station at Daytona, Fla.

There is now under construction in the city of Daytona, Fla., a complete sanitary sewerage system, including a sewage pumping station and an outfall line, and a drainage system. The sewage collection system comprises about 21 miles of vitrified pipe sewers, ranging in size from 8 to 24 ins., with manholes, flush-tanks and other appurtenances, and seven ejector stations with cast iron force mains and air piping for supplying the same from the sewage pumping station.

The sewage pumping station, the substructure of which is shown in detail in the accompanying cut, comprises a concrete receiving well with a brick superstructure located on the city water works lot. In this station will be placed two 50-hp. oil engines, two air compressors of a capacity of 150 cu. ft. of air per minute each, three centrifugal pumps—two having a capacity of 2,000,000 gals. daily each and one a capacity of 1,000,000 gals.—one automatic sewage screen of the Riensch-Wurl type, and apparatus for sterilizing the sewage with liquid chlorine. The outfall sewer will be about 10,600 ft. in length, consisting partly of a cast iron force main and partly of vitrified pipe, the latter supported on piling. It will extend from the sewage pumping station to the southern city line, and thence to a point in the Halifax River about 500 ft. from shore. The drainage system will comprise about 7,500 ft. of pipe sewers, ranging in size from 12-in. vitrified pipe to 30-in. reinforced concrete, storm water inlets, and the widening of two canals.

Owing to the fact that the Riensch-Wurl screen is just becoming recognized in this country, it is thought the engineer's specifications governing the design and construction of the screen of this type, proposed for Daytona, will be of wide interest to sewage disposal engineers. Consequently that portion of the specifications is here quoted in full. The use of liquid chlorine as a disinfecting agent for raw and screened sewage is also still somewhat new, so the portion of the specifications pertaining to the sterilizing equipment is also quoted herewith:

SPECIFICATIONS FOR SEWAGE SCREEN.

General Description.—The screen is to be a device for the clarification of sewage by mechanical means. It is to consist of a rotating, inclined, perforated plate, 8 ft. in diameter, the lower part dipping into the sewage, the higher part being exposed and provided with a device for the continuous cleaning of the screen plate and removal of the solid deposits and the transferring of same to suitable transporting or containing device. The screen is to be of the overhung type. The entire weight of the screen and driving mechanism to be supported by a service bridge located directly over the screen. The screen shaft and brush shaft will each be held in position by a cast iron hanger attached to the service bridge. The service bridge will also serve as an operating platform upon which the lubricating devices will also be located. All bearings and driving mechanism will be supported from the service bridge, no bearings or any part of the driving mechanism to come in contact with the sewage.

Screen Plates.—The screen plates are to have perforations of the size hereafter specified and removably fastened in sections to a stiff cast iron frame carried on bearings of the ball or roller type. The screen plates must be fastened to the frame in such a manner that the maximum effective open area of the screen plate is maintained as hereinafter specified. The clearance between the screen periphery and the screen chamber must be reduced to a slit not exceeding in width that of the screen perforations. This clearance must be constant the full distance of the screen perimeter.

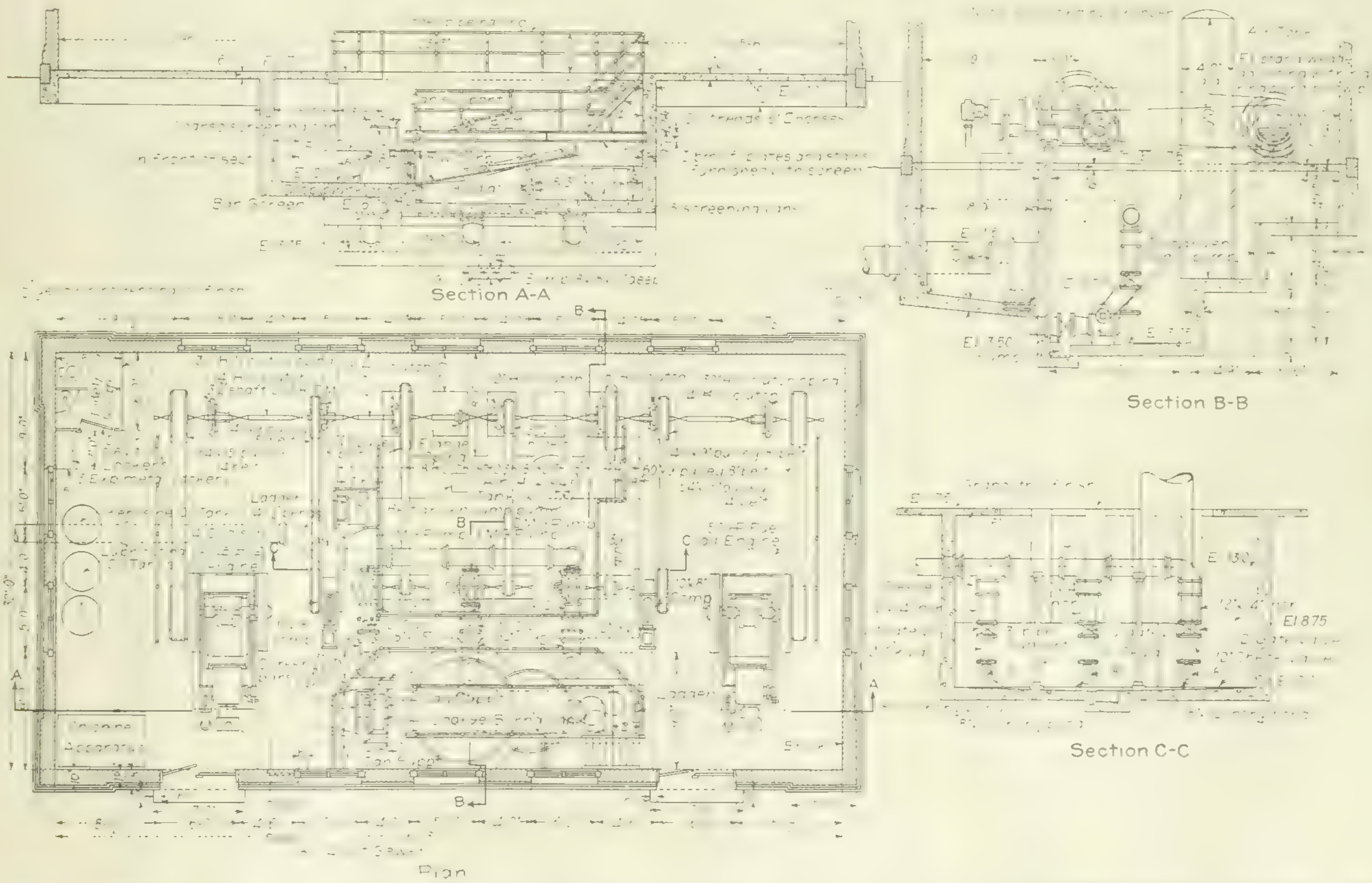
Lubrication of Screen.—The lubrication system must be complete in every detail. Wherever possible, oil bearings are to be fed and regulated from a central point.

Performance of Screen.—The screen will be required to remove from the sewage all solids sedimentable and non-sedimentable above 0.1 in. diameter, and must deliver these screenings to the transporting or containing device with a minimum moisture content, accomplishing these results with practically no disintegration of the screenings.

Method of Operation.—The screen is to be operated by belt power as hereinafter specified, the main driving pulley to be located on one side of the screen. The screen disc is to rotate at a speed of approximately 0.5 revolutions per minute.

The screen must be arranged so that the sewage flows against it in the inclined position, effluent passing on through the screen to be discharged, and all solids of the size herein specified being held back. The rotation of the screen is to lift the solids gently and continuously from the sewage until they are above the line of flow, where they will come into the cleaning sphere and be continually swept from the surface of the screen by a series of rotating brushes.

These brushes are to rotate around the vertical axis of the brush shaft at a speed of 7.5 revolutions per minute, with a direction of rotation opposite to that from which the screen disc [illegible]. At [illegible] the brushes are to revolve on their own longitudinal axis



Details of Sewage Pumping Station Substructure and Machinery, Daytona, Fla., Showing Arrangements for Fine Screening and Sterilization.

[illegible] per minute, and must rotate in such direction and be suspended from the spider carrying them in such a manner that their action is to avoid forcing screenings down through perforations, but rather to lift them off the disc and roll the same along ahead of them.

The method of hanging the brushes must also give a constant brush pressure on the screen plate and compensate any unevenness in the disc.

The relative speeds of the disc brush cleaning mechanisms are to be adjusted so that all parts of the screening surface are cleaned and wiped at least three times during each revolution of the screen.

Workmanship.—The workmanship and finish of all parts throughout shall be highest grade in every respect. Working parts shall be machined accurately to gage and be made interchangeable without any further fitting. No variations from thicknesses or finishes shown on drawings, shims, filling pieces or patches will be allowed except in special cases where specifically approved by the engineer.

Any parts that should be finished, either by machine or hand, to insure the proper assembly or working of the various parts must be finished even if not shown or described on drawings or in specifications.

Details of Construction.—Screening plates to consist of six monel metal plates approximately 4 ft. by 3 ft. 5 ins. by ⅛-in. thick. Plates must be cut true to size and lie perfectly flat.

All sections must fit nearly, no space being allowed in which solid matters may accumulate. Plates to be arranged so that each section will be readily removable.

Screening plate perforations to be rectangular, 5/64 ins. wide by 2 ins. long, tapered slots. All perforations to be machine cut. Both sides of the plate to be free from burrs. Perforations must be arranged so that in the operation of the screen the maximum number will have their long axis parallel with the travel of the cleaning brushes. Perforations must be arranged to give an effective open area of at least 15 per cent of the total screening surface, and they must be staggered.

Screening Plate Mounting.—Plates to be fastened to the screen frame by 5/16-in. flat head countersunk brass bolts and nuts. Brass washers ¼-in. thick must be used on each bolt between the bolt and the screen frame. All screens must be finished perfectly flat and smooth with the face of the screen plate.

Screen frame to consist of a cast iron spider supported from a central hub. Spider to have twelve T-shaped arms extending out to a ring to which the screen plates will be attached. The frame must be amply stiff to retain its shape in its inclined position and also to resist the thrust imposed on it on account of the flow of sewage. There must be no pockets or ledges on the frame upon which solids may be retained.

Cone—The cone is to be made of cast iron and will be of the solid type of construction, measuring about 2 ft. in diameter at the bottom and 4 ins. high.

Shafting.—Main drive shaft, countershaft, screen shaft, brush shaft, brush spider shaft, brush body shaft, will consist of cold-rolled steel stock, finished all over and perfectly true. All bearing portions are to be accurately turned. Shafts to be provided with necessary key-seats and thrust collars.

Bearings.—Main drive-shaft bearings, countershaft bearings, screen-shaft bearings, brush-shaft bearings, are to be of ball-bearing or roller-bearing type, radial or thrust bearing, with cast iron housing. Necessary provision to be made for lubrication. All bearings to be grease-packed and sealed where necessary.

Screen-Shaft and Brush-Shaft Hangers.—Screen-shaft and brush-shaft hangers are to be made of cast iron and are to be bolted to the service bridge. Hangers to be of sufficient size and strength to support the weights of the screen and brushes rigidly and maintain all parts connected thereto in perfect alignment. Thrust bearings and radial bearings for the brush and screen shaft are to be bolted to these hangers.

Brush Spider.—In order to carry the brushes from the brush main driving shaft, a supporting iron casting is to be furished consisting of a hub and four radial arms, and is to be a tight fit on the driving shaft and keyed thereto. The arms are to be cast in "U"-shaped sections, provided with bearing at each end of support of brush main shafts.

Screen Brushes.—Screen brushes are to be four in number, each consisting of an aluminum body cylindrical in form. The outside surface of this cylinder to be drilled and countersunk to receive circumferential rows of bristles. The brushes will be equipped with best quality of wild hog bristles, will be wire-drawn into the body of the brush and trimmed so as to project 1¼ ins. above the circumference of the outside of the brush body. The brush is to be carried on cast iron hubs, mounted on and fastened to the brush body shaft.

Screening Trough.—Screening trough for receiving and guiding the screenings after they are swept from the disc is to be constructed of steel plate and vertical walls to be concentric to conform to the radius of travel of the brushes and provided at some point in the path of travel with a discharge opening. Trough to be braced and stiffened with necessary stiffening angles and supports provided for holding trough securely in place.

Driving Mechanism.—The screen is to be arranged to drive from an 18 by 3-in. tight and loose pulley mounted off to one side of the screen and running at a speed of 150 r. p. m. The main driving shaft to extend from this pulley to a worm-gear speed reduction, ratio 50 to 1. The worm gear is to drive countershaft running parallel with the counter line of the screen, which in turn drives the screen-shaft through a pair of bevel gears, ratio 6 to 1, and also drives the brush-shaft through a pair of bevel gears, ratio 25 to 1. Worm to be provided with oil pot for lubricating of worm and gear. Worm to run immersed in oil.

The disc brushes are to be driven by means of a fixed bevel gear, which is to be fastened to the brush-shaft hanger and meshing with bevel pinion on the end of the brush spider shafts. The brush body shaft to rotate in a direction opposite that of the screen and the brushes are to revolve on their own axes at a speed of about 60 revolutions per minute.

The brushes are to be driven by a set of spur gears and mounted on a swivel which will allow the brushes to trail, this giving a constant brush pressure on the screen plate and compensating for any unevenness in the disc.

The entire driving mechanism must be designed and manufactured to secure the maximum efficiency and screen must operate without undue vibration under all conditions.

Screen Seal.—The clearance between the periphery of the screen and the screening chamber must be reduced to an air gap, not exceeding the width of the screen plate perforations. This is to be accomplishd by means of a 3-in. channel iron set into the concrete, bent to conform to the shape of the perimeter of the screen. This channel to be secured to the concrete by ½-in. approved anchor bolts or sockets. The upper flange of the channel is to be drilled and tapped on approximately 8-in. centers for ½-in. phosphor bronze tap bolts; upon this are to be mounted 14 cast iron segments having elongated [illegible] to permit adjustment.

Screen Service Bridge.—Screen service bridge to consist of two I-beams of suitable size for carrying the entire weight of the screen and driving mechanism. The space between the beams to be framed with 4-in. channels and covered with approved cast iron or pressed steel floor plates. Framing for supporting the service bridge is not included in this contract. The bridge is to be provided with trap doors for removing sludge cans and sides of bridge to be provided with 1½-in. galvanized iron hand rail of approved design.

Lubricating System.—Lubricating system must be complete in all details. Bearings of medium speed, lubricated with oil, are to be provided for ring oiling. On slower running bearings the oil will be fed and regulated from a central lubrication system on the service bridge. Here all the oil cups to be mounted together. The ball or roller bearings, wherever possible, are to be packed in grease and sealed to exclude foreign matter and sewage.

Shop Assembly.—On completion the screen must be assembled in the shop, the parts properly fitted and tested. The disc of the screen is to be then assembled on the shaft in a vertical position and properly balanced, this being done while all parts are mounted on it that will be used in operation. After this the screen must be inclined to correspond at an angle at which it will be finally installed. In this connection, the balancing is to be as near perfect as possible to provide against any uneven running or variable strains in the shaft when operating in this position. After assembly screen is to be dismantled, every part being properly marked to facilitate re-erection.

Painting.—Before leaving shop all exposed castings shall be filled and rubbed down smooth and given two coats of paint as instructed.

Shipment.—All parts must be properly packed, skidded, boxed or crated, and prepared for shipment. Especial attention must be given to the manner of shipment and the size of the pieces shipped so that there will be no trouble getting the screen parts into the screening chamber. The finished surfaces to be properly slushed.

Screening Cans.—The contractor shall also furnish and deliver one screening can for the coarse-bar screen, as indicated on the drawing, made of No. 16 gage galvanized iron. He shall also supply three screening cans for the fine screen, 24 ins. in diameter by 30 ins. high, of No. 16 Birmingham wire gage. All cans shall be provided with lids, lift handles and suitable lifting hooks. The contractor shall also supply two lifting devices, each consisting of a small jib crane suitable for attachment to the wall, one to lift out the coarse-bar screening can and one to lift out the fine screening cans. Each jib crane shall be provided with a suitable chain hoist.

STERILIZING EQUIPMENT

The following quoted paragraphs comprise the engineer's specifications for the sewage sterilizing equipment:

Chemical Equipment.—The contractor shall furnish and install in the sewage pumping station equipment equal to that of the Electro-Bleaching Gas Co. suitable for the treatment of raw sewage with liquid chlorine. The apparatus shall be capable of feeding chlorine gas at any desired uniform rate between the limits of 10 lbs. per 24 hours and 150 lbs. per 24 hours. The delivery of the chlorine must be controlled by means of simple devices which will afford easy and reliable operation in spite of temperature changes. The installation must be so arranged as to allow the ready removal of empty and insertion of full gas containers. The system shall be equipped with necessary reducing and regulating control valves, together with gages, and all other necessary appurtenances, all of which shall be of proper design and of material particularly adapted to use in connection with chlorine gas. The apparatus shall provide for a perfect and absolute absorption of the necessary amount of chlorine in a minimum quantity of water without escape of gas to the atmosphere. All connections shall be made of suitable piping to allow of both sterilizing the raw sewage entering the pumping station before screening and the clarified sewage after screening.

Operation of Chlorine Apparatus.—The contractor shall control the operation of all equipment for a period of 15 days after erection, and shall operate the various devices to the satisfaction of the engineer. The chlorine gas required for this test will be furnished by the city. The contractor shall also furnish detailed written instructions for the operation, care and maintenance of the apparatus.

Guarantee of Chlorine Apparatus.—The contractor shall guarantee to keep all the apparatus in repair for one year after it is put in regular service, such repair extending only to the correction of defects in the design, material and workmanship of the apparatus, but shall not be held to cover ordinary wear and tear. At the end of the guarantee period the engineer will make an examination of the apparatus, and any part or detail found defective or injured through excessive wear, overstrain, bad materials or faulty design shall be replaced by the contractor at his own expense. The contractor shall assume all responsibility for infringements or alleged infringements of patents of this apparatus.

ACKNOWLEDGMENT.

The foregoing quotations are from the set of specifications prepared by Mr. George W. Fuller, consulting engineer, 170 Broadway, New York City, and Mr. C. M. Rogers, engineer, Daytona, Fla. We are indebted to Mr. Fuller for a copy of the specifications and for the detailed drawing of the pumping and screening station substructure and machinery.

Design of the Sewage Treatment Experimental Plant at Brooklyn, N. Y.

The design of an experimental plant for the study of sewage disposal offers many complex problems and presents an interesting study in itself to the municipal engineer. The conditions that affect sewage disposal are so many and so various, and the methods of treatment which may be applied differ so greatly, in cost as well as in the results secured by their employment, that it has become quite usual to make experimental study in each case before deciding upon the method to be selected and the design of extensive plants. The engineer in this field owes much to the many investigations which have been made by the use of such plants during the last few years. One of the conclusions that each of these plants enforces is that each was of special value for the local object, and that the solution of the local problem could best be solved by local experimentation.

It is generally believed that the description of sewage treatment experimental plants is of greater general value than the specific results of the experiments performed. This is due to the fact that the latter are chiefly significant only to the locality whose sewage is tested. The present article describes the design of the sewage treatment experimental plant at Brooklyn. The information given is taken from a paper before the recent annual meeting of the American Society of Municipal Improvements by George T. Hammond, Chief Engineer of Sewers, Brooklyn, N. Y.

The data obtained by the Metropolitan Sewerage Commission, conclusively prove the urgent need of adequate sewage disposal in New York City; but no experimental work for the purpose of determining the most available method of sewage treatment has hitherto been undertaken. The sewage treatment and disposal which experiment shall demonstrate will be the most reliable, economical and simple and which will, without offense or nuisance, for the least cost of construction and maintenance, insure an effluent, satisfactory and suitable at all times for discharge into our local bodies of water or waterways, we consider the most available for the purpose.

Naturally the scope of the experimental work for which this plant is designed includes investigation connected with sewage disposal generally, as well as the special and practical experiments which are more directly related to the special object of selecting the most suitable method of treating the local sewage. The general study includes rainfall and storm-water runoff; subjects so closely related to sewage treatment and disposal that their investigation could not properly be omitted, especially as the larger portion of the existing sewerage is now, and probably will continue to be, on the "combined" plan. Of course the investigations include a complete analytical study of the sewage and storm-water flow, and such biological study as may be necessary for the end in view. The chemical and bacteriological work has been carefully provided for, a completely equipped laboratory being an essential feature of the plant.

It is intended, in the experimental work, not only to follow the general scheme of investigation for which immediate provision was made in the structures now erected, but also to change the structures, from time to time, as may seem desirable to follow other lines of investigation suggested as the work progresses.

It is not intended in this paper to present results which at this time could only be derived from a short period of experimental operation; our object is to describe the plant and the general purpose of the work; and call attention to the employment of such means for solving sewage problems.

The experimental plant is located in close relation to the existing 26th Ward Sewage Disposal Plant, and obtains its sewage from the main sewer, which serves the 26th Ward district of Brooklyn. The sewerage system, for which this sewer is the outfall main, serves a population of about 200,000, occupying an area of approximately 5,000 acres. The sewers are on the combined plan and have rather flat grades. The disposal plant is about two miles from the center of population and the sewage becomes septic, during the warm months of the summer, before reaching the outlet. A new disposal plant is urgently needed for this district, and has been authorized, the design to await the results of experimental study to determine the most suitable method of treatment for local conditions.

The daily flow in dry weather, determined by weir measurements in the outfall, varies from 18,000,000 to 22,000,000 gals. The storm-flow ranges up to about 1,000 cu. ft. per second, though the ordinary storm does not give more than 300 to 500 cu. ft. per second. The dry flow is mainly of a domestic character, but there are some trade wastes that may require special study. The suspended matters vary widely in quantity at different seasons and different hours of the day.

The sewerage system of the 26th Ward was mainly installed between the years 1890 and 1896 and includes a chemical precipitation sewage disposal plant, designed in 1888 and intended to take care of a maximum population of 35,000, which, at the time, was considered ample provision for the future. This plant was completed in 1896, when the population had already reached about 60,000, and was inadequate from the beginning of its operation, although, for several years, it rendered fairly good service. It cost $350,000 and is, at the present time, nearly useless. The grade of "purification" secured was never satisfactory for an effluent to be discharged into a body of water of the character of Jamaica Bay, in which the shell fish industry is extensive and of great value. All the sewage passing through the plant flows through fixed screens, through long, narrow settling tanks, with considerable velocity, to a central well, from which it is pumped into the outfall sewer; the pump capacity is about 16,000,000 gals. per day. The storm-water flows directly into the bay without treatment or screening.

The area of Jamaica Bay and its tidal tributaries is 19.28 square miles, at mean tide level, and the tide range is about 4 ft. It is a tidal reservoir connected with the ocean by Rockaway Inlet, which affords a rather narrow and deep channel. The greater portion of the bay has a depth, at low water, of 2 to 5 ft., with extensive flats along the northerly portion, which have become considerably polluted with sewage sludge near the outlets of the sewers. The bay contains numerous small islets and mud flats, bare at low tide, and extensive bars used in the cultivation of shell fish. The sewer outlets are remote from the shell fish beds at present worked.

The main sewer above referred to is a combined sewer with flat grades; it is a twin sewer of sectional area equivalent to a circle 15 ft. in diameter.

EXPERIMENTAL PLANT.

The experimental plant consists of three Imhoff tanks, each of different depth but with other dimensions equal; six sprinkling filters, two of which receive forced aeration within the mass of the medium; various tanks and apparatus for the investigation of sewage treatment by forced aeration; secondary settling tanks for the sprinkling filters and the sewage treated by forced aeration; a plain settling tank for crude sewage in connection with an airtight sludge digestion tank, which receives the settled matters and sludge from the plain tank, being in effect the two essential portions of an Imhoff tank separated; a roughing filter; ten sludge drying beds; various experiments for screening sewage, and for drying sludge, including a fixed mesh screen, a rotary di c type screen, and a canvas vacuum filtration system; various provisions for disinfection experiments, etc. The mechanical plant consists of steam-actuated sewage pumps and an air compressor.

The twin sewers pass, at the location of the existing disposal plant, through a silt basin which, in dry weather, acts as a grit chamber for the sanitary sewage. This basin, which is covered with a masonry roof carried on I beams and piers, is 80 ft. by 60 ft. in interior dimensions, and 9 ft. deep. The line of the sewer is continued through it by the outfall sewer, the invert of which is 2 ft. higher than the inverts of the twin sewers. The outfall sewer is a single section, 26 ft. wide, of the same discharge capacity as the twin sewers, and for the first 300 ft. is of masonry with a flat roof supported by I beams, after which it becomes an open timber structure which crosses the salt marshes, about 4,000 ft., to Jamaica Bay.

A passage 48 ins. in diameter is provided with its invert at the floor elevation of the silt chamber, for carrying the dry weather flow into the existing sewage disposal plant, where after having received its charge of milk of lime, and having passed through the screens and tanks, it flows into the central pump well, from which it is pumped, against an average head of 20 ft., into the outfall sewer, about 100 ft., downstream from the silt basin. A low dam, 9 ins. high, is provided in the outfall sewer to prevent the sewage, pumped from the well, from backing up into the silt basin. During storms the sewage by-pass valves are closed and the entire flow of the combined sewer discharges directly from the silt chamber, over the dam referred to, through the outfall sewer to the bay.

The pumps that furnish the sewage for the experiments are located within the existing disposal plant building, and steam is obtained from the boilers of the plant. There are two sewage pumps, both of them direct acting piston pumps, installed so that either pump may be cut out and cleaned or repaired without stopping the other; either pump may be operated alone; or both may be operated at the same time. The larger pump has a capacity of 1,200,000 gals. per day, and the smaller 650,000 gals. Provision is made for placing a movable screen around the lower end of the common 12-in. suction pipe through which both pumps operate. This screen consists of a steel frame, upon which wire meshes of varying fineness can be used. The screen is cleaned by means of a water jet and brush.

The sewage for experiments is taken from the sewage passage between the silt basin and the disposal plant, and is practically free from grit. It is discharged from the pumps through an 8-in. iron pipe into the quieting tank, which supplies sewage to the various units of the experimental plant by gravity.

As the experiments include an extensive study of sewage aeration by means of compressed air, a sufficient supply of air is of great importance. The compressor installed is a duplex crank and fly wheel machine; automatic in starting, stopping and speed; with a displacement capacity of 228 cu. ft. of air per minute at not exceeding 210 r. p. m. for 30 lbs. air pressure, with 100 lbs. of steam at the throttle. It is equipped with a combination speed and pressure governor, arranged to bring the machine to a dead stop if no air is demanded, and to start up automatically when the pressure drops. It has enclosed splash lubricated frames and bearings, making lubrication automatic. The air cylinders are provided with water jackets. The air passes from the compressor to an air receiver, of steel, 24 ins. in diameter and 72 ins. high, with pressure gage and safety valve.

The low elevation of the ground at the site

—but a few inches above ordinary high tide, a tidal marsh in fact necessitated the design of an experimental plant above the reach of the highest tide, and pumping of the sewage up to the required level from which a gravity flow could be obtained for every unit of the plant. This was not only the most available design, but afforded some advantages, such as the possibility of getting at the structures from the sides, in most plants inaccessible on account of being placed underground.

The datum line was at mean high water and the surface of the sludge beds made at an elevation of 2.67: all of the other units of the plant were given such elevations above this as their operation required. Every unit was provided with a measuring device for the determination of the quantity of flow, usually consisting of an adjustable calibrated orifice above which a constant head is maintained by a system of overflow weirs. For measuring compressed air, venturi meters were provided. The accuracy of all the measuring devices was carefully tested in place.

The first thing determined, in making the design, was the required elevation of the surface of the sewage in each unit of the plant. This determined, the unit was designed to comply with it; and to support the structures in the situation selected, pile or other foundations were provided, as required. The design of such structures as the sprinkling filters, carried on piles, which in the permanent plant might be of concrete, with the peculiar form of "slab-between-piers" for outside walls, to secure the lightest construction with the largest possible aeration surface from the sides, was considered a study no less interesting than the treatment of sewage in the filters themselves. The above ground construction of the plant affords opportunity for studying the flow of sludges of different kinds, Imhoff especially, which could not as easily have been observed otherwise. The effect of cold weather on exposed sprinkling filter beds; and the danger of freezing of the various channels carrying sewage, and the proper method of protecting and operating the same, are incidental studies of great importance in view of the projected construction of a plant on piles over an extensive marshland.

Early in the studies which preceded the design of the experimental plant it was of importance to ascertain the quantity of sewage flow per day and per capita from the drainage a d sewer district, the entire flow passing into Jamaica Bay at this point through the main sewer, as already described. This was done by means of a knife-edge weir with end contractions suppressed, installed in the open outfall trunk sewer. This weir was of notable length on the knife-edge, 26.83 ft. The experimental nature of the work indicated that all of the structures provided for the plant should be so designed and built that changes and variations might be made, without great cost or difficulty, to carry out more fully such variations in the investigations as may seem of interest or may be suggested by the tests as they proceed.

Quieting Tank.—The quieting tank stills the flow coming from the pump, and supplies a regulated supply of sewage to every part of the plant, maintaining a constant head in the supply at an elevation of 33.42. It is placed on top of a strongly braced platform. This entire structure is built of timber. The supporting posts are 6 ins. by 6 ins. yellow pine, and the tank walls are of yellow pine 2½ ins. thick. A platform around the tank is provided for convenient inspection and operation, which is connected by means of a bridge with the tops or "decks" of the three Imhoff tanks. A handrail is also provided around the platform for safety. This tank is rectangular in shape, 5 ft. in depth, and 12 ft. 3 ins. by 8 ft. 10 ins. in plan. In its interior each end is divided off by a partition, forming overflow chambers, which are connected with the waste pipe; the main chamber is between these internal partitions and is 9 ft. in length by 8 ft. in width. Each partition is cut down to the waterline a distance of 3 ft. from the outlet side of the tank, forming overflow weirs designed to maintain a constant head over the outlet orifices. Two baffles are placed lengthwise of the tank, between the above mentioned partitions and normal to the line of flow. The baffle nearest to the entrance of the sewage extends 3 ft. upward from the bottom, and is 2 ft. 6 ins. from the entrance side of the tank. The other baffle extends downward, from the top of the tank, to within 1 ft. of the bottom, and is 2 ft. 6 ins. from the first baffle, parallel with it, toward the outlet side. Thus, the sewage which enters, submerged, raises over the first baffle and then passes downward, under the second, before entering the constant level chamber from which it is fed, through the adjustable orifices, to the various units.

In the side of the main chamber of the tank, opposite to the entrance of the sewage, are placed six outlets, each provided with an adjustable orifice having a calibrated scale, which may be set to any rate of discharge within its range of capacity. These orifices discharge into flumes that lead to the various experimental units, each orifice into a flume which it serves. The method of measurement employed depends upon the use of a graduated adjustable orifice, discharging under a constant head, calibrated in place by actual measurement of the discharge in one of the large tanks emptied for that purpose. The means of maintaining a constant head is, in all cases, an overflow weir, over which a surplus flow is kept wasting to a lower level, where it may be used, if required, for other experiments or led back to the main sewer. As the supply pumps are steam operated piston pumps, capable of adjustment to the requirements of the experiments, no great amount is wasted, and no difficulty is experienced in keeping the head constant. This method of measurement was adopted as part of the design of the plant after extensive investigation of possible methods of measuring sewage under the circumstances of these tests. The orifice boxes with their overflow weirs were built by the contractor for the experimental plant; but a separate contract was made for supplying and installing the orifice fixtures themselves, and the Venturi meters for measuring compressed air, including their calibration in place, with the Wallace & Tiernan Co., Inc., of New York. The smallest orifice called for was required to measure flows ranging up to 150,000 gals., the largest up to 600,000 gals., in 24 hours. The adjustable orifices are made of bronze, all sliding parts are machined so as to work easily and yet fit tightly. They are made according to the principle of hydraulic discharge; but the discharge, as actually measured, varied somewhat owing probably to the velocity of approach through the 2-in. planking of the wall of the constant level boxes, on the outside of which they were placed.

Three of the flumes leaving the quieting tank pass to the Imhoff tanks, each tank having its individual flume by which it receives sewage from the quieting tank. These flumes are supported on a bridge or platform carried on angle iron supports projecting from the sides of the Imhoff tanks.

Imhoff Tanks.—The Imhoff tanks, three in number, differ only in depth, each being provided with a sedimentation chamber with the depth proportionate to the depth of the tank in which it is placed, so that the effect of depth (other conditions being equal) may be observed in the comparative performance of the tanks.

The scum boards, placed 12 ins. from and opposite the entrances and exits of the tanks, are in all cases 2 ft. in depth, and no other baffling is provided for in the first series of experiments; the aim being to subject an equal flow, disregarding theoretical retention period, in each tank in parallel operation to the same baffling. The dimensions of the horizontal section of each sedimentation chamber, at the lower edge of the scum boards, are equal in every particular in all the tanks; and the entrance and exit weirs are of identical design and dimensions. When the experiments upon the comparative effect of depth in sedimentation chamber, and sludge digestion chamber, have been completed, baffling, will be provided in connection with experiments upon rates of sedimentation at various rates of flow. If found desirable the sedimentation chamber of either of the tanks, or, indeed, of all, can be temporarily removed and a rearrangement made for Dortmond tank experiments. Connection can be made between the tanks so that the whole three can be operated in series, either as Imhoff or as Dortmond tanks, by a few inexpensive changes.

The tanks are of pine staves 3 ins. thick, with round iron hoops. Each tank is 15 ft. in internal diameter, and rests upon a yellow pine platform, supported by caps carried on piles. The inlet and outlet of each tank is made of galvanized iron, forming a distributing or a collecting chamber, as the case may be, in front of the entrance weir, or outside of the exit weir, which weirs are both full width of the sedimentation chamber; the aim being to distribute the flow uniformly, and to take off the effluent in the same manner.

The material of construction of the sedimentation chambers is pine, so put in as to be easily removed and reconstructed, as may be desired in the course of the experiments. The sloping floors, which are of 2-in. pine, planed smooth, were placed first, after which the vertical sides were put in place, which are of 1¼-in. pine, planed smooth, their lower ends resting upon the sloping bottom planks, so as to eliminate a vertical joint. These vertical walls are 10 ft. 8 ins. between sides and a maximum of 2 ft. between these and the outer shell. The sides and sloping floor are of tongued and grooved boards, carefully matched and smoothed inside. The spaces between the vertical walls and outer shell serve as gas outlets for the digestion chamber.

In making the openings from the sedimentation chamber for the passage of settlings into the digestion chamber, the inclined floors do not lap or pass, the one below the others, as is frequently the case in such tanks, the intention being to study the form of opening by changing it, perhaps several times; the form at present installed having been considered the most troublesome to try out was selected for the first trial. There is some reason to think that this form of opening is less liable to become clogged than the form ordinarily used, and in six months' service it has worked so well that no collection of settlings has at any time remained upon the slopes. The opening is guarded by means of a timber shield or baffle board placed below it with its upper slopes the same as the bottom slopes of the sedimentation chambers, through which the sludge pipe passes at the center of the tank, care having been taken to make the pipe passages gas tight through the wood. The sludge pipe in each tank is 8-in. iron pipe with a flange at the top, provided with a cover, and is carried by two 6-in. by 8-in. yellow pine timbers crossing the top of the tank; the pipe is suspended from iron channels crossing from timber to timber and placed under the flange at the top. The sludge pipe is provided with a branch pipe for drawing sludge, which passes through the side wall of the tank, having a gate valve and being connected with the sludge outfall channel into which it discharges.

The bottom of each digestion chamber is formed inside of the cylindrical tank, in the shape of an inverted truncated hexagonal pyramid, made in two sections, the upper overlapping the lower. A perforated lead pipe, 1¼ ins. in diameter, connected with the city water supply, and controlled by a gate valve, is placed entirely around the tank, under the overhanging edge of the upper pyramid, for use to start the sludge sliding down the slopes, if necessary, and for cleaning the slopes.

The effluent from the tanks may be distributed by gravity from the outlets to all the experimental units where its use is required. As there will, at all times, be a considerable surplus of effluent, provision is made to return this to the main sewer by means of a waste pipe controlled by a gate valve leading from the outlet trunk of each tank. As these waste pipes, which are all of iron, 4 ins. in diameter, are led down the sides of the tank they afford branch connections, provided with gate valves, for tapping the tank at lower points. Thus it is possible to entirely discharge the contents of the sedimentation chamber without disturbing the sludge digestion chamber. It is also possible either to obtain samples of sludge from the bottom of the tank without the disturbance that would be caused by drawing through the 8-in. sludge pipe, or to discharge the entire contents of the whole tank when repairs or alterations are required, through this lower outlet.

The tanks are covered on top, except for the opening over the inlet and outlet weirs, with movable floors, made of 2-in. pine plank, in sections, with lifting rings for handling. The floor affords an acceptable deck, which is reached by means of a stairway from the main floor; it is surrounded with a handrail, and bridges are provided between the tanks which afford easy access for taking samples or conducting the tests, as well as for the convenience of visitors.

All of the Imhoff tanks are wooden cylinders 15 ft. in internal diameter, and they differ only in vertical dimensions and cubic capacity. The bottoms of all the tanks are exactly alike in all dimensions, as are the tops, the inlet and outlet weirs, and the scumboards. It will be necessary, therefore, only to mention the dimensions in which the tanks differ, the depths being given inside from the waterline downward:

	Depth of tank at center.	Depth of sedimentation chamber. Vertical side.	Center.
Tank No. 1...	30.38 ft.	9.22	13.97
Tank No. 2..	21.88 ft.	5.30	10.05
Tank No. 3..	13.67 ft.	2.42	7.00

As these tanks are constructed of timber, changes in the interior arrangements for experimental purposes, such as increasing or diminishing the degree of the sloping bottoms, both of sedimentation and of sludge digestion chambers, are easily possible; also changing the design and width of the opening between the upper and lower chambers. The intention is to vary these parts during the course of experiments for purposes of study. With this end in view the slopes were provided as flat as was thought safe, but so far they have not retained any sediment and probably might have been flatter without causing trouble. The tanks have operated since Oct. 4, 1913. Probably the smooth timber surface affords less friction to the sliding of settled matter than would concrete. It is intended to cover the slopes with a concrete surface, before the completion of the experiments, to investigate this question.

The effluent from the Imhoff tanks flows by gravity to the following units, all of which may be simultaneously in action: 1. The sprinkling filters and roughing filter. 2. The aerating tank. 3. The aerating siphon. 4. The sprinkling filters with compressed air aeration. 5. To a secondary settling tank. 6. To disinfection tanks. 7. To mechanical vacuum filter tanks of the Moore pattern.

Sludge for testing and drying is discharged to the Imhoff drying beds; the surplus sludge is washed into the main outfall sewer.

Plain Sedimentation With Sludge Digestion.—Plain sedimentation experiments are provided for in a tank of the Dortmond type, which is one of a group of four tanks, each of the same size and design, constructed of concrete, 8 ft. by 8 ft. in interior plan and 8 ft. deep from the waterline in the center, the bottom being designed of pyramidal form. The other three of these tanks are used as secondary settling tanks for observing the effluents of the aerating siphon, aerating tank, and the Imhoff tank. The flow, entering, is carried down under the center of the tank and, raising, is taken off through V-shaped notches, of which two are provided on each side, into wooden troughs which completely surround the top of each tank. A 6-in. sludge discharge pipe is placed in the center, terminating with a bell at the bottom and provided with a clean-out at the top of the vertical portion above the water surface. Sludge is discharged through a horizontal branch, passing through the tank wall below the waterline, into a flume that carries it to the sludge beds. The tank selected for raw sewage sedimentation has, in addition, a sludge discharge pipe branching from the pipe described above, which passes to a sludge digestion tank especially provided for the purpose of experimenting with sludge taken from raw sewage.

The sludge digestion tank is of steel, made to be air and water tight. It is 5 ft. in diameter and 15 ft. in depth, with a pyramidal bottom, set vertically in the ground so that no part of it shall be above the waterline of the plain sedimentation tank from which it receives sludge. The effect of varying temperature or of chilling from ground-water is minimized by a double shell with an air space between shells. In operating the tank sludge is drawn from the settling tank under water pressure due to the head of water in the tank, by means of a branch in the sludge pipe, and passes through the sludge inspection box; after opening the valve into the digestion tank. The sludge remains in the digestion tank until it is digested. Before any sludge enters it is necessary to let out enough of the water from the top of the tank to furnish sufficient difference of head from the sludge to flow in. The digested sludge is discharged upon the Imhoff drying beds by means of the sludge pipe in the same manner as from an Imhoff tank.

Direct Aeration of Sewage With Compressed Air.—Direct aeration experiments may be carried out with raw sewage supplies from the quieting tank, or with the effluent of the Imhoff tanks. The principal direct aeration experiments provided for are to be carried out by means of an aerating siphon, and an aerating tank. Experiments will also be made with sprinkling filters supplied with compressed air through a grid placed within the mass of medium.

The aerating tank is a design developed from experiments made at the 26th Ward Sewage Works, in 1911, by Col. William M. Black and Prof. Earle B. Phelps, which gave much promise. Other experiments will be made than those mentioned.

Siphon Aeration.—The siphon aerator is an application of the siphon air compressor principle to the aeration of sewage, proposed by the late C. C. Beddoes, who obtained a patent covering the use of the siphon for sewage aeration. It may be operated with raw sewage or Imhoff effluent. The flow of sewage is led, by gravity, to the bell at the top of the siphon down-take pipe, into which the sewage falls, entraining or sucking the air in with it, and passing vertically downward through the pipe with considerable velocity, the entrained air becoming compressed. It is claimed that the sewage exposed to air under pressure absorbs a greater portion in consequence of the pressure, as the volume of air absorbed will be in proportion to the pressure. The apparatus consists of a 4-in. pipe extending vertically, downward, 130 ft. from the bell at the top; first through the center of a vertical tank 30 ft. deep and 4 ft. in internal diameter, and, second, from the bottom of the tank through an 8-in. pipe, leaving an annular space through which the sewage can flow upward from the bottom of the 4-in. pipe into the vertical tank, in which it is retained for a period of time in proportion to the quanity of flow and capacity of the tank. The effluent is discharged from the top of the vertical tank by means of a waste pipe and a measuring orifice box from which a portion of the flow is led, for observation, to the settling tank.

Tank Aerator.—The tank aerator is for the purpose of experimenting, both upon crude sewage and Imhoff tank effluent, with forced aeration, either by fill and draw or constant control. It is a tank 12 ft. in diameter and 25 ft. 8 in. in height. The sewage enters at the top of the tank by gravity at eight points from the quieting tank or the Imhoff tank outlets. The sewage may fill the tank so that these points of entrance are submerged, or the tank may be operated at lesser depths of content. The sewage or effluent of the tank is taken off at the bottom by means of four 3-in. openings into a 5-in. pipe.

A grid for supplying compressed air is placed at the bottom of the tank, upon 7½ ins. f broken stone, the same depth of broken stone being placed over it. The grid consists of 1½-in. pipes at right angles, forming a cross, connected in the center, the arms of the cross being connected with quartercircles of ¾-in. pipe forming concentric rings, of which there are five; each ring being perforated at 6-in. intervals with 1/16-in. holes. The air enters through the 1½-in pipes and is distributed thereby to the rings, and is discharged into the broken stone surrounding the grid, which tends to break up any upward streaming effect. The main outlet for the tank effluent is 1 ft. above this grid.

Through the central axis of the tank is placed a vertical 4-in. pipe which serves to center and support the deflector discs, of which there are nine, provided for the purpose of deflecting the downward flow of sewage and upward flow of air bubbles, so as to obtain even distribution of both air and sewage.

The deflectors are designed in the form of a wheel with a hub which is of iron; six radial arms are provided between which slats are placed, running from arm to arm, the slats being set in grooves in the arms and at an angle of about 45° with the horizon, the slats in each alternate deflector being set at angles alternating from and toward the center in order to give the sewage a sinuous motion in passing downward.

Sprinkling Filter Beds.—There are six percolating or sprinkling filter beds, all of them supplied with sewage by means of dosing tanks, and the sewage applied may be: (1) crude from the quieting tank; (2) aerated sewage effluent from the siphon aerator; (3) aerated sewage effluent from the aerating tank; (4) settled sewage from the Imhoff tanks; (5) any of the foregoing passed through a gravel roughing filter; (6) effluent from fine screens of the Reinsch-Wurl pattern (not yet installed).

The sewage flows by gravity to the dosing tanks, except the fine screen effluent which requires pumping.

Each dosing tank is provided with a 5-in. Miller siphon which discharges the dose into the inverted pyramidal feeding tank from the bottom of which it is carried by a pipe embedded in the medium to the sprinkling nozzle, by which it is sprayed over the bed. These tanks, and the inverted pyramidal feeding tanks into which they discharge, are constructed of yellow pine. The gravel roughing filter is also constructed of yellow pine and is so arranged that sewage, on its way to the dosing tanks, may be passed through it; the medium provided is gravel passing a ¾-in. ring and retained on a ½-in. ring. The gravel is 12 in. deep, supported in the middle third of a wooden tank by means of a wire screen of ¼-in. mesh. Such a filter has been found very effective by Mr. Watson at Birmingham for protecting the spraying nozzles. It is very effective as a remover of hairs and small particles that have escaped the settling tank. The roughing filter may be used or by-passed at will.

The elevation of the dosing tank water line at the instant of siphon discharge is 26.74 ft., which is 9.40 ft. above the surface of the filter beds. The elevation of the water line in the inverted pyramidal feeding tanks is controlled by the amount of sewage discharged from the dosing tanks, and its maximum elevation with the largest dose that can be discharged is 23.50 ft., which is 6 ft. 2 in. above the beds. This head can be varied by a movable bulkhead placed in the dosing tank which varies the quantity of discharge delivered by the siphon and increases the number of doses per hour.

The sprinkling filters, all of which are served by a single group of dosing tanks, are divided into two groups. The first consists of four filter beds of the ordinary type; the second of a tank 12 ft. in diameter and 16 ft. high, in which are placed two beds. A partition wall divides the tank into two equal parts and each side is filled with stone filtering medium to the same depth. The bottom of each side is underdrained with 6-in. half-pipe tile on a concrete bed, and is closed from external air by the tank walls. Each side is kept entirely separate from the other and drains independently to a secondary tank. Each side is provided with a grid for supplying compressed air, placed within the medium near the bottom of the beds, formed of ¾-in. iron pipe perforated every 6 in. with ⅛-in. holes, through which the compressed air is supplied.

In operation, the sewage is sprayed upon the surface of the beds by a single nozzle placed at the center of the two beds, over the dividing wall between them. Both beds may be operated as ordinary sprinkling or percolating filter beds, in which case air is carried into the bed from the surface only. Compressed air may be supplied to both sides at the same time, or one side may be operating as on ordinary sprinkling filter while the other is operated as a sprinkling filter with compressed air added in the bed, in order that the effluents may be compared and the effect of the added air be observed.

The filtering medium in both sides is best selected trap, broken to pass a ring 2½ ins. and be retained by a ring 1¼ ins. in diameter. The depth of the medium is 10 ft. which may be increased to 14 ft. by adding to the surface and raising the nozzle.

The ordinarv type of sprinkling filters consist of four beds, built in one group; the foundations are carried on piles and caps; the bottoms of the beds are 6 ft. above the level of the marshland over which they are built. High water at times washes underneath, covering the marsh to a depth of 6 or 8 ins. Each bed is square and has an effective area of .005 of an acre. A partition 4 ins. thick is carried from the floor to the surface of the medium between each adjacent bed.

The floor is formed of concrete with a slight slope to the outlet of the underdrains. Half-tile, 6 ins. in diameter, is laid with the convex sides up, on the concrete floor, to afford drainage, the effluent flowing to gutters or troughs placed around the bottom of the beds, outside of the wall piers; each bed having its individual gutter, discharging by means of iron pipe to its individual secondary settling tank.

The outer walls of the beds are formed by means of reinforced concrete piers carrying a slab coping of reinforced concrete at the top. Piers are cast with slots for receiving 3-in. yellow pine slabs or shutters, which are set at an angle of 45°, sloping inward, spaced 2 ft. apart, against which the medium rests, affording a maximum admission of air and preventing the escape of sewage. The medium is 10 ft. in depth over the top of the underdrains and is of very carefully selected broken trap rock, many runs through the screens having been necessary to obtain the result required. Following are the sizes of the mediums in the filter beds.

Bed No. 1, stone passing ring 1½ ins. in diameter, retained by ¾-in. ring.
Bed No. 2, stone passing ring 2 ins. in diameter, retained by 1-in. ring.
Bed No. 3, stone passing ring 2½ ins. in diameter, retained by 1¼-in. ring.
Bed No. 5, stone passing ring 2½ ins. in diameter, retained by 1¼-in. ring.
Bed No. 6, stone passing ring 2½ ins. in diameter, retained by 1¼-in. ring.
Bed No. [illegible], stone passing ring [illegible] ins. in diameter, retained by 1¾-in. ring.

In order that the effect of the depth of filter medium under similar conditions of operation may be obtained, test trays with outlet pines are placed in the filter beds at different depths. The trays are V-shaped, 10 ins. wide, and extend from the wall to the center of each bed. Each is provided with a drain pipe with a stopper, used when samples are being obtained. The trays are so placed that samples may be taken at depths from the surface of the bed of 6 ft., 7 ft. 3 ins. and 8 ft. 6 ins. Samples taken from the bottom of the bed give the result of 10 ft. depth. Thus, samples from four different depths of medium are available for observation from each bed.

In order to prevent the effects of wind on the sewage distribution, a shield is provided, consisting of a board fence at the surface carried between the beds and around them.

The secondary settling tanks to which these filters discharge are placed in a group. Each tank is an inverted truncated pyramid, 10 ft. deep from the water line. The flow enters through a 2-in. pipe down to a point 2 ft. above the bottom. Settlings are removed by means of a 6-in. sludge pipe operated by the hydraulic head of the tank. The tank effluent is taken off by troughs passing entirely around the top of each tank through V-shaped notches.

Sludge Drying Beds.—For the purpose of testing the character of sludges from the different experimental units, and making observations on rates of drying, sludge drying beds of the Imhoff type are provided. There are 10 beds, each 5 ft. wide by 12 ft. long. The elevation of the surface of each bed is 2.67 ft. above datum. These beds are constructed upon a timber platform carried on caps supported on piles. Each bed is supplied with a 6-in. half-tile pipe underdrain, placed along the center from near the inlet to the lower end of the bed. The medium consists of 8 ins. of steam ashes surfaced with 1 in. of coarse sand. Timber partitions separate the beds, each bed being in effect a tank into which the sludge is discharged bv means of sluices controlled by gates. Sludges can reach these beds by gravity from all of the sludge producing units of the plant. For the experiments only a portion of the sludge formed in the various units will be applied, the surplus being discharged into the main sewer outlet.

Disinfection Experiments.—Disinfection investigations will be undertaken when the various units of the plant have been given sufficient time to develop the best possible effluents, and the Reinsch-Wurl screens, and vacuum filters are in operation. Tanks are provided for the purpose.

The experimental plant was, for the most part, completed in December of last year and put into regular service January 1st of this year. Sufficient time has not yet elapsed to make any results available for publication. The service of the sprinkling filters, during the zero weather of last winter, was accompanied with the formation of considerable ice.

Sewage Screens.—The portions of the plant not yet completed include the major portion of the screening apparatus of which two Reinsch-Wurl screens are intended to constitute an important part. A description of the screens designed, and now awaiting letting of the contract for installation is, therefore, added as showing this important portion of the plant. It is intended that the screens shall remain a permanent part of the proposed sewage disposal plant to be installed at this location. Each screen will have a capacity of 6,000,000 gals. of screened sewage per day, giving a total of 12,000,000 gals.

This location will require a much greater ultimate capacity, and additional screens of the same type may be installed at the plant as required in the future. The design of the plant provides for a by-pass between the screens so that they may be operated individually or in series so that double screening may be tried out, the second screen in the series being very fine.

The screens will be 14 ft. in diameter and operated by steam engine drive, a 15 HP. engine is provided for the initial installation. The estimated power required, as a maximum, to drive each screen and the screen cleaning devices is 4 HP., leaving 7 HP. available for driving a belt convevor, etc., for removing the screenings from the building to the carts.

The apertures in the screens, or openings through which the sewage passes in being screened, are to be made of such size as may be found to give the best results with the average flow of sewage at this location. Screening practice and experience elsewhere shows that this should be determined experimentally, and with this end in view the specifications provide, in relation to this subject, that four complete sets of screen surface plates shall be supplied; each set being a complete surface outfit for a screen; each set to be cut with apertures of different dimensions, as follows—the first set to be cut with apertures 5/64-in. wide; the second with apertures 1/16-in wide; the third with apertures 3/64-in. wide; and the fourth with apertures 1/64-in. wide; in each set the apertures will be 2 in. long and will be staggered in the bronze surface plate, which is ⅛-in. thick, each aperture to have a counter-sunk cross-section with the narrow part of the opening on the face of the screen.

These sets of screen plates are to be mounted on the screen frame successively and tried out, and the aperture dimension decided to be the most suitable for screening the sewage at this plant will be selected; the screen plates not selected remaining the property of the screen contractor who may recut them, if the apertures are too small, or use them elsewhere.

The size and number of apertures in the screen surface must be such that 6,000,000 gals. of sewage will pass through the screen in 24 hours, the difference of head of sewage on entering and leaving sides of screen plate being not more than 12 ins. The screen must remove from the sewage practically all particles of suspended matter with a diameter 50 per cent greater than the cross section of the aperture; the removal of suspended matter is intended to be approximately equal to that effected in a plain sedimentation tank with a retention of one hour, but what shall be considered a satisfactory removal of suspended matter will be determined by tests to be made with the plates having graded apertures under the engineer's direction.

In designing sewage screens one has but little information to go on. Data are very meager and unsatisfactory. Claims are made that removal may be effected of from 25 per cent all the way to 80 per cent of the matters in suspension which would settle in an ordinary sedimentation tank in four hours. Sewages differ very widely in the character of suspended solids, and naturally the kind of screens which will serve best with a given sewage, and the required fineness of the screening surface, varies widely. In Dresden, Germany, where there are four large screens of the Reinsch-Wurl design, the breadth of the apertures is 2 mm., which is found at that place to give satisfactory removal of suspended matter, and this is the only treatment found necessary for the sewage of that city, which, after screening, is discharged through multiple outlets directly into the Elbe River, affording at all times a satisfactory disposal of sewage. Dresden has a population of between 300,000 and 400,000, and at its lowest stages the river is a small stream.

Slits or apertures of lesser width than 2 mm. are said, in German practice, to reduce the capacity of the screens without much added advantage. This is especially the case where, from the nature of the suspended matter, a mat tends to form over the screen surface and produce a straining effect. In Dresden the difference of head through the screens varies from 2 to 6 cm., under ordinary conditions, but at times is much greater than the larger figure—possibly four times as much.

The four screens at Dresden, each of which is 8 meters in diameter, with an ordinary submergence of 2.4 meters, together handle 18,000 second-liters.

Douglas Fir Principally Used in Hawaii.—The lumber bought on Puget Sound for Hawaii is almost all Douglas fir, but is known in the Territory as Oregon pine and is sometimes called Northwestern pine. Nearly all of it is No. 1 merchantable grade and is graded and sold under the certificate of the Pacific Lumber Inspection Bureau. Small quantities of rough clears are also included in some of the orders. In October, 1914, the base price on rough Douglas fir varied from \$24.50 to \$28 per thousand feet, according to the size of the order, credit demanded, and other factors. In other years the base price has been as high as \$32.50.

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Traffic Statistics and Their Import in the Financing of Road Improvements and the Selection of Pavements.

In a paper before the Fourth American Road Congress, Henry G. Shirley, state highway engineer of Maryland, makes the following statement:

"Before selecting the type of pavement to be used, a close and accurate census of the different kinds of traffic should be taken, a very thorough study made of the surrounding section, and an estimate made as to the possible increase of the different kinds of traffic, or the decrease of one kind and the large increase of another. It is the opinion of the writer that *in no other line of engineering should there be a larger factor of safety used than in estimating the amount, intensity, and kind of motor and self-propelled traffic that will pass over our improved roads in the near future.* The great change in the character of traffic developed in the past five years, is but a small index to what can be expected in the next five years to come."

"*The types of pavements used on heavy traffic roads should be selected for their fitness to stand the kind and intensity of the traffic that will travel them.* Roads in the outlying districts, where horse-drawn traffic comprises the larger percentage, should be constructed of macadam with a light surface treatment. Concrete will also be found serviceable and desirable. Where motor traffic is in the majority, bituminous macadam or concrete will give good results. Near the centers of population, where the traffic is mixed and heavy, concrete, bituminous concrete, asphalt or vitrified brick will prove the most economical. Where the heavy traffic is concentrated, brick, asphalt or stone block are most suitable."

The two statements we have italicized are worthy of comment; the first, with regard to interpretation of the term "factor of safety," and the second, in connection with some recent tendencies in the selection of pavements.

It is well to remember when discussing the selection of pavements that the pavement is essentially a wearing surface and bears but little relation to the location, culverting and other important details of the road, although affected to some extent by maximum grades. Furthermore, the former great importance of adapting the pavement to a greater or less degree to soil conditions and the availability of road materials has been overshadowed by traffic requirements. This is rightly so. The primary function of a pavement is to provide a means of quick and economical communication and transportation.

There has been much learned discussion of late concerning the folly of building roads that will wear out before the expiration of the life of bonds issued to pay for their construction. The shaking of heads and wagging of beards has become contagious. Financial geniuses have developed in the period of a single night. According to them the debit side of the ledgers of many states presents a fearful spectacle of accumulating interest charges. But what is shown on the credit side of the account? Does not the rapid and profitable expansion of any business—or section of the country—require the use of large amounts of capital which ordinarily can be obtained only by borrowing and paying interest? What percentage of the total cost of the road is invested in parts that will wear out? What saving of money is derived from the realignment of roads while property values are low? What is the value of an improvement that permits a man to do in one day work which formerly required two days to accomplish? Finally, what have been the results in communities where extensive improvements have been made on borrowed capital? In short, what "factor of safety" have the citizens of various communities applied to the traffic requirements of their section? Are the benefits all to disappear in a few years, as some would lead us to believe, or has a "factor of safety" been applied to provide for the future welfare of the community? These ideas are perhaps somewhat foreign to the meaning intended by Mr. Shirley in the paper quoted but the application is no less striking than in the other more technical phase of the use of the term.

A rigid technical interpretation of the "factor of safety" as applied to traffic requirements is illustrated in the construction of a type of pavement that careful study indicates will be adapted to future traffic. Present tendencies indicate this will be influenced by the type of vehicles used and the size of loads hauled, the attraction the improvement of the thoroughfare will exert upon traffic using other roads, and the general gross increase in traffic due to improved facilities. When these matters are considered the importance of obtaining accurate data upon which to base estimates—a point frequently emphasized by this journal—becomes evident to a marked degree. The very fact that the use of a factor of safety is advocated indicates a lack of data. The great folly lies, not in investing large sums of money in roads, but in building expensive roads through sections, or in regions where future traffic will never justify such construction. We have some most interesting examples of the application of this principle in railroad building and, for the lack of something more directly applicable, we earnestly recommend to highway engineers engaged in the laying out and improvement of roads the conscientious study of Wellington.

There is much written concerning the superiority of this road material, or that method of construction, or the excellence of another type of pavement. Discussion is fostered and the volume of printed matter augmented by business organizations crying their wares. Much of this discusion is of great value and important investigations have been made and the results published by commercial concerns. The experimental work on hauling and traction carried out by a firm of wagon manufacturers is especially notable. But wisdom dictates the use of the proverbial "grain of salt" as adding much to the palatable qualities of statements with regard to the superiority of various pavements. The engineer is his own judge as to the fitness of a pavement to fulfill the requirements of the location to which he may adapt it. Fads in pavements are as transitory as fads in fashions. The tendency to restrict road pavements to certain types cannot be too heartily condemned. Experience in this country and abroad indicates that there are many efficient types of pavements and the adoptions and advocacy by an independent engineer of any one type to the exclusion of all others can but only indicate ignorance, sloth, or the use of undue influence. Such opinions are excusable in a layman but are inexcusable in an engineer.

Statistics of traffic, the number and rate of increase of vehicles in a community, types and amounts of crops raised, industrial conditions and the many other conditions that may affect traffic are perhaps dry and uninteresting to some engineers. How much easier it is to say offhand, "We will build such and such type of pavement." This was the method of early day railroad builders and we are still paying interest on the errors of their "practical judgment," and while many mistakes have been corrected many more are irremediable. The sangfroid with which automobilists are today laying out through roads holds one agape. We sometimes wonder how much the fact that such a one lives in one town, or that another city has an unusually active chamber of commerce, affects the location of these roads. Work of this nature is a function of the national government in relation to interstate roads and of the state government as affecting state roads, and such selection of roads should be made only after most careful studies if it is expected to expend any large sums of money in their improvement. If this selection is merely for the convenience of travelers unacquainted with the roads then their platting is of utility. The kind and intensity of traffic that travels and will travel the road is the main criterion for the type of pavement to employ, but the selection of the road to improve depends upon many other factors.

A Civil Engineer Member of the Maine Public Utilities Commission.

It is probable that Mr. Charles W. Mullen's nomination as member of the Maine Public Utilities Commission will be confirmed by the legislature. The legislature refused to confirm the nomination of Mr. S. W. Gould, the opposition to Mr. Gould being based on the belief that the commission should not consist entirely of lawyers. Accordingly the governor has nominated Mr. Mullen, a civil engineer, who has been mayor of Bangor. Mr. Mullen is an alumnus of the University of Maine, class of '83.

In this connection it is pertinent to call the attention of California engineers to the opportunity to secure the appointment of an engineer to take the place of Mr. John M. Eshleman, who has been elected lieutenant governor.

Mr. D. W. Stanrod has resigned from the Idaho Public Utilities Commission, and Mr. John H. Roemer, chairman of the Wisconsin Railroad Commission, has announced that he will resign Jan. 1. In a recent issue we called attention to the expiration of the terms of two public service commissioners on the two commissions in New York state.

There will soon be vacancies on at least five public service commissions. Engineers in the states where these vacancies exist, or will soon exist, should lose no time in concertedly urging the appointment of experienced engineers to fill these places.

It was contended by the former governor of Maine that a utility commission is really a court, and that it should therefore be composed of lawyers. He advanced the timeworn argument that engineering services could be hired by the commission, and that consequently none of the commissioners need be an engineer. But with equal logic may it be urged that a commission may hire attorneys, and with much greater force may it be contended that a commission is not a court in the ordinary sense of that term. A utility commission should primarily be a body of appraisal and rate-making experts, qualified by training and experience to analyze data and evidence relating to the cost of constructing and operating public utilities. When has it been shown that a legal training is superior to an engineering training as a grounding in a knowledge of the significance of cost data?

We grant that a lawyer may be a better judge of law than an engineer. But must it not be conceded that an engineer is a better judge of the costs of constructing and operating all those great classes of engineering plants commonly called public utilities? Merely to name them is to carry conviction that they are engineering properties—railways, water works, gas plants, electric power and light plants, telephones, etc.

There should be a lawyer as member of every utility commission, not because a commission is a court, but because it is a semi-judicial body that needs the lawyer's point of view when considering rate questions behind closed doors. Precisely the same sort of reasoning commends the appointment of an engineer on every utility commission.

We shall not repeat the numerous reasons why an engineer is well qualified by training and experience to weigh most of the evidence submitted in a rate case. It is no answer to these reasons to say that an "engineer is merely a technical expert," for a competent engineer is not merely a calculating machine. He is a keen judge of economics, a judge of men, and motives, a judge of everything save, perhaps, the legal technicalities, that go to form the data upon which an equitable rate is based. But until engineers individually and collectively undertake to make these facts clear to the public, there is small hope that engineers will be appointed members of public utility commissions.

Bridge Renewal Under Traffic.

The replacement of a railroad bridge under traffic always presents special problems which usually tax the ingenuity of the engineers and the resources of the operating department. At the start it must be decided whether it will be better to repair and strengthen the old bridge or to replace it entirely by a new structure. With a view to economy it may be decided merely to strengthen the old bridge and to renew worn-out parts, only to find later that the difficulties had been underestimated and that economy would have justified the construction of an entirely new bridge. In either case the problem of maintaining traffic is a serious one, and it is often the deciding factor in determining the type of bridge to be used. In the "Bridges" section of this issue we are publishing Part I of an article on the design, construction and cost of the renewal under traffic of a 970-ft. railroad bridge at Lacolle Junction, Quebec. Part II, which gives the detailed costs of this work, will appear in our Dec. 23, 1914, issue. The construction methods used in the renewal of this bridge are of particular interest, and the detailed costs are exceptionally complete.

The old bridge consisted of a long pile trestle, near the center of which was a swing span. The pivot pier of the movable span consisted of a timber crib filled with rubble stone surrounding a cluster of piles, the latter being capped with a timber grillage and a concrete slab. The new structure comprises a series of plate girder spans and a 250-ft. swing span. The caisson and pier construction for the new bridge involved some unusual features. As the old first pier was too light and too unstable to support the new swing span it was decided to reinforce this pier by enclosing it with a double-wall timber caisson, and to fill in the space between the old construction and the caisson with concrete. A 12-in. space was left between the two sections of the double wall, and the caisson was sunk by filling this space with concrete and by providing additional weights, where necessary. During the reconstruction of the old pivot pier the swing span was supported by heavy I-beams, the ends of which rested on the new caisson. In constructing the piers for the twelve plate girder spans, those in deep water were concreted inside of double-wall caissons, while those in shallow water were built inside of single-wall caissons. The total and unit costs for each part of this work are given in detail, and these costs should prove of considerable value in estimating the cost of similar work. The detailed costs were used as a basis of payment to the substructure contractor, who did the work on a cost-plus-percentage basis. The superstructure was erected and track facilities provided by the operating department of the railroad company. An interesting comparison is given of the relation between the cost and height of the twelve concrete piers which support the 60-ft. girder spans.

ROADS AND STREETS

Methods and Cost of Removing An Asphaltic Macadam Road Surface, Reworking the Old Material and Relaying It as Asphaltic Concrete.

Contributed by G. C. Dillman, Civil Engineer, Michigan State Highway Department, Lansing, Mich.

A poured process asphaltic macadam surface 16 ft. wide was laid on the extension of Woodward Ave., Detroit, in 1912. This road is a trunk line between Detroit and Pontiac and is subjected to a heavy traffic the approximate type and amount of which is indicated by the traffic record for two days shown in Table I. Holes began to appear in the surface soon after its completion. The condition of a portion of the surface in July, 1914, is shown in Fig. 2.

The original improvement was made by Royal Oak township of Wayne Co., Mich., which did not make the repairs required by the state reward road law and in July, 1914, the surface was reconstructed under the direction of F. H. Rogers, state highway commissioner. The new work consisted of tearing up the old asphaltic surface, heating it with necessary new material added and relaying on the old slag macadam base.

REMOVING OLD SURFACE.

The asphaltic surface to be removed was first swept clean. Twelve men picking, and lifted and picked into small chunks and shoveled into wheelbarrows, the material covering about 250 lin. ft. of roadway surface each day.

In scalping off the old bituminous top, two places were broken up at the same time to hasten the work. At first the men picking stood on the surface already broken up, pulling the chunks as picked up, toward them. This did not work out satisfactorily so another method was tried, in which the men stood on the old surface, driving the pick beneath the broken edge. The surface coat was then readily lifted and broken into pieces that could be handled. These large pieces were then thrown back where three men picked them into still smaller ones to save time in the mixers. The old material broke up readily in the morning but towards noon softened and more time was needed in breaking. Although the depth of penetration was far from being uniform, the bituminous top separated readily from the slag foundation.

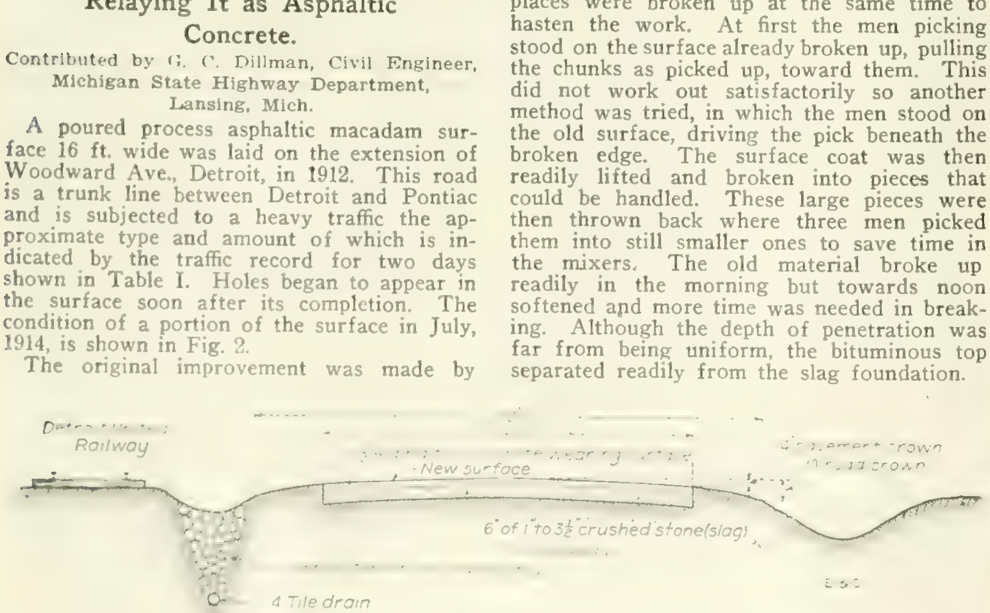


Fig. 1. Typical Cross Section of Roadway Improved. Asphaltic Macadam Wearing Surface Replaced by Asphaltic Concrete.

Long stretches of the old surface were laid when the foundation was wet, and the bad effects of water present could be seen. The asphalt had not penetrated to a sufficient depth, and there was practically no bond in the pavement in such places. In other places a 6-in. penetration was observed. When the old surface was broken up the stone was often covered with moisture and the asphalt could be peeled from the stone. This presence of moisture may be partially accounted for in that the general drainage of the road was poor, yet it is probable that the foundation had not been in a dry condition since the pavement was laid. When the road was resurfaced, the foundation was allowed to dry out and the general drainage was also taken care of.

TABLE I.—TRAFFIC RECORD ON WOODWARD AVE., DETROIT.

Kind of vehicle.	Day and date. Saturday, 8/15/14.	Sunday, 8/16/14.
Single horse	34	30
Double team, light	15	.....
Double team, loaded	35	15
Motor touring car	994	1,912
Motor runabout	142	315
Motor trucks	62	39
Cotor cycles	73	179
Total between 7 a. m. and 9 p. m.	1,355	2,490

Fig. 2. View Showing the Condition of Old Surface Prior to Resurfacing.

MIXING AND PLACING.

The equipment for mixing and placing consisted of 2 ½-cu. yd. hot mixers, 1 500-gal. heating kettle, 1 5-ton roller, carts, small tools, etc.

The old surface picked into small chunks was delivered to the mixers in wheelbarrows. An average batch consisted of sufficient material to lay 3½ sq. yds. of 2½ in. surface, and contained 728 lbs. of old top, 252 lbs. of new stone and 15.92 lbs. of new asphalt, the variations in different batches being shown in Table II.

The old top was changed into the mixer with about 25 per cent. of ½ to ¾ in stone added and about 0.45 gals. per square yard, or about 1.6 gals. per batch, of new asphalt. Mixing was usually continued 8 mins. at the end of which time the temperature of the material would average about 240° F. Two high-wheeled carts were used to convey each batch to the point where it was to be laid.

Fig. 3. Tearing Up Old Surface and Wheeling to the Mixer.

TABLE II.—DAILY VARIATION IN QUANTITIES AND PERCENTAGES OF MATERIALS USED.

Date.	Batches.	Lin. ft. laid.	Sq. yds. laid.	Asphalt in mix. Lbs.	Asphalt in mix. Per cent.	Old bituminous top in mix. Lbs.	Old bituminous top in mix. Per cent.	New stone in mix. Lbs.	New stone in mix. Per cent.
July 15	76	172	305.8	1,520	2.2	53,200	74.4	16,720	23.4
16	67	146	259.6	402	0.5	68,005	82.5	14,070	17.0
17	99	190	337.8	1,386	1.6	68.310	74.6	21.780	23.8
18	70	132	234.7	980	1.6	46,300	74.6	15,400	23.8
20	106	242	430.3	1,484	1.6	73,140	74.6	23,320	23.8
21	103	206	366.3	1,442	1.5	70,070	74.7	22,660	23.8
22	106	215	382.3	1,378	1.5	73,140	74.7	23,320	23.8
*23	51	100	177.8	510	1.1	35,190	75.0	11,220	23.9
24	103	206	366.3	927	0.9	71,070	74.3	23,690	24.8
25	115	211	375.2	1,150	1.1	79,350	75.0	25,300	23.9
27	137	227	403.6	1,781	1.5	94,330	74.7	30,140	23.8
28	127	208	369.8	1,905	1.6	87,430	74.6	27,940	23.8
29	120	219	389.4	1,560	1.4	82,800	70.9	59,400	27.7
30	125	254	451.6	1,875	1.5	93,750	72.4	33,750	26.1
31	105	228	405.4	1,575	1.5	78,750	72.4	28,350	26.1
Aug. 1	97	228	405.4	1,455	1.5	72,750	72.4	26,190	26.1
3	103	204	362.7	1,545	1.5	77,250	72.4	27,810	26.1
4	129	287	510.3	1,935	2.4	96,750	71.8	34,830	25.8
5	125	260	462.3	3,125	2.4	93,750	71.8	33,750	25.8
6	115	247	439.2	2.300	2.0	86,250	72.1	31,050	25.9
7	136	236	419.6	2,720	2.0	102,000	72.1	36,720	25.9
8	137	245	435.6	2,740	2.0	102,750	75.8	30,140	22.2
10	96	160	284.5	1,920	2.0	72,000	75.8	21,120	22.2
11	97	178	316.5	2,425	2.5	72,750	73.9	23,280	23.6
*12	41	87	154.7	1,025	2.5	30,750	73.9	98,400	23.6
Total	2,586	5,088	9,046.5	41,065	...	1,882,835	....	651,790	....
Average per batch	....	1.97	3.50	15.92	1.6	728	73.1	252	25.3

*One-half day.

The base, after removing the surface material, was found to be very rough due to the original poor grading, and to the varying depths of penetration of the asphalt. All depressions in the bottom layer were filled with ¾ in. stone, after which it was thoroughly compacted by rolling.

After the bituminous macadam had been laid and rolled to a thickness of 2½ ins., a squeegee coat of asphalt was applied at the rate of about 1 gal. per square yard. A ¼ in. layer of stone chips free from dust, was then put on and rolled in. It was believed that ½ gal. of asphalt per square yard would be sufficient for the squeegee, but due to the large size stone in the old surface material, it was necessary to double this quantity.

Each day a sample of the surfacing laid was analyzed by the chemist and from the results of his analysis, together with the appearance of the old material, the mix was determined. Almost constant attention had to be given the mix on account of the varying composition of the materials. Table III (see p. 534) gives the chemist's analysis of samples. An attempt was made to keep the per cent of bitumen between 5 and 6, but as will be noted, it was low at times due to the large stones in the sample tested.

The road was closed to traffic July 13, 1914 and opened to traffic August 12, 1914, about an hour after completion. Thirty-eight men were required 24 working days to tear up, remix and relay 9,055 sq. yds. In a 10-hour day a maximum of 510 sq. yds. was laid.

COST AND PERSONNEL.

Table IV (see p. 534) gives an itemized statement of the costs per square yard of pavement laid. These figures were compiled from the daily expenses as gathered by the state inspector and are very close to the exact cost. The contract price for laying the bituminous pavement was 73 cts. per square yard, $500 being allowed for the extra asphalt used for squeegee.

The prevailing rate of wages was $2.25 a 10-hr. day for ordinary labor; $2.50 and $3.00 being paid enginemen, firemen and rollermen, and $4.00 for raker. The teams cost $5.00 per day.

The estimate for foreman is based on 30 days at $10.00. The contractors had two foremen on the work at all times, both being members of the contracting firm.

The equipment cost cannot be stated in exact figures, since it was owned by the contractor, but the figures given represent an average rental price.

E. W. Seamans of Grand Rapids, Michigan, was the contractor, working under the general direction of H. B. Pullar Company, chemical engineers, Detroit. G. C. Dillman was engineer for the state highway department.

Fig. 4. Placing the Asphaltic Concrete. Note the Type of Cart Used.

Fig. 5. Placing, Rolling, Applying Flush Coat and Applying Coat of Stone Chips.

Some Practical Notes on Brick Road Construction.

Excellent specifications for brick road construction are readily obtainable and from them instructions as to methods to pursue may be obtained. Many faults, however, that exist in brick paving are the results of errors of commission rather than of omission and in a paper before the Northwestern Road Congress James M. McCleary, county engineer of Cuyahoga County, Ohio, points out what not to do.

Grading and Drainage.—Do not place too much trust in a fill which was partially made a generation or two ago. The older portion may be the more treacherous. Perhaps trees and brush were used in making the original fill. If they have decayed, the fill is in a honeycombed condition and likely to give way. The best method of locating void is by puddling.

Enclose the sub-grade with temporary

TABLE III.—ANALYSES OF SAMPLES FOR EACH DAY'S RUN.

Date.	Bitumen. per cent.	Stone in per cent. Size of mesh passed. 200	80	40	10	4	2	3/4	1	1½	2
July 15	5.3	4.3	0.7	1.7	2.1	9.1	24.1	24.8	27.9	...	...
16	4.0	3.2	1.5	1.3	2.8	6.6	16.9	23.9	39.9	...	...
17	4.9	3.8	1.3	1.2	3.3	10.2	17.5	24.6	33.2	...	...
18	6.2	5.8	1.0	1.2	5.6	13.6	22.3	24.1	14.4	5.8	...
20	5.4	4.2	1.1	1.3	3.1	6.4	7.3	21.4	18.5	31.3	...
21	4.6	4.0	0.7	0.9	3.0	6.5	13.8	28.6	37.9	...	...
22	6.6	7.6	1.5	1.8	5.3	12.4	18.8	25.0	14.8	6.2	...
23	4.7	5.5	0.8	1.0	3.6	7.1	14.2	28.4	13.2	21.5	...
24	3.1	2.9	0.5	0.6	2.2	4.9	10.3	15.7	15.6	44.2	...
25	5.1	4.2	0.9	0.8	3.2	7.7	21.8	32.9	12.6	10.8	..
27	5.3	6.3	1.2	1.3	3.8	8.0	30.0	27.2	16.9	...	...
28	3.6	3.6	0.7	0.8	1.8	3.7	12.9	14.2	7.9	50.8	...
29	2.4	2.5	0.6	0.6	1.2	2.2	5.1	14.3	25.1	10.8	35.2
30	3.8	4.2	1.3	0.6	2.6	4.6	20.8	21.1	20.5	5.5	15.0
31	5.3	6.0	1.7	2.1	3.8	5.2	13.7	11.7	6.1	6.1	38.3
Aug 1	4.1	4.4	0.9	1.0	2.3	3.7	11.9	25.4	11.0	35.3	...
3	3.1	3.0	0.6	0.6	1.3	3.1	11.4	29.0	21.3	26.6	...
4	3.8	3.8	0.8	0.8	2.0	2.7	8.7	26.7	18.0	32.7	..
5	3.6	4.1	0.7	0.7	2.3	5.2	20.2	25.2	11.4	26.6	...
6	5.8	3.7	0.6	0.7	2.7	3.6	7.0	12.8	33.0	30.1	...
7	5.5	3.8	0.9	0.8	2.8	4.7	9.8	23.9	15.1	32.7	...
8	5.5	3.4	0.7	0.8	2.5	4.6	13.8	23.8	10.0	34.8	...
10	3.9	3.5	0.6	0.6	1.6	3.9	16.6	27.9	8.8	32.6	...
11	4.6	4.4	1.0	1.0	2.6	5.6	6.7	19.1	19.3	35.7	..
12	8.0	6.2	1.7	1.8	5.6	19 2	16.2	15.0	26.3	...	...
Average	4.7	4.3	1.0	1.0	2.9	6.6	14.9	22.7	19.1	25.2	29.5

earth dikes 2 ft. higher than the subgrade and divide it into compartments by similar cross dikes. Fill the compartments with water, one at a time, and weakness, if any, will shortly appear. The usual method of relying on a roller for compacting a fill is not nearly as efficacious. In my experience, not more than ten per cent of the fills could be properly compacted with a roller alone.

Do not fall into the error of thinking that the province of a roller is to produce a smooth even surface on the fill. Rolling, like puddling, ought to hold in developing hidden weaknesses where more or different material is needed.

Do not undervalue the necessity of drainage. My rule has been to use it as a precaution in dry places and as a necessity in wet places. Tile drainage is much better than ditch drainage, but do not place your longitudinal drain beneath the pavement where it is less efficacious in removing the water which the pavement sheds and where, also, it offers a source of back seepage in wet times which may keep the pavement moist and be responsible for shifting in the sand filler. The place for the longitudinal drain is below the gutter. Cross drains should be proportioned in frequency to the nature of the soil and the character of the natural drainage. In some muddy places it may be necessary to lay them every ten or fifteen feet. In other places they will be unnecessary for a considerable interval. The essentials of my practice in under drainage have been: 1st, open tile; 2nd, position below the frost line; 3rd, the use of gravel or cinders in filling the trench.

One advantage of under drainage as compared with a longitudinal ditch, is the lessening of danger to traffic and the more level shoulder of earth that can be graded above an underground drain. Such a shoulder can be kept free from tall grass and weeds by means of a mowing machine.

Concrete Bases.—I would issue three warnings with respect to concrete bases; do not use concrete that is not homogeneous; do not tolerate the existence of voids; do not be satisfied with a finished surface that is not uniformly smooth. In addition to these points, one should observe all the other cautions that apply generally to concrete mixing.

The value of the first and second warnings is apparent when you consider that the sole object of a foundation is to strengthen the natural bearing surface and transmit the burden widely and uniformly. The third caution is to prevent such projections of depressions in the concrete as shall result in a different depth of and cushion at different points.

The importance of this feature will be apparent after the rolling of the brick surface is in progress. An undulating surface of brick means a sounding board effect when the pavement is brought into use and a possible breaking of the bond. Mere spreading of the sand is never sufficient.

It should be rolled and re-shaped repeatedly until both the surface and the density are uniform. Too often the road builder contents himself with one rolling after which he fills the depressions with loose sand and finishes the surface with a template. Each refilling should be followed by rolling, a hand roller of 350 pounds weight being most satisfactory. A soft, uncompacted sand cushion will work up between the brick, when the latter are rolled.

Culling Brick.—In the culling of brick, a good judgment is the exception. Many seeming defects, as viewed by the casual inspector, are not defects. The cull pile may well be examined for later decision on some of the brick that were hastily eliminated. Softness is the chief defect to avoid. Kiln marks frequently indicate unusually good brick, because they are due to fusibility and pressure from

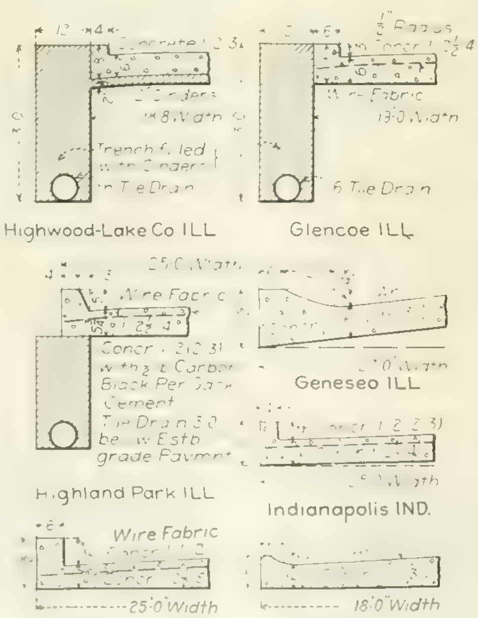


Fig. 1. Types of Concrete Curbs Built Integral with Concrete Pavements.

the weight of overlying brick in the kilns and fusibility means vitrification. Be sure that your brick are used their best side up. Delivery to the setter in such a position is recommended. Be sure that the lugs are all in the same direction. The purpose of the lug is to provide a uniform interstice and permit the grout to descend to the bottom of the brick. Laminations are not to be avoided. If you visit a brick plant you will see that the very process of making bricks entails the existence of laminations.

Rolling and Grouting.—Do not begin your rolling in the center of the pavement. Roll adjacent to the curb first, approaching the center gradually. When the center has been reached, start at the opposite curb and repeat the process.

Good grouting, like charity, covereth a multitude of sins. No badly grouted pavement was ever a good pavement, but well grouted pavements have sometimes passed muster for considerable periods in spite of gross faults in other details of construction. Three successive applications of a one to one mixture of Portland cement and clean sand has been my rule. The utmost care in selecting materials, in applying the grout and sweeping it into the very bottom of the cracks will be repaid in results. Anything less than the most exacting care in the application of grout is just like throwing labor and material away.

Be careful not to injure your curbing during the later stages of construction. Careless hauling of heavy loads of machinery over an unprotected edge will cause breaks which expose the brick to abnormal wear. This caution has frequently been violated in my observation.

TABLE IV.—UNIT COSTS OF WORK.

LABOR:		Cost per sq. yd.
Preparing old bituminous top—		
2 enginemen		$0.016
2 firemen		0.014
2 platform men		0.013
6 wheelers		0.029
12 pickers		0.064
Total	$0.136	
Placing new material—		
4 cartmen		$0.020
1 kettleman		0.008
1 raker		0.013
1 rollerman		0.008
Total	$0.049	
Preparing grade—		
3 grademen		$0.015
Putting on squeegee and chips—		
3 squeegeemen		$0.021
Other labor—		
2 helpers		$0.012
1 waterboy		0.003
1 nightwatchman		0.007
Incidental labor		0.016
1 team (steady)		0.017
Total	$0.055	
Cost of labor per sq. yd.	$0.276	
MATERIALS:		
Asphalt—		
20.53 tons in mix		$0.051
49.80 tons in squeegee		0.123
Cost of asphalt per sq. yd., $0.174.		
Stone—		
325.9 tons in mix		$0.068
90.5 tons in chips		0.019
12.7 tons in grade		0.003
Cost of stone per sq. yd., $0.090.		
Incidentals—		
Fuel, insurance, freight on equipment, etc.		$0.073
Equipment (estimated)—		
Roller and kettle		$0.023
Two mixers		0.100
Small tools		0.006
Total	$0.129	
Foreman (estimated)		$0.033
Grand total		$0.74

SUMMARY.

Item.	Average for 2,586 batches.	Totals.
Length surfaced, lin. ft.	1.97	2,586
Area surfaced, sq. yds	3.50	9,046.5
New asphalt added, lbs.	15.92	41,065
New asphalt added, per cent.	1.6	....
Old bitu. top, lbs.	128	1,882,835
Old bitu. top, per cent.	73.1	....
New stone in mix, lbs.	252	651,790
New stone in mix, per cent.	24.3	....

Some Types of Concrete Curbs Built Integral With Concrete Pavements.

(Staff Article.)

It has become a well accepted practice in the construction of concrete roads and street pavements to construct the curb as an integral part of the pavement, in cases where a curb is deemed necessary. This practice has much to commend it since the danger of curbs getting out of line is eliminated and the cost of construction is materially reduced.

The methods employed in constructing these curbs vary somewhat with the type used. In those types having a vertical inside face a form board is necessary to retain the concrete. Other types that provide a smooth flowing curved joint with the earth beyond the limits of the pavement require no forms other than the outside form. As a rule concrete in the curbs is the same as that used in the body of the pavement and no special mortar facing is provided. The surface, however, is usually floated or rubbed to improve the appearance of the exposed face.

Details used in various cities are shown in Fig. 1. It will be noted that care is used in several cases to provide adequate sub-drainage and that no standard shape or depth is followed. We are indebted to the Universal Portland Cement Co. for the illustration shown.

Hauling Gravel with Motor Truck and Trailers: A Service Test in Concrete Road Construction.

A service test to determine the utility of motor trucks and trailers in hauling gravel to be used in the construction of a concrete road was undertaken at Kenosha, Wis., on Oct. 5. Complete time studies were made and the results obtained are given here.

DETAILS OF TEST.

Crushed stone was hauled from a railroad team track near the center of the city of Kenosha, a distance of 2.7 miles to the site of the concrete road under construction. One-half of the distance traveled was over a recently completed concrete road, the remainder was over roads of the type shown in Table I.

To avoid undue congestion at the team track a portable hopper loader, Fig. 1, was used. This hopper held 8 cu. yds. and was intended to be kept filled continuously by the shovelers. A load of 1½ cu. yds. was taken on in 2 minutes, with the aid of this loader. In the test five round trips were made. There was no special preparation for the test, the trucks being operated in connection with the team hauling.

On the first trip a 1½-cu. yd. Jeffery Quad truck was used, with one 1½-cu. yd. Bain dump wagon trailing. The time required for the various operations was as follows:

Item.	Time in minutes.
Loading	6
Coupling to trailer	3½
Time of loaded haul to job	24
Dumping and coupling for return	16½
Return trip empty	13½
Total time of round trip, 5.4 miles	63½
Remarks—	
Length of loaded haul, miles	2.7
Gravel hauled, cu. yds	3½
Speed through sand, miles per hour	2
Average loaded speed, miles per hour	6.75

On the second trip two trailers were used and the average loaded speed was 7.28 miles per hour. The loading time for the truck was 2¾ minutes. The first trailer was picked up empty and loaded from the hopper. The second one was already loaded, and coupled behind the other with very little delay. Three other trips were made, with intervals between two of them to allow of both trailers being picked up already loaded.

CONCLUSIONS FROM THE TEST.

The following conclusions were derived from the test:

1. To use four trailers, leaving two at the team track to be loaded while the others were being trailed.

2. To hire enough extra shovelers at the team track to keep the hopper and the two extra trailers always full when the truck arrived with its two empty trailers at the end of each trip. The two loaded trailers were coupled together and ready to be fastened by a clevis to the spring-supported trailer ring on the rear frame member of the truck without loss of time.

3. To use one of the teams on the job to reverse the trailer wagons after dumping their loads at the end of the run, while the truck was making its turn in the deep sand off the end of the concrete road. (The road was only 14 ft. wide at this point.)

4. To let the truck driver do nothing but drive his truck, and leave to the other men such operations as coupling, uncoupling and dumping both the trailers and truck.

ESTIMATE OF COST OF HAULING.

Based on data obtained in this test and other data derived from other sources an estimate was made of the cost of operating truck and trailers in hauling gravel on this job.

TABLE I.—SHOWING THE LENGTHS OF VARIOUS KINDS OF ROADS TRAVERSED ON HAUL.

Pavement.	Miles.
Brick (good)	0.20
Concrete (good)	0.05
Asphalt (good)	0.05
Brick (good)	0.50
Macadam (fair)	0.05
Dirt (fair)	0.05
Loose sandy dirt (bad)	0.40
Concrete (excellent)	1.35
Sand (very bad)	0.05
Length of haul	2.70

The time of the series of operations required to make a full round trip would be as follows:

Item.	Time. Mins.
Time to load truck	2
Time to pick up loaded trailers	¾
Running time to job	22¼
Time to dump truck and trailers, turn trailers around, reverse truck, and couple up again	9¾
Running time to team track, truck and trailers empty	15¼
Total	50

TABLE III.—TRAILER STATEMENT.

Fixed Costs, Per Day:	
Interest on investment of $1,000 (four Bain 1½ yd. patent dumps at $250 each) at 6 per cent, is $60 a year, which, reckoning 300 working days to the year, is, per day	$0.20
Yard charges for storage, four trailers, at $3 a month each, are $12, which, reckoning 25 working days per month, is, per day	0.48
Painting and annual overhaul (say $60 a year for four of them) amounts, per day, to	0.20
Total fixed costs, per day	$0.88
Variable Costs, Per Mile:	Cts.
Depreciation on two trailers (two running while two are being loaded), per mile	1.0
Grease, etc., for two trailers, per mile	0.3
Repairs and maintenance, for two trailers, per mile	1.0
Total variable costs, per mile	2.3

At this rate, the truck and its trailers can make exactly 12 round trips in a 10-hr. day.

The test further showed that rather more than 5 miles were averaged to a gallon of gasoline, including both the outgoing and incoming journeys. The oil figures were also carefully kept and appear in Table II. The tire cost is an estimate made after careful and

Fig. 1. Method of Loading Motor Truck Used in Hauling Test at Kenosha, Wisconsin.

TABLE II.—TRUCK STATEMENT.

Fixed Costs, Per Day:	
Interest on investment of $3,350 ($2,750, cost of chassis, plus $600, cost of dump body) at 6 per cent, is $201 a year, which, reckoning 300 working days to the year, is, per day	$0.67
Garage rent, dead storage, $10 month, which, reckoning 25 working days to the month, is, per day	0.40
Painting and annual overhaul (excluding running repairs) $150, which, reckoning 300 working days to the year, is, per day	0.50
Insurance and taxes, per year, $180, or per day	0.60
Driver's wages, per day	3.00
Total fixed costs, per day	$5.17
Variable Costs, Per Mile:	Cts.
Tires, per mile	1.78
Depreciation of chassis (less tires), assuming life of 7½ years at 65 miles per day, is, per mile	1.67
Depreciation on dump body (value $600, life 5 years), is, per mile	0.60
Gasoline, average 5 miles to the gallon (truck loaded and unloaded), at 15 cts. per gallon (to allow for possible tax increases), is, per mile	3.00
Oils, waste and grease, per mile	0.40
Repairs to truck, per mile	3.00
Repairs to dump body, per mile	0.40
Total variable cost, per mile	10.85

TABLE IV.—QUAD AND TRAILER STATEMENT.

Total Fixed Costs, Per Day:	
Quad truck	$5.17
Trailers	0.88
Total	$6.05
Total Variable Costs, Per Mile:	Cts.
Quad truck	10.85
Trailers	2.30
Total	13.15

TABLE V.—HAULING COSTS WITH QUAD 2-TON TRUCK AND FOUR TRAILERS, TWO WORKING AND TWO AT TEAM TRACK, ON ROAD BUILDING OPERATIONS NEAR KENOSHA, WISCONSIN.

No. of Trips Per Day.	Miles Per Day.	Fixed Cost Per Day.	Running Costs at 13.15 Cts. Per Mile.	Total Cost Per Day.
5	27.0	$6.05	$3.55	$ 9.60
6	32.4	6.05	4.26	10.31
7	37.8	6.05	4.97	11.02
8	43.2	6.05	5.68	11.73
9	48.6	6.05	6.39	12.44
10	54.0	6.05	7.10	13.15
11	59.4	6.05	7.81	13.86
12	64.8	6.05	8.52	14.57

long-continued tractor and trailer tests on these very roads. This figure may be too low for the average roads in the same class of work, but it is very accurate in this particular case.

TABLE VI.—COSTS PER YARD AND YARD-MILE OF HAULING GRAVEL BY QUAD TRUCK AND TRAILERS, AS AGAINST [illegible] A YARD BY HORSES, ON ROAD BUILDING OPERATIONS NEAR KENOSHA, WISCONSIN.

No. of Trips Per Day.	Total Cost Per Day.	No. of Yards Hauled Per Day.	Cost Per Yard Hauled.	Cost Per Yard-Mile.
5	$ 9.60	22.5	$0.42	$0.15
6	[illegible]	27.0	0.38	0.14
7	11.02	31.5	0.35	0.13
8	11.73	36.0	0.32	0.12
9	12.44	40.5	0.30	0.11
10	13.15	45.0	0.29	0.11
11	13.86	49.5	0.28	0.11
12	14.75	54.0	0.27	0.10

The depreciation item, Table II, is one which must be considered in connection with the annual overhaul charge and the repairs item. A life of about 7½ years is assumed for the truck, and if the machine made 12 trips a day on such a job as the one under consideration it would run about 65 miles a day, or approximately 20,000 miles a year. At this rate the 3 cents a mile allowed for repairs amounts to $600 a year, which, added to the $150 a year for the annual overhaul, makes a total of $750 a year to keep the truck in first-class condition.

In 7½ years the truck would travel 150,000 miles, and in this time the sum allowed for upkeep amounts to $5,550, or more than double the original investment in the chassis, even with the tires included. And it should be further noted that the tires are well provided for in another item.

The figures shown in Table IV indicate that with an outfit consisting of a Quad truck and 4 Bain dump wagons (two always being loaded at the track) the cost of operation on this job would be $6.05 fixed cost per day, plus 13.15 cents a mile for each mile operated.

Table V shows the assumed cost of operating this hauling outfit for 5, 6, 7, 8, 9, 10, 11 and 12 trips a day. If the truck is operated more than 10 hours a day the total costs can be ascertained by adding the extra mileage cost at 13.15 cents a mile plus the cost of extra driver.

Table VI shows these costs worked into cost per yard hauled and the cost per yard mile, not counting the cost of extra shovelers that may be needed at the team track.

[illegible]

This test was undertaken by the Thos. B. Jeffery Co. of Kenosha, Wis., manufacturers of the motor truck described, in co-operation with George Wade, the contractor on the road in connection with which the test was run, and the data given was presented in a copyright pamphlet issued by that firm.

Unit Costs of Resurfacing Portions of the Niagara River Road in Queen Victoria Park, Canada.

In reconstructing a five-mile section of the Niagara River Road in Queen Victoria Park cost data were obtained of considerable interest. The road was formerly constructed as waterbound macadam, the width of macadam pavement being 18 ft. Reconstruction work was divided into three parts, i. e., light resurfacing, heavy resurfacing in which the entire top course was reconstructed, and a bituminous carpet treatment of another section of the same road. Contractor's profit and overhead cost are excluded.

LIGHT RESURFACING.

On this section, which was 2.77 miles in length, the work consisted of spiking up the old surface with a steam roller, repairing depression with new stone when needed, shaping by raking, and watering and rolling. The cost of this work is given in Table I.

TABLE I.—LIGHT RESURFACING WATER-BOUND MACADAM ROADWAY.

Conditions:
Aug. 5, 1913, to Oct. 21, 1913, from Slater's Dock, South.
Average length of haul—3.4 miles from M. C. R. siding, Chippewa.
Length, 14,625 ft. = 2.77 miles; width, 18 ft. = 29,250 sq. yds.

	Total cost.	Cost per sq. yd., cts.
Labor:		
Loading 2-in. stone and screenings	$ 232.56	.8
Hauling	442.95	1.51
Pumping and watering	45.29	.15
Repairing roadway	275.18	.94
Rolling and spiking	97.29	.33
	$1,093.27	3.73
Material:		
2-in. stone—205.5 tons at $1.25	256.88	.88
Screenings—150.2 tons at $1.00	150.20	.51
	$ 407.08	1.39
Summary:		
Labor	1,093.27	3.73
Materials	407.08	1.39
Totals	$1,500.35	5.12
Cost per mile	540.00	....

Remarks.
297 cu. yds. of stone and screenings were placed on 29,250 sq. yds.
1 cu. yd. of stone and screenings was placed on 98.5 sq. yds.
Ratio of 2-in. stone to screenings used—1 to .731.
Ton-mile cost of hauling materials—36.2 cts.
Wage rates:
Teams, per hour $0.55
Laborers, per hour .22
Foremen, per hour .30

macadam construction, by flushing and rolling. The cost of this work is given in Table II.

HEAVY RESURFACING.

On this section, 2.74 miles in length, the top course of the road was scarified, new stone added and the top course finished as in new

TABLE II.—HEAVY RESURFACING WATER-BOUND MACADAM ROADWAY.

Conditions:
Aug. 1, 1913, to Dec. 1, 1913, from Black Creek, North.
Average length of haul—1.98 miles from Black Creek siding.
Length, 14,467 ft. = 2.74 miles; width, 18 ft. = 28,934 sq. yds.

	Total cost.	Cost per sq. yd., cts.
Labor:		
Loading 2-in. stone and screenings	$ 521.00	1.8
Hauling	1,509.00	5.2
Pumping and watering	215.00	.8
Repairing roadway	547.00	1.9
Rolling and spiking	425.00	1.5
	$3,217.00	11.2
Material:		
2-in. stone—750 tons at $1.10	825.00	2.9
Screenings—[illegible] tons at $1.00	357.00	1.2
	$1,182.00	4.1
Summary:		
Labor	3,217.00	11.2
Materials	1,182.00	4.1
Totals	[illegible]	[illegible]
Cost per mile	1,600.00	....

Remarks.
895.4 cu. yds. of stone and screenings were placed on 28,934 sq. yds.
1 cu. yd. of stone and screenings was placed on 32.3 sq. yds.
Ratio of 2-in. stone to screenings used—1 to [illegible]
Ton-mile cost of loading and hauling materials [illegible]
Wage rates:
Teams, per hour $0.45
Laborers, per hour .20
Foremen, per hour .30
At Queen Victoria Park rates for teams, 55 cts., and men, 22 cts., the above ton-mile cost would be 38.4 cts.

BITUMINOUS CARPET TREATMENT.

A section of road 2.77 miles in length was covered with a bituminous carpet. A refined tar known as "Tarvia A" and ½-in. stone chips were used. The tar was shipped in tank cars to the nearest railway siding and heated by means of a steam boiler to a temperature of 100° F., when it was forced by steam pressure into the distributing apparatus, and then hauled to the site of the work and attached to the steam roller. Connection was here made to the boiler of the roller and the tar heated to a temperature of between 175° and 200° F. Steam pressure at 35 lbs. was then applied to spray the tar from the distributing machine onto the road. The distributing apparatus used was supplied with nozzles at the rear through which the hot tar was forced in a fine spray.

The portion of the hot tar that did not penetrate the road surface was absorbed by ½-in. stone chips applied at a rate of 1 cu. yd. of chips on 65 sq. yds. of surface, the depth of stone and tar being slightly over ½ in. Tar was applied at a rate of ½ gal. a square yard. As shown in Table III the total cost, including labor, was $0.123 per square yard, or about $1,800 per mile for an 18-ft. roadway.

It is estimated that the only cost of upkeep

Fig. 1. Bituminous Carpet Treatment of a Road in Queen Victoria Park.

for this surface will be an annual tar spraying, using about ⅛ gal. of tar per square yard, at a cost of between 3 and 4 cts. per square yard, or about $320 per mile for an 18-ft. roadway. Figure 1 shows the type of surface obtained.

TABLE III. — BITUMINOUS CARPET OF TARVIA A AND ½-IN. STONE.

Conditions:
Sept. 2 to October 16, 1913, vicinity of Usher's Creek.
Length of haul 3.4 miles.
Length, 14,625 ft. = 2.77 miles; width, 18 ft. = 263,250 sq. ft. = 29,250 sq. yds.
Depth—½ in.

	Total cost.	Cost per sq. yd., cts.
Costs:		
Loading, hauling and placing stone	$ 861.05	2.90
Loading, hauling and placing tarvia	353.71	[illegible]
Placing and removing plant	56.50	.19
	$1,271.24	4.28
Materials:		
½-in. stone—487.5 tons at $1.30	633.75	2.14
Tarvia "A"—14,307 gals. at 10c	1,430.70	5.58
Freight, $188.35; demurrage, $32	220.35	
Soft coal for heating and operating roller	88.90	.30
Totals	$2,373.70	8.02
Summary:		
Labor	1,271.24	4.28
Materials	2,373.70	8.02
Totals	$3,644.94	12.30

2.77 miles cost $3,644.94
1 mile cost 1,300.00
1 square yard cost 0.1230

ACKNOWLEDGMENT

The data from which this article was prepared were collected by John H. Jackson, superintendent of Queen Victoria Park, and were contained in his report to the Park Commissioners.

Methods and Cost of Reconstructing Old Brick Pavements By Turning the Brick in Savannah, Georgia.

(Staff Article.)

During the last year 12,635 sq. yds. of brick paving in Savannah, Ga., has been reconstructed by taking up the worn and uneven pavement, Fig. 1, culling out the broken, badly worn or disintegrated paving brick, and relaying with the bright edge of the brick uppermost, the result obtained being shown in Fig. 2.

The method employed in accomplishing this work was to remove the old brick from the pavement, culling out and discarding brick unfit for relaying, and piling the good brick at convenient points. The sand cushion was then formed to the desired cross section, adding new sand where necessary, and the old bricks laid in the usual manner but with the unworn edge uppermost. It is readily seen that this process is only feasible where a sand or bituminous filler is used.

The cost of turning 10,400 sq. yds. of brick pavement on West Broad St. was $3,490.39, or 33.5 cts. a square yard. Of this cost, 25 cts. a square yard was for labor and 8½ cts. for material. On State St., 2,235 sq. yds. of pavement were turned at a cost of $432; or 19.3 cts. a square yard. It was necessary to replace about 10 per cent of the old brick with new ones.

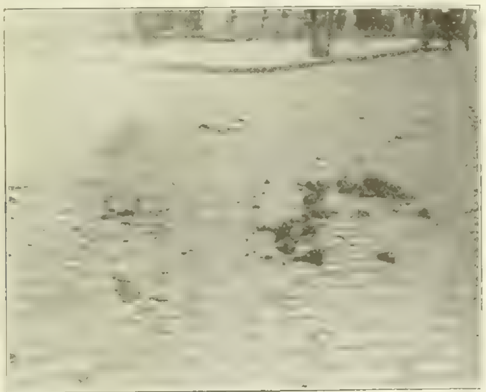


Fig. 1. Old Brick Pavement in Savannah, Ga., Prior to Turning.

It is the opinion of E. R. Conant, city engineer of Savannah, to whom we are indebted for the data given here, that the life of the surface has been increased at least three or four years, and perhaps longer. A recent inspection of the relaid pavement shows the surface to be even and smooth.

Methods and Cost of Resurfacing Water Bound Macadam With Rocmac Macadam.

During the past year a section of heavy traffic macadam road in the Queen Victoria-Niagara Falls Park was resurfaced with rocmac macadam. The work was accomplished under the direction of J. H. Jackson, superintendent of the park, and is described in his annual report.

The section of driveway resurfaced is on the Rambler's Rest loop about ½-mile from the Victoria Park railway station. This portion of the drive is subjected to a very heavy traffic. To secure a proper grade and prepare a suitable foundation it was necessary to loosen and remove the old macadam wearing surface to a depth of 4 ins. This work was accomplished, as shown in Table I, at a cost of 11.26 cts. per square yard.

METHOD OF CONSTRUCTION.

The rocmac wearing surface was constructed in the following manner: Upon the rolled stone base of the old macadam road was deposited a layer of rocmac matrix about 1⅓ ins. in thickness. Rocmac matrix consisted merely of a mortar made of limestone screenings mixed with rocmac solution. Rocmac solution is a liquid silicate of soda which reacts with carbonate of lime contained in the limestone screenings and assumes a set similar to that of Portland cement, although differing markedly from Portland cement in that chemical action is not completed immediately but is recommenced with each addition of moisture after the road is completed. This fact tends to prevent the wearing of holes since abraided parts of the surface heal over after a rain. The rocmac matrix, or mortar, was covered with 4 ins. of coarse crushed stone and rolled. This rolling forced the stone down into the matrix, filling the voids in the stone. As the matrix rose to the surface it was broomed forward and diagonally across the road and rolling continued until a smooth even surface was secured. The road surface was then sprinkled with dry limestone screenings.

COST.

Table I gives the detailed cost of this work. The resurfacing was accomplished by day labor.

TABLE I.—COST OF RESURFACING OLD MACADAM WITH ROCMAC MACADAM IN QUEEN VICTORIA PARK, CANADA.

Labor—	Total.	Per sq. yd. Cts.
Removing old surface (96 cu. yds.)	$ 97.20	11.26
Hauling 144 cu. yds. of stone and screenings from Victoria Park station	119.08	13.80
Rolling	28.00	3.25
Resurfacing	63.16	7.31
Foreman	60.20	6.97
Totals	$367.64	42.59
Material—	**Total**	**Per sq. yd. Cts.**
2-in. stone, 114 cu. yds.—136.8 tons at $1.25	$171.00	19.82
Screenings, 30 cu. yds.—36 tons at $1.00	36.00	4.17
Rocmac solution, 374 gals. at 45c.	168.30	19.50
Totals	$375.30	43.49
Summary.	**Total.**	**Per sq. yd. Cts.**
Labor	$367.64	42.59
Material	375.30	43.49
Totals	$742.94	86.08

Note.—Length of haul—3,400 ft. = .644 mile (from Victoria Park siding). Area treated—Length 370 ft., width 21 ft. = 7,760 sq. ft. = 863 sq. yds. Depth removed—4 ins. Material removed—370 ft. x 21 ft. x 4 ins. = 2,590 cu. ft. = 96 cu. yds. (in place).

Notes on Road Location in Pennsylvania and Limiting Grades for Various Pavements.

In a pamphlet of instruction to employes, S. D. Foster, chief engineer of the Pennsylvania highway department, gives some interesting notes on road location and satisfactory grades for various pavements.

The location of most roads which come under consideration for reconstruction by the engineer are usually physically established and invariably require lengthy legal process to change. If a change is necessary to insure the future safety of the traveling public and economic maintenance, the principles of proper location of this change will become applicable. Proceed by establishing a line between the objective points that will avoid swampy or unsuitable soil if possible and give due thought to the economic possibilities in the problem of construction. The shortest distance between the two points may or may not be the proper line to adopt, as the first cost may be reasonable, but conditions may be perpetuated which will accrue to the account of heavy maintenance charges, also the purpose of constructing a highway—to meet future demands of travel—may be defeated.

It is well to keep in mind the direction of the heavy hauling and, if topographically possible and practicable, to arrange the grade line so that the maximum grades will be descending grades in this direction. If we have a certain market or shipping center to reach, the profile should be carefully studied to ascertain the maximum grade along the road which cannot be eliminated for topographical or other reasons. With this information as a guide, considerable money can be saved by avoiding ruthless slashing of knolls and excessive filling in other places.

As the grade increases, the load a team can pull rapidly decreases; for instance, only one-half as large a load can be drawn on a 6 per cent grade as on the level, and one-fourth as much on a 10 per cent grade.

It is readily seen that considerable study and forethought must be given to this question. Grades should be established so as not to damage abutting property unnecessarily and regulated to provide properly for drainage, underdrainage and foundation. On existing old stone roads a considerable saving may be effected by following the old road surface as nearly as possible so as not to disturb this as a foundation.

The determination of surface aside from the grades will depend largely on the funds available and the nature and amount of traveling to which a road will be subjected.

Fig. 2. Brick Pavement After Turning, Showing the Type of Surface Resulting.

Table I shows the limiting grades, maximum and minimum, which may be used in the several types of pavement which do not receive daily attention and where there are no curbs and water is carried in the side ditches, making it possible to secure additional fall between cross drains.

TABLE I.—LIMITING GRADES TO BE USED ON PENNSYLVANIA STATE ROADS.

Class.	Maximum Grade. Percent.	Minimum Grade. Percent.	Adaptable to Traffic.
Sheet asphalt	[illegible]	[illegible]	Municipalities.
Brick	8	6	Municipalities and interurban
Brick (grooved)	12	[illegible]	On steep grades.
Asphaltic concrete	[illegible]	[illegible]	Municipalities and interurban
Cement concrete	6	55	Suburban.
Cement concrete (Bit. Sur. treatment)	7	[illegible]	Suburban.
Telford macadam	14	8	Rural.

Notes on the Selection of Pavements for Heavy Traffic Roads.

The requirements of pavements used on heavy traffic roads are quite different from those bearing a light traffic. In a paper before the American Road Congress Henry G. Shirley, state highway engineer of Maryland, outlined the factors involved in the selection of pavements for use on country roads and his paper is given here. Some costs of maintenance work on Maryland roads were given [illegible] June 17, 1914.

Traffic.—The rapid change that has taken place, and which is daily taking place in the character of traffic on our highways, makes the selection of the type of surface more difficult each day for the highway engineer. In selecting a type of surfacing for any particular road, the engineer not only has to study the amount and kind of traffic that daily passes over the road, but has to make a very comprehensive study of the amount and kind of traffic that will probably pass over the road in the future, by virtue of the development of the surrounding territory on account of the improved road.

The writer has made studies of roads where the traffic, before improvement, consisted of light vehicles and nothing heavier than 2-horse loads, but as soon as the road was reconstructed, the amount of traffic increased from 50 to 300 per cent, and the loads from light 2-ton loads to 10 to 12-ton motor trucks, and 14 to 18-ton tractors. He also recalls constructing a section of road through a very sparsely settled section, and estimating that it would be quite a long time before the adjacent territory would be more thickly populated, and accordingly selected a soft local limestone for the metal surfacing, but which had sufficient strength and hardness to carry the traffic that was passing over the road at that time. Scarcely had the road been completed when several large tracts of woodland, not a great distance from the road, were cut down, and the lumber was transported on wagons, drawn by large traction engines with cleats, over the road to the railroad station. The effect of this heavy traffic on the soft limestone surface can be easily surmised.

Before selecting the type of pavement to be used, a close and accurate census of the different kinds of traffic should be taken, a very thorough study made of the surrounding section, and an estimate made as to the possible increase of the different kinds of traffic, or the decrease of one kind and the large increase of the other. It is the opinion of the writer that in no other line of engineering should there be a larger factor of safety used than in estimating the amount, intensity, and kind of motor and self-propelled traffic that will pass over our improved roads in the near future. The great change in the character of traffic developed in the past five years is but a small index to what can be expected in the next five years to come.

Types of Pavement.—The types of pavements used on heavy traffic roads should be selected as to their fitness to stand the kind and intensity of the traffic that will travel them. Roads in the outlaying districts, where horse-drawn traffic comprises the larger percentage should be constructed of macadam with a light surface treatment. Concrete will also be found serviceable and desirable. Where motor traffic is in the majority, bituminous macadam or concrete will give good results. Near the centers of population, where the traffic is mixed and heavy, concrete, bituminous concrete, asphalt or vitrified brick will prove the most economical. Where the heavy traffic is concentrated, brick, asphalt or stone block are the most suitable.

There can be given no hard and set rule for selecting the type of construction that should be used on a given section of road to carry a known traffic. For local conditions, the availability of materials, etc., play such an important part in the selection of the type of surfacing in any locality, that each individual case must be worked out on its own merits.

Criterion for the Selection of the Type of Pavement.—The following method of selecting a type of surfacing to carry an estimated traffic, however, will prove fairly accurate where a study can be made and the maintenance cost can be had of roads constructed and maintained under similar conditions:

Where the annual cost of maintenance of a less durable type of road surfacing will exceed the annual cost of maintenance of a more durable type of surfacing, plus 4 per cent on the excess cost of the more durable type over the less durable type, the more durable type should be used, and vice versa.

The maintenance on heavy traffic roads should be continuous and thorough—never allowing the surface to remain broken any length of time, but as soon as the slightest defect or indication of failure appears, it should be speedily repaired.

Drainage, Width, Foundation and Thickness.—Drainage of a road-bed that is required to carry heavy traffic, should be well taken care of by tile or other sub-surface drains, so as to render the sub-foundation as dry and firm as possible. The maximum grade should not exceed a 6 per cent, and the alignment should be as straight as possible, with all sharp curves and bends eliminated. The width of the roadway and the width and thickness of the metal surfacing should be designed to meet the requirements of the present as well as the future traffic which it will have to accommodate, but the minimum width should not be less than 30 ft., nor the metal surfacing less than 18 ft. Broken stone or gravel make a fair foundation, but concrete is almost as cheap and is more preferable.

The thickness of macadam and gravel should not be less than 5 ins. after rolling, nor more than 10 ins., while concrete should not be less than 4 ins., nor more than 8 ins., depending primarily upon the character of the soil of the sub-base, and the intensity and character of traffic it will have to sustain. In some cases where the loads are very heavy, but the number of loads small, it has been found economical to lay a strip of high-class and durable pavement in the middle of the road for a width of 9 to 14 ft., with a cheaper and less durable material on each side.

Motor Hauling.—There is a great necessity for adequate laws to regulate the heavy loads that have to be borne by the surface of the many hundreds of miles of roads that have been and are daily being constructed. The manufacturers of tractors, motor trucks, and other hauling engines, have given but little study to the effect and injury that is being done and may be done by the heavy loads propelled over the road surface, and the strain and stress caused by narrow tires, steel cleats, ribs, and other devices, but it seems that the greater amount of their energies have been to develop an engine or motor truck that will haul the largest load in the shortest period of time, using the least amount of fuel—all of which is very commendable, but it is the duty, as well as to the welfare of these manufacturers, to devote quite as much energy and brains in constructing their engines and motor trucks in such a way that the least amount of damage will be done to the surfacing in passing over the roads. It is most important that the manufacturers of hauling equipment and highway engines should work together in framing a set of adequate laws controlling the use of hauling equipment over improved roads, as well as developing wheels and other devices so as to do the road surfacing the least amount of damage. By co-operating and working together, large sums can be annually saved on maintenance, which will greatly benefit all concerned.

WATER WORKS

Method and Cost of Constructing and Repairing Submerged Water Pipe Lines at Portland, Ore.

The present gravity water supply for the city of Portland, Ore., is taken from Bull Run River, in the Cascade Mountains, and is brought thence to the eastern border of the city through two steel conduits 24 miles in length, one 33 to 42 ins. in diameter, built in 1893 and 1894, and one 44 to 52 ins. in diameter, completed in 1911. The supply for the West Side, or main business district of the city, is brought across the city through a 32 in. cast iron main, the river being crossed by one 24-in. and one 30-in. flexible joint steel main. The present article describes the pipe laid for the river section of this West Side conduit, the construction methods and costs, and the methods subsequently employed in repairing and reconstructing the submerged pipes as originally laid. The work described covers: (1) The laying of a 28-in. cast iron, flexible-jointed pipe in 1894; (2) the laying of a 24-in. steel flexible-joint pipe in 1898; (3) the repair of the 28-in. cast iron, flexible-joint pipe in 1909; (4) the laying of a 30-in. steel, flexible-joint pipe in 1911; and (5) the work undertaken during 1913 and now just completed, consisting in taking up and relaying at greater depth the 24-in. line originally laid in 1898, and taking up and storing for future use at another point the 28-in. cast iron pipe laid in 1894. The information given is from a paper entitled: Submerged Pipe Work at Portland, Ore., prepared by Mr. D. D. Clarke, Engineer of the Portland Water Bureau, for presentation before the American Society of Civil Engineers, and published in the Proceedings of the Society for November, 1914, Vol. XL, pp. 2657 to 2675.

28-in. Line.—The pipe laid in 1894 consisted of 2,006 ft. of 28-in. cast iron, flexible-joint pipe, 1½ ins. thickness of shell, in net laying lengths of 15 ft. 8 ins.

	Per lin. ft.
The contract price for furnishing this pipe was a lump sum for a certain specified length, which equaled	$31.4275
The contract for [illegible]	2.50
Dredging, 18,746 cu. yds. at 40 cts.	3.738
Total	$37.6655

or for 2,006 lin. ft. of 28-in. flexible-joint pipe, in place, a total of $75,556. During 1894 the water department labor wage in Portland was $1.75 per day.

For details regarding this work reference is made to the paper, Transactions, Am. Soc. C. E., Vol. XXXIII (1895), p. 257, by Franklin Riffle and the late Albert S. Riffle, entitled "A Line of 28-in. Cast Iron Submerged Pipes Across the Willamette River, at Portland, Ore." This paper and the discussions thereon are exceedingly interesting and instructive.

24-in. Line.—The second pipe, the 24-in. submerged line laid in 1898, consisted of 2,041.6 lin. ft. of lap-welded steel pipe, ⅜-in. plate, including 58 flexible joints, of special design, placed at intervals varying from 20 to 40 ft., depending on the alignment and grade of the pipe trench.

Messrs. Smyth and Howard of Portland, Ore., were the general contractors for this work, the contract prices being as follows (including two joints and 72 lin. ft. of pipe not used):

60 flexible joints, $260 each	$15,600.00
1,938 lin. ft. of 24-in. pipe at $8.50	16,472.32
15,332 cu. yds. dredging at $0.40	6,132.80
2,041.6 lin. ft. pipe-laying, including joints, at $1.77	3,613.63
Total	$41,818.75

During 1898 the Water Department labor wage in Portland was $1.75 to $2 per day.

This work was undertaken in the late summer, after the subsidence of the June freshet in the Columbia (which sometimes causes a rise of 20 ft. or higher at Portland), and was completed the same season.

The methods and appliances adopted for this work were similar to those adopted in laying the 28-in. line, four years before. A ladder-dredge was used in excavating the pipe trench, and the excavated material was taken away on barges to be utilized in filling low ground adjacent to the harbor line in another portion of the city. A cradle was also used in laying the pipe in the excavated trench, as described in the paper by Messrs. Franklin and Albert S. Riffle.

The flexible joint used of the type shown in Fig. 1 was made in Portland, from designs prepared by the water department. Figure 2 shows the cradle with which the pipe was laid.

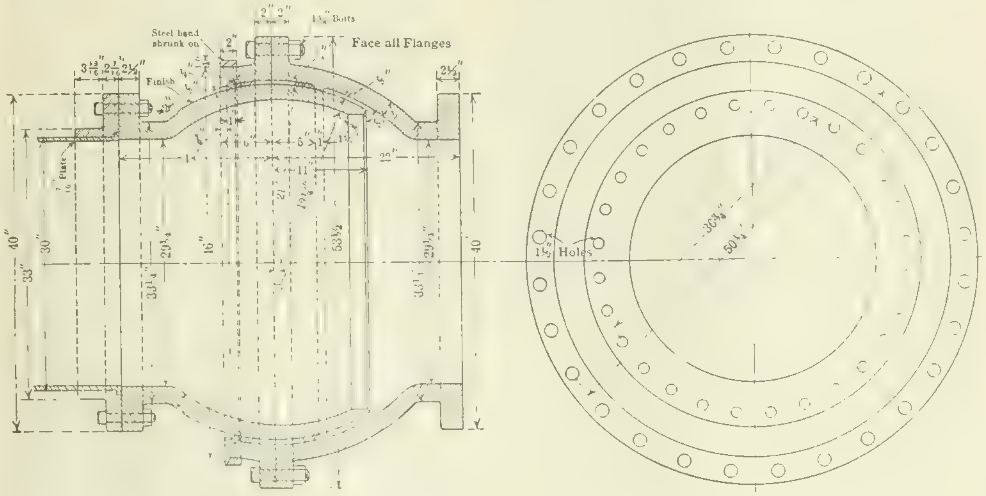


Fig. 1. Type of Cast Iron Ball and Socket Joint Used for 30-in. Submerged Steel Pipe Line at Portland, Oregon.

The pipe, after it was laid, was tested in a manner similar to that adopted for the 28-in. line in 1894. The results secured did not indicate that the pipe was absolutely tight, but they were accepted at the time as being the best that could be obtained. The final test, under a pressure of from 170 to 180 lbs., showed a leakage of approximately 7 gals. per minute, or 10,080 gals. per day, or about 8 gals. per lineal foot of joint, per day, under the pressures noted, which are approximately 50 per cent in excess of the normal working pressure.

The location selected for this pipe was parallel to and approximately 10 ft. from the 28-in. line, until the harbor line was reached on each side of the river. The line was then deflected southward, or upstream, until a point was reached 100 ft. from the 28-in. line, which it paralleled for the remaining distance.

The profile of the original line shows a 22 to 24-ft. channel, 100 ft. or more in width, near the west shore, and, for the larger part of the distance thence to the east harbor line, the low-water depth did not exceed 12 to 16 ft. In establishing the grade for the original line, it was decided first to place the pipe in an open trench dredged to a depth of 8 ft. below the bed of the stream, a cover of 4 ft. being thus provided which was deemed ample to protect the pipe from injury by the passing or anchoring of vessels. In view of the existence of at least a 23 to 24-ft. channel adjacent to the west harbor line, which it was thought would provide for all shipping interests in that portion of the harbor (being at the head of deep-water navigation), it was not thought necessary to incur the extra expense required to dredge a trench to the depth of 30 ft. or more below low water for the entire distance between harbor lines. The matter was compromised, therefore, by placing the pipe grade so as to permit of dredging a channel at least 16 ft. in depth at all points within the harbor limits. This required dredging in places to a depth of from 12 to 14 ft. with at least an 8-ft. trench at all points. The original pipe line (the 28-in.) was laid on a grade established as just described, and the second pipe (the 24-in.) also conformed to the same grade, as nearly as practicable.

For each of the pipe lines heretofore mentioned the flexible-joint portion was a little more than 2,000 ft. in length (2,006 ft. for the 28-in. and 2,041.6 ft. for the 24-in. line). This length covered the distance from river bank to river bank and reaching a point where the grade of the pipe trench was above low-water level in the river. For several hundred feet additional on each side of the river, standard cast iron pipe was laid to a connection with a concrete gate-chamber, where the two pipes were united with one 32-in. line forming a section of the main Bull Run Conduit for the supply of the West Portland District, the total distance between the east and west gate-chambers being 2,706 ft., and the approximate distance between harbor lines being 1,335 ft.

Repair of 28-in. Line.—As has been stated, the 28-in. line was completed in 1894 and the 24-in. line in 1898. It was then thought that the double line would provide amply for all the needs of the West Side District for a long term of years, and that, by means of the two lines, the possibility of any interruption to the supply was practically eliminated. In fact, such a feeling of security existed for more than nine years, or until a sudden break in the 28-in. line caused all dreams of security to vanish.

The break in this main was discovered early on the morning of April 10, 1909, when it was found that the water had been falling in the City Park Reservoir, No. 3, which was then being supplied through the broken main. Not being able to find any trace of a leak along the street mains leading to the river, the attention of the inspectors was turned to the submerged pipe crossing. Signs were then discovered of an under-water break at a point near the east shore and about 100 ft. back from the established harbor line, then built up with a dock owned and operated as a part of the terminal system of the Oregon Water Power and Railway Co., then operating a short line of railway leading from Portland to the valley of the Clackamas, a distance of about 40 miles.

It was impossible at first to tell in which line the break had occurred, as both the 28-in. and the 24-in. lines were in service at the time. Therefore it became necessary to shut the gates at each end of first one and then the other of the two submerged pipes, in order to see if the pressure on the main would come up again and the flow become re-established.

The first trial did not prove successful, the foreman in charge, in opening the gate that had just been closed and in closing those on the other line, became confused, in the haste and excitement of the moment, with the result that one of the gates shutting off the broken section was not fully closed, the effect being still to allow so much water to escape as to hold down the pressure far below normal. The conclusion immediately reached was that both pipes were broken.

No sooner was this condition reported to the Superintendent than he at once made arrangements for starting the pumps at the Palatine Hill Pumping Station, in order to supply the West Side District, the entire west side storage at that time being only about 35,000,000 gals. or approximately two days' supply available for all points. The Palatine

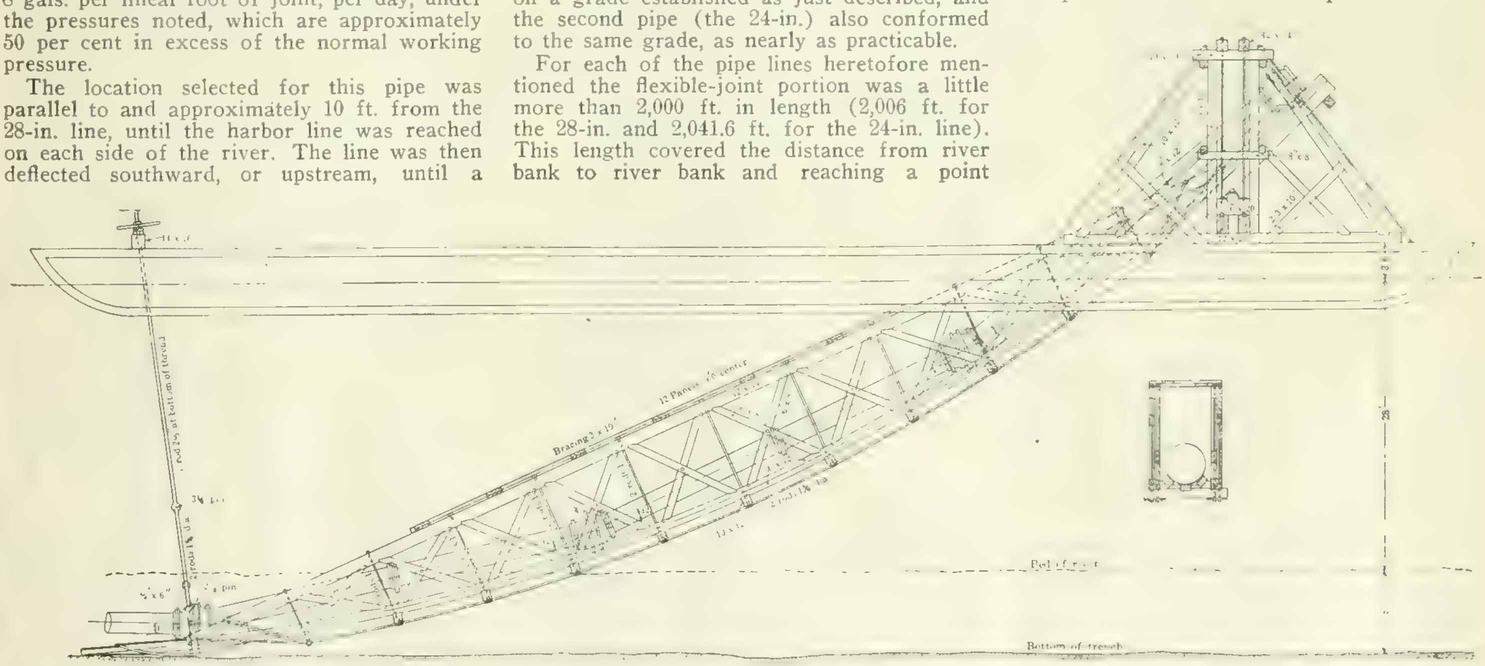


Fig. 2. Detail of Pipe Laying Cradle Used in Laying Submerged Pipe Under Willamette River, Portland, Oregon.

pumps were started at about 10 a. m. on April 11, for the first time since they were shut down on Jan. 2, 1895.

The pumping of Willamette River water into the city mains in the West Side District, as was done during three days (April 11 to 13, inclusive), was unfortunate, and, as it proved, entirely unnecessary, for, when the broken pipe was so nearly uncovered that one pipe could be distinguished from the other, the true condition of affairs was discovered, namely, that the 28-in. pipe was the only line broken.

Fortunately, no harm came from the infusion of Willamette River water into the city mains, although at one time it was feared that a typhoid epidemic might result. On the renewal of the Bull Run gravity supply, the pumps were shut down, the reservoirs and distribution mains were promptly flushed, and no unusual sickness was reported.

The cause of the break, so far as conditions have been diagnosed, was no doubt due to the strain caused by an unequal settlement of the pipe brought about by a gravel fill, 16 ft. or more in depth, which had been deposited over the pipe by the owners of the adjoining property. The pipe line right of way at this point was only 30 ft. in width. The railway company owning the property on each side used a suction dredge to fill in its property between the shore and the harbor line. This was done at the time of the June high water, when the entire tract was submerged. The dredged material was deposited at some little distance from the city property, but when the freshet had subsided it was found that the filled material had taken a very flat slope under water, and had actually buried the submerged pipes to a considerable depth. In the paper by Messrs. Franklin and Albert S. Riffle is noted the fact that "On the east side the bed of the river consisted of boulders and hard pan firmly cemented together," and it was at the point where this material was encountered in the pipe trench that the break occurred, presumably caused, as has been suggested, by the pipe not having a uniform bearing on the trench bottom, and hence being unequally loaded by the weight of the increased depth of fill.

It was the removal of the overlying deposit of sand and gravel which was largely responsible for the delay in uncovering and repairing the broken pipe, which was not completed until Oct. 11.

At first a clam-shell dredge was used in the removal of this material, but, later, a suction dredge produced more satisfactory results, the flat slope taken by this material under water greatly increasing the volume of the excavation required. When the pipe was uncovered sufficiently to admit of examination by a diver, it was found that one of the 15-ft. 8-in. lengths of the 28-in. cast iron pipe had been fractured diagonally, the break being about 12 ft. in length and extending from a point within a few inches of one bell joint for about three-fourths of the full length of the pipe section, and crossing the pipe from one side to the other, leaving a point about 1 ft. in width at the end of each fragment. These two pieces were separated about 3 ft. by the force of the jet at the time the break ocurred, the pressure at this point being about 150 lbs. per square inch.

In no case were the balls injured, but the fracture came so near to them that it was not possible to use a band or sleeve that did not cover the entire length of the fractured pipe, together with a ball at each end.

When the condition of the pipe was ascertained, the engineer reported the facts to the Water Board and suggested a possible method of repair. In view of the proximity of the 28-in. to the 24-in. pipe at the point where the break occurred, and the necessity for securing a continuous supply for the West Side without danger of interruption through the failure of the 24-in. line, the engineer decided that the safest plan would be to uncover the 28-in. line in the river at a point about 100 ft. from the east harbor line (the distance between the two pipes being approximately 75 ft.), and there, by means of the diver, make an under-water connection with the main and lay a new main thence to the east shore of the river, where a junction could be made with the old line above water level.

This would have required about 400 ft. of new pipe, which could have been quickly made of steel plates in local shops, and could have been laid from a platform along the north line of the pipe line tract. The only work in connection therewith requiring the services of a diver would have been the making of the joint with the old main in front of the harbor line. It was planned so that the use of three or four joints would have been sufficient for laying that portion of the pipe which would have been below water level. The estimated cost of this work was approximately $9,500.

The foregoing did not meet with the approval of the Water Board, owing largely to the cost, and it was therefore abandoned for the plan of constructing a split sleeve and encasing a full length of the broken pipe in its original bed, including a flexible joint at each end, the divers giving assurances that they could make the necessary joints under water.

Owing to the fact that at the point where the break occurred the two pipes were near to one another (approximately only 2 ft. between them for a short distance) and that the broken pipe was also about 2 ft. lower than the 24-in. steel pipe which must be kept in service, it was determined that, prior to undertaking the repair work, and in order to protect the 24-in. main thoroughly, a row of timber sheet-piling should be driven between the two pipes, so as to avoid any danger of undermining the steel main by the excavation for the broken pipe. The pressure on this main being approximately 150 lbs. at the bed of the river, it was decided to reduce this considerably before the piles were driven. This was done by shutting down the flow during the day, when the work of pile-driving was in progress, and turning it on again at night in order to maintain the supply in the City Park Reservoirs. One hundred and ten 9 by 12-in. "Wakefield" piles were driven, which occupied 5 days, and on the completion of this work, the additional dredging required to expose the 28-in. line fully was undertaken.

Preparatory to placing the split sleeve designed for this purpose, Fig. 3, the ends of the broken pipe were lifted partly from the position in which they were found, and also moved horizontally a short distance from the sheet-piling in order to give room for the sleeve. The fractured ends of the pipe were drawn together as closely as possible and then wired in place and afterward wrapped in heavy tarred canvas, securely fastened, the object of the latter being to prevent the cement grouting placed inside the sleeve from gaining admission to the interior of the pipe.

The sleeve was designed to be built of riveted steel plates, of sufficient diameter and length to take in an entire length of the pipe, with a flexible joint at each end. This meant that the main barrel of the sleeve must be about 4 ft. in diameter and 16 ft. long, with a reducer at each end, diminishing from 4 ft. to a diameter of about 32 ins. at each end in a distance of about 3 ft., thus fitting the outside of the 28-in. pipe, making the extreme length over all 22 ft.

A contract for the construction of this sleeve was awarded to the Moran Company for $1,026.50, and it was placed by department forces. The weight of the sleeve was 14,100 lbs.

The progress of the work was delayed by the high stage of water due to the June flood in the Columbia River, and time was also required to secure the necessary materials and manufacture the sleeve. The sleeve was not received until Sept. 8, but the placing of it was commenced immediately thereafter. The work of placing the sleeve, caulking the joints, testing the pipe, and grouting the space between the sleeve and the pipe occupied the time from Sept. 8 to Oct. 11, and on Oct. 12 the main was reported as being again ready for service.

For the longitudinal seams rubber packing was used, but the end joints were caulked with lead wool, or shredded lead, which was held in place by a steel ring fitting closely around the pipe and bolted to the sleeve.

In making the test for leakage, the small force pump used in testing the pipe could not raise the pressure to more than 40 lbs. per square inch and maintain it in the full length of the river crossing, the estimated leakage being at the rate of 119,500 gals. per day on the entire line between the gates, a distance of 2,706 ft. In February, 1910, it was again tested, and the leakage was noted as 38,000 gals. in 24 hours; a third test, in December, 1911, shows a decrease to 25,000 gals. per day under normal working pressure. When first tested, no leakage could be observed through the joints of the sleeve, and the matter was allowed to rest, as it was not considered of sufficient importance to warrant further investigation and expense.

It cannot be claimed that this work was done at small expense, or in an especially economical manner, the exigencies of the case demanding all possible speed without undue regard to cost.

The work of uncovering the pipe was delayed on account of its proximity to the rail-

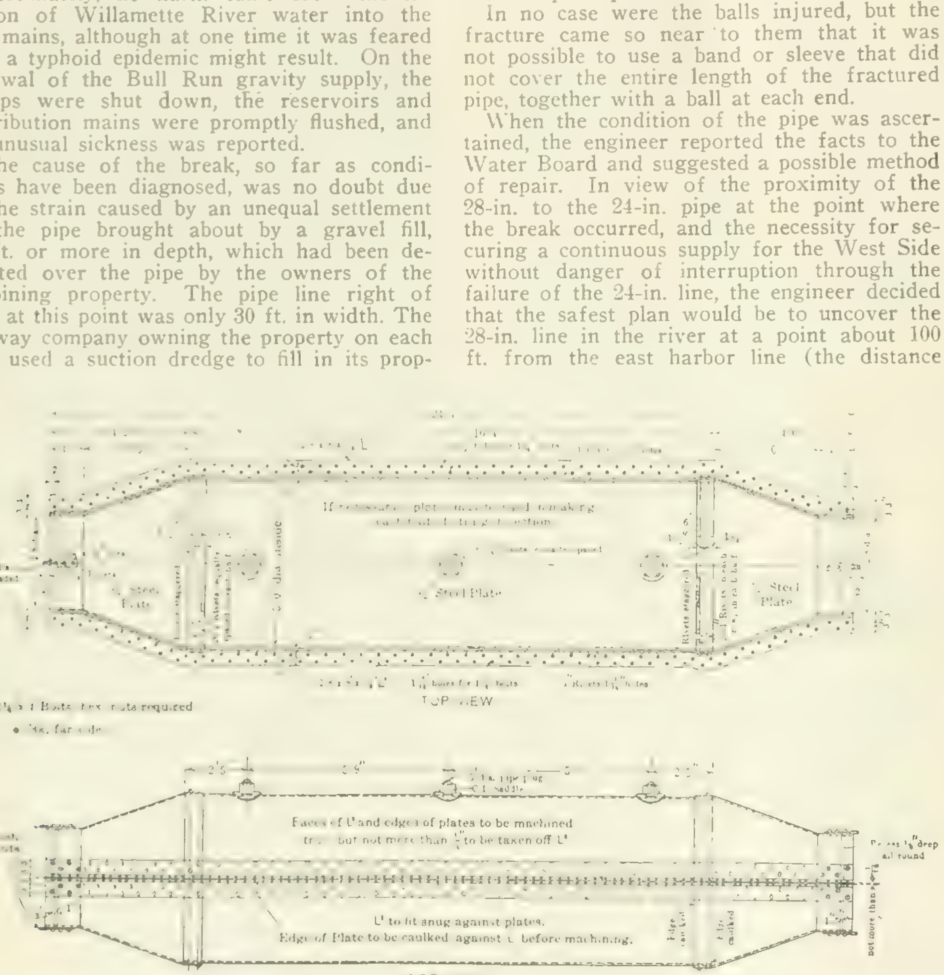


Fig. 3. Detail of All-Steel Split Sleeve for Repairing 28-in. Submerged Cast Iron Pipe, Portland, Oregon.

way dock, a section of which had to be taken out in order to admit the dredges; and, besides, it was necessary to exercise care not to injure the pipe during the dredging operations. The cost data follow:

The dredges were engaged by the day, and the total cost for them, including towage and removal of docks, etc., amounted to	$2,850.00
The split sleeve cost, f. o. b., Portland..	1,026.50
Placing and grouting the sleeve, including the sheet-piling and timber platform, cost	3,138.00
For the services of divers, who were in constant attendance while the pipe was being uncovered and the sleeve was being put in place, the allowance was	2,250.00
The total cost of the repairs, including the items just mentioned and incidental expenses, was	$10,750.00

Lowering the Pipes.—As already stated, the grade for the first pipes laid was established so as to provide for an available channel depth of only 16 ft. over the central portion of the upper harbor. It was not then anticipated that the development of the manufacturing and shipping interests in the southern part of the city would demand increased facilities so quickly.

As early as 1902, a demand was made on the Water Committee for the lowering of the central portion of the two pipes, in order to admit of a channel being dredged on a direct line from the draw-span of the Hawthorne Avenue Bridge to the Inman-Poulsen Co. mill—a large lumber manufacturing plant on the east side. Complaints and demands of a similar character were made nearly every season thereafter, but no action was taken thereon, except to call on the engineer for reports and estimates of cost.

In 1911 it finally became manifest that the shipping interests in the upper harbor would soon require improved facilities, and it was also learned that the plans of the Port of Portland and the United States Government engineers called for a 30-ft. channel in the upper harbor above the Hawthorne Bridge, it being claimed that this would suffice for the future needs of the port in that direction, regardless of the depths that will ultimately be required in the lower Willamette and Columbian Rivers, where even now a 40-ft. channel is dreamed of and talked about by the commercial bodies of the city and state as among the possibilities of the next few years.

Investigations and studies undertaken in connection with this work developed the fact that to lower the two submerged pipes so as to give a 30-ft. depth at low-water stage would practically necessitate the lowering of the pipes for the entire distance between harbor lines, approximately 1,335 ft., together with an additional distance at each end to provide for the approaches to the lower level, or a total of at least 1,500 ft. This would involve a large quantity of dredging, in addition to the adjustment of the pipes in the new trench.

The first plan prepared called for the construction of a series of pile bents of two piles each, with timber caps and ties, placed at intervals equal to one pipe length, from which the pipe could be suspended by slings, and rods with threads of sufficient length to permit the pipe to be lowered into the new position after the supporting earth had been removed and the new trench excavated. It was thought that the necessary excavation could be made from the side of the pipe with a suction dredge which could be manipulated so as to undermine the pipe gradually and admit of lowering it.

This plan was abandoned later, owing, in part, to the difficulty in handling the dredged material which had to be barged away and disposed of outside of the river channel.

The method finally adopted called for taking up the two pipes and, after making necessary repairs, relaying them in the trench excavated to the required level, as described later.

30-In. Line.—After considering the difficulties attending the lowering of the two pipes and the danger of breaking one or both of them, and the consequent interruption of the supply for the entire West Side, or main business district of the city, the Water Board decided that, in the interests of safety, a new and entirely separate submerged pipe line should be laid in advance of any work which might in any manner disturb the two pipes then in use. Plans and specifications were therefore prepared by the office staff calling for a 30-in. line crossing the river at the foot of Clay St., or approximately 600 ft. distant from the two pipes already in use.

The new line was planned to leave the 32-in. supply main at a point about 400 ft. east of the original East Side gate-chamber at Third and Stephens St., and to connect again with the 32-in. main at the West Side gate-chamber at Water and Mill Sts.

This plan called for 4,312.6 ft. of entirely new pipe. Of this, 2,265.6 ft. on the east and west sides were to be standard, 30-in., cast-iron pipe, Class "F," New England Water Works Association specifications, and the remaining 2,047 ft. were to be 30-in., lap-welded, galvanized and asphalted steel pipe, of 7/16-in. plate with flange joints. Inserted in the steel main, at intervals of about 40 ft., were placed 27 flexible joints of cast iron of the type shown in Fig. 1.

The grade established for the pipe trench between the harbor lines was 38 ft. below low-water level, which provided for a 3 to 4-ft. cover over the top of the pipe, with a 30-ft. depth of channel, as called for by the plans of the Port of Portland and the U. S. Engineers. This work was commenced in August, 1910, and completed in March, 1911.

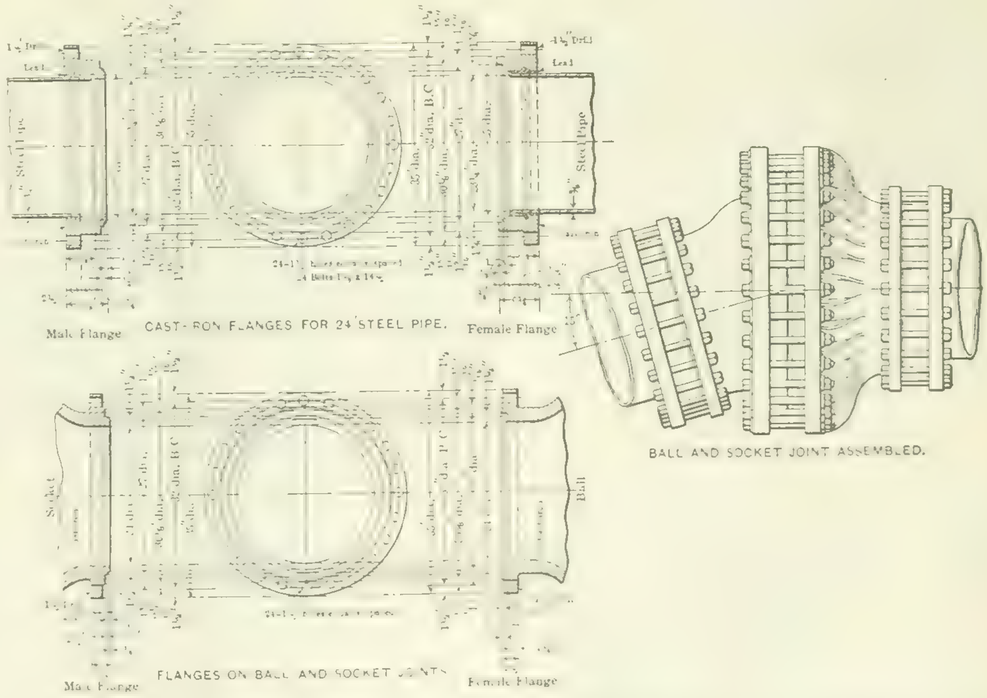


Fig. 4. Details of Flanges Used on 24-in. Submerged Steel Pipe at Portland, Oregon.

The dredging of the trench for this main was commenced by one of the suction dredges operated by the Port of Portland Commission, at a rental of $250 per day. The work with this dredge was soon discontinued, however, as it was needed for channel work at another point, and the dredging was completed with the dipper-dredge owned by the Pacific Bridge Co., this Company being in a position to utilize the dredged material for filling purposes in a nearby district.

The 30-in., standard, cast iron pipe used for the shore connections was manufactured by the Oregon Iron and Steel Co., at its foundry near Portland, the price, in place, being $11.10 per foot.

The flexible joints were made by the John Wood Iron Works, of Portland, from plans furnished by the Water Department, for $345 each. The 30-in., lap-welded steel, galvanized and asphalted pipe was purchased from the National Tube Co. (2,200 ft.) at $14 per foot.

The contract for laying the pipe and making the joints was awarded to Robert Wakefield and Co., of Portland, for the prices given in Table I.

TABLE I.—COST OF LAYING 30-IN. CAST IRON AND STEEL PIPE AT PORTLAND, OREGON.

Trench excavation, 5,705 cu. yds. at $0.90	$5,134.50
Cast iron pipe in place, 2,265.6 ft. at $11.10	25,148.16
Steel pipe, with flexible joints, laid, 2,047 lin. ft. at $4.50	9,211.50
Ball-and-socket joints (only 27 used), 30 at $345	10,350.00
Port of Portland dredge, estimated cost of 102,655 cu. yds	16,160.14
Pacific Bridge Co.'s dredge, estimated cost of 29,700 cu. yds	4,158.00
Valves, 30-in., 4 at $590	2,360.00
Concrete, in two-gate chambers, 59 cu. yds. at $12	708.00
Specials, 43,962 lbs. at 6 cts	2,637.72
Extras—Labor and materials, dock repairs, etc	10,197.84
Steel pipe used, 2,047 ft	28,658.00
Total	$114,723.86
Summary of total costs:	
Steel pipe	$ 29,135
Dredging	20,318
Cast iron pipe and pipe laying	65,747
Labor and engineering	4,808
Total	$120,008
Original estimate	125,000

On the completion of the work, the pipes, including the standard cast iron and flexible-joint steel pipe, were tested to 175 lbs. per square inch and the leakage under the normal pressure of 150 lbs. was found to be approximately 30,000 gals. per day.

Having become satisfied that this loss was from no one leak of special size, but rather from a number of small leaks, no attempt was made to check them. It may be stated that the last test was made on Sept. 28, 1912, when the seepage loss was at the rate of 5,000 gals. per day—a decrease of 25,000 gals. per day from that which occurred shortly after the main was laid.

Lowering of 24-Inch Line.—Soon after the completion of the new line, in 1911, a renewed application was made by the milling interests and the Port of Portland Commission, for the removal of the original submerged pipe lines, which were claimed to be a serious obstruction to navigation in the upper harbor.

An attempt was made to induce the Port of Portland Commission to share in the expense of this work, at least to the extent of contributing the use of its dredges for the removal of the material which it would be necessary to excavate to uncover the pipe and prepare the new trench at a lower level, but the inability of the Water Department to secure a satisfactory dumping ground prevented the consummation of the arrangement finally agreed upon, namely, that the Port of Portland would deduct from the sum due for the use of its dredges (at $350 per day) a sum sufficient to cover the cost of dredging a channel 300 ft. wide at the submerged pipe crossing a distance of 100 to 200 ft. on the line which had been planned, extending from the draw-span toward the upper river channel. This comprised a very small part of the total work required for this channel, which had been laid out in such a position as to oblige the city to lower its pipes for the entire distance between harbor lines, the natural deep-water channel near the west harbor line having been entirely ignored.

The conclusion of the whole matter was a decision of the Water Board to advertise for bids for the entire work, independent of any promise or expectation of aid from any source. It was furthermore decided to ask for proposals, on a unit basis, for the necessary dredging and the disposal of the dredged material; for uncovering and taking up both the 28- and 24-in. pipes; and for relaying the 24-in. pipe only—the 28-in. pipe to be stored for future use at some other crossing.

It was deemed that the two steel pipes, 30 and 24 ins. in diameter, respectively, would provide an ample supply for the West Side District, the estimated capacity of the two pipes being nearly 40 per cent in excess of that of the 32-in. line forming the shore ends of the submerged pipes and connected with the city reservoirs, and therefore the relaying of the 28-in. cast iron main would not be necessary or advisable under the circumstances.

Proposals were invited for doing the work on a unit basis, but a contract was finally awarded to Mr. A. C. U. Berry, of Portland, for the lump sum of $69,400, this being a considerable reduction from the unit prices bid, and to include the necessary dredging and the disposal of the material; the taking up and relaying of the 24-in. line; and the taking up of the 28-in. line. All necessary repairs to the 24-in. line before relaying were to be done by force account (actual cost plus 10 per cent).

The actual cost of this work to the department, as shown by the final estimate, equals $80,785, divided as follows:

85,000 cu. yds. sand and gravel dredging..	
6,960 cu. yds. shore excavation..........	
1,725 ft. 24-in. steel pipe removed........	
1,630 ft. 28-in. cast iron pipe, removed..	
1,714 ft. 24-in. steel pipe, relaid........	
Agreed price for above items	$69,400
Extras, at cost plus 10% profit—Repairs to 49 ball and socket joints for 24-in. pipe, including cleaning, machining, testing, painting, etc..................	3,438
Repairs to 1,714 ft. of 24-in. pipe, including cleaning, painting, repair of flanges, testing, etc., together with 808 lin. ft. of new riveted steel pipe, 24-in., at $5.15 per ft...........................	8,458
Miscellaneous extras.....................	478
	$81,774
Deduct for breakage of pipe by contractor, and for minor supplies furnished by city..........................	989
Net amount of contract................	$80,785

The test of the 24-in. line, since relaying in new position, compares favorably with the original tests when the pipe was first laid, in 1898, the maximum leakage being 2,100 gals. per day.

The 28-in. pipe, completed in 1894, was built under the direction of the late Col. Isaac W. Smith, Chief Engineer of the Water Committee, Emery Oliver, being the Assistant Engineer in immediate charge of the work for the city. The flexible joints used for the 24-in. line, completed in 1898, were designed by the late J. A. Lesourd, mechanical engineer, of Portland, W. W. Amburn, M. Am. Soc. C. E., was the Assistant in charge of the dredging and pipe laying. The joints used for the 30-in. line, built in 1911, were made from plans prepared by W. R. Phillips, M. Am. Soc. C. E., also of Portland. The laying of the 30-in. line was under the direction of F. M. Randlett, Assoc. M. Am. Soc. C. E., Assistant Engineer, Water Department; and the taking up of the two pipes in 1913, and the relaying of one of them, was also under the direction of Mr. Randlett with B. S. Morrow, Assoc. M. Am. Soc. C. E., Assistant in immediate charge of the work.

The writer was also connected with the work described, first as Principal Assistant Engineer in charge of pipe lines for the Bull Run gravity supply, completed in 1894, and since 1897 as Engineer in charge of all construction operations for the Portland Water Department, now organized as the Bureau of Water, Department of Public Utilities, Will H. Daly, Commissioner in Charge.

Since the foregoing was prepared the writer has read the able and instructive article on Flexible Joints for Submerged Pipe Lines (ENGINEERING & CONTRACTING, April 15, 1914) by the late Emil Kuichling, M. Am. Soc. C. E., who made an exhaustive study of the subject, and by his presentation of the matter placed the profession under obligations. The data he gathered regarding the several types of flexible joints heretofore in use, and his conclusions drawn therefrom, form a valuable reference for all who may hereafter have occasion for investigation along similar lines.

Reduction in Virginia Pig Iron Rates to New England and Middle Atlantic States.—The Interstate Commerce Commission on July 9 ordered a reduction in pig-iron rates from Virginia to points in New England and the Middle Atlantic States. The decision was in the case of the Lowmoor Iron Company, et al., versus the Chesapeake & Ohio Railroad, et al. The commission finds that the pig-iron rates for Virginia furnaces to the districts named are unreasonably high as compared with rates quoted to producers of pig iron at competing points in Pennsylvania and New York. New tariffs were ordered in effect as of August 15 reducing the rate to Baltimore from $2.45 a ton to $2.25; the Philadelphia rate from $3 to $2.75; the New York rate from $3.95 to $3 and the Boston rate from $3.75 to 3.25. In the original petition the rates complained of were called excessive because of the large proportion demanded and received by the Pennsylvania Railroad. As the complainants alleged, a comparison of rates from different districts indicated that "generally speaking the rates from Virginia furnace to destinations substantially equally distant from furnaces in Pennsylvania and New York are from 30 to 50 per cent higher." The old rate from Virginia furnace to Boston, for example, figured out 5.8 mills per ton mile. From Pittsburgh to Boston it was 4 mills; from other western Pennsylvania points 4.1 and 4.2 mills and from Buffalo, 4.9 mills.

BRIDGES

Design, Construction and Detailed Costs of the Richelieu River Bridge, Lacolle Junction, Quebec.

(Staff Article.)

The renewal under traffic of the Richelieu River Bridge at Lacolle Junction, Quebec, by the Grand Trunk Railway possesses a number of interesting features. This bridge, which spans the Richelieu River, the northern outlet of Lake Champlain, originally consisted of a 180-ft. swing span (providing two clear channels of 73 ft. each) and pile trestle approaches, the east approach having a length of 350 ft. and the west one a length of 500 ft. The center pier and the rest piers of the old swing span consisted of timber cribs filled with rubble stone surrounding the supporting piles, the latter being capped with timber grillage which in turn supported concrete tops. The superstructure for the new bridge consists of one 250-ft. swing span and twelve 60-ft. plate girder spans. The substructure consists of thirteen piers, including the pivot pier, and two abutments. In addition to the construction of these piers and abutments the renewal of the bridge required the reconstruction of the old protection cribwork, the construction of wing protection cribs and booms, and the removal of the old protection works, rest piers and trestle. The work was completed in 1913.

Part I.

DETAILS OF DESIGN AND CONSTRUCTION.

General Features.—Figure 2 shows a longitudinal section of the bridge along the center line and a plan of the substructure and protection works. These drawings indicate the character of the work and illustrate the general features of the old and new bridges. It was necessary to construct ice breakers at short distances upstream, and to provide crib protection works for the rest and pivot piers.

Substructure.—The new pivot pier was built around the old pier and a new concrete top constructed, the superstructure being supported during construction on steel grillage beams. These beams had for their support the new concrete shell which surrounded the old pivot pier. The open caissons used in constructing the pivot pier and seven of the twelve remaining piers, including the two rest piers, had double walls consisting of 10 x 10-in. timbers, the two parts of each wall being separated by 12-in. vertical timbers resting on a heavy shoe. These caissons were sunk by filling the 12-in. space in each wall with concrete and by adding other loads. The caissons used for the other five piers were also of the open type, but they had single walls consisting of 10x10-in. timbers. These caissons were sunk by loading them with rails. The two abutments required timber coffer-dams.

The old pivot pier consisted of a timber crib 26 ft. square and 33 ft. high, filled with rubble stone, which surrounded the 108 piles. These piles were capped with a timber grillage, which was 3 ft. below low water and which supported a concrete top 8 ft. high and 20 ft. in diameter. A timber wall surrounded the concrete top, and the space between it and the crib was also filled with rubble stone. As this pier was considered too unstable for the loads which would be thrown upon it by the new bridge, it was reinforced in the following manner:

A double-wall caisson, 38 ft. square, outside dimensions, built up of 10x10-in. horizontal timbers and 12x12-in. vertical timbers between the walls at intervals. was sunk around the old pier, leaving a 3-ft. space between it and the old crib. After this space was filled up to approximately 6 ft. below low water with plain concrete, reinforced concrete walls were carried up to the required level to receive the eleven 26-in., 166-lb. I-beams, on which the swing span was erected and operated during the completion of the piers. Figure 1 gives a plan, a half section showing the caisson and the old pivot pier before alteration, and a half section of the completed pivot pier.

It was originally intended to remove the rubble stone filling from the old crib one pocket at a time, but this was found to be impracticable owing to the existence of fissures in the slate-rock foundation, which made unwatering impossible. The stone was, however, taken out to a level 2 ft. below the old timber grillage. The old piles, the timber grillage and the concrete top were left in place, except the upper 18 ins. of the latter, which were removed by blasting. Instead of unwatering the pier, water was pumped into it until a 3-ft. head was produced, this head being utilized in forcing a 1:2 grout into the voids of the rubble stone. After the voids were filled the water was pumped out, and the concrete work was completed in the dry, grillage beams being embedded in the coping to distribute the loads from the swing span.

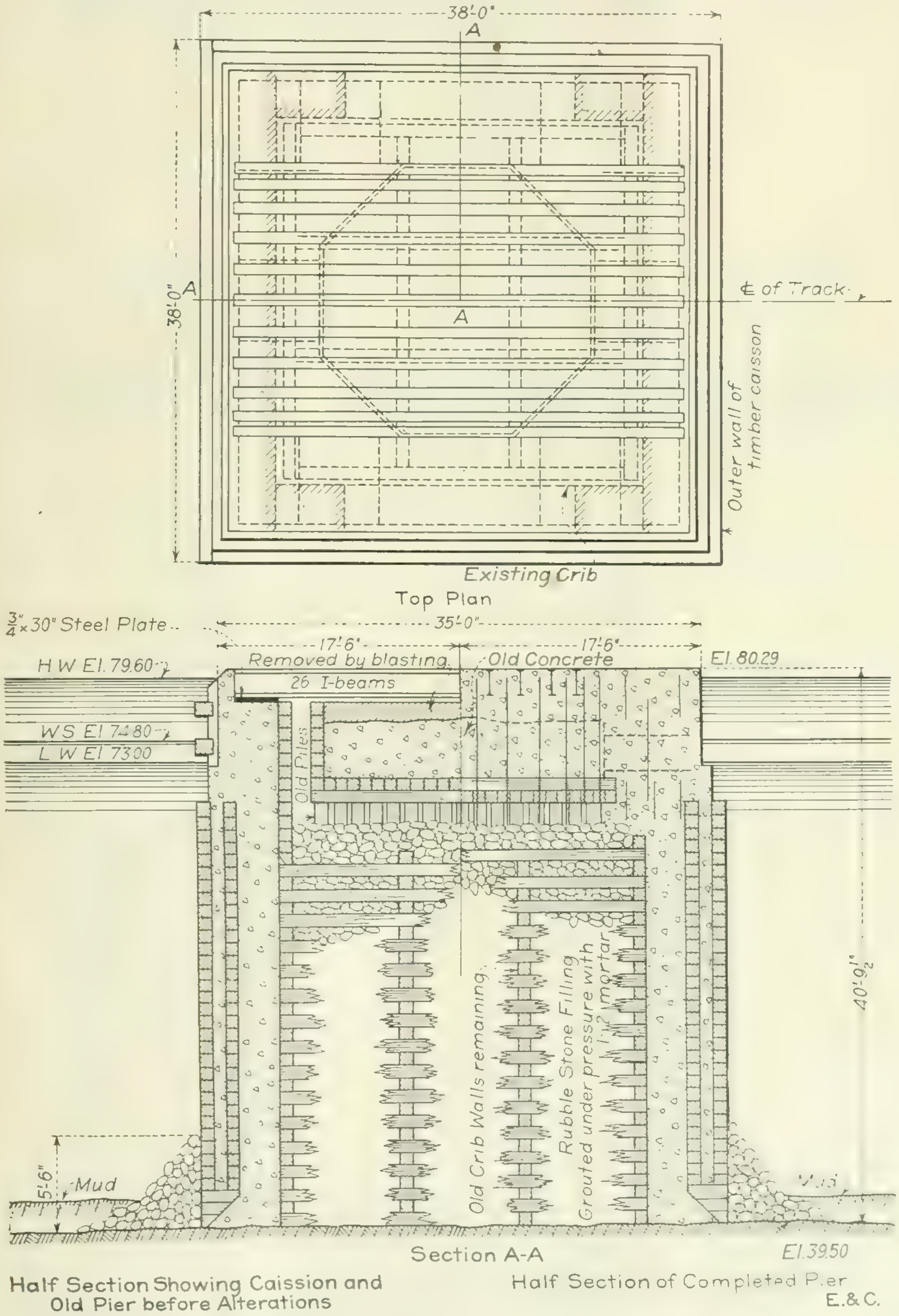


Fig. 1. Plan, Half Section of Old Pivot Pier and Half Section of Completed Pivot Pier of Richelieu River Bridge.

Seven of the intermediate piers (Nos. 4, 5, 6, 7, 9, 10 and 11, see Fig. 2) have caisson foundations, which are of similar construction to that used for the pivot pier and which differ only in shape. These piers are pointed, both on the upstream and downstream ends. Figure 3 (a) shows a half cross section and a half end elevation of a typical intermediate pier and caisson in which the double-wall type of caisson was used; Fig. 3 (b) shows a side elevation of the pier and caisson; and Fig. 3 (c) shows a plan of the caisson. Above elevation 70 (the top of the permanent caisson) the construction for all of these piers is alike. The respective heights of the seven piers, from foundation to base of rail, are: Pier No. 4, 29 ft.; pier No. 5, 34 ft. 6 ins.; pier No. 6, 37 ft.; pier No. 7, 39 ft.; pier No. 9, 41 ft. 6 ins.; pier No. 10, 39 ft. 6 ins.; and pier No. 11, 41 ft. The tops of these concrete piers have a length of 16 ft. and a width of 9 ft. From the top of each pier, at elevation 80.5, down to elevation 72.5 (which is 6 ins. below low water level) the downstream face is vertical, while the upstream face is beveled and is reinforced with a 15x½-in. plate. Below elevation 72.5 each pier has a width of 13 ft., and a length of 40 ft. 1⅞ in. Originally it was intended to make the length of the rest piers, Nos. 7 and 9, 36 ft. long and 10 ft. wide, as shown in Fig. 2, but it was finally decided to make the dimensions of all piers requiring double-wall caissons the same. The caissons for these piers have an outside width of 15 ft. and an outside length of 43 ft. The upstream nose forms an angle of 90°, while the downstream face makes an angle of 60°. The outside 10-in. timber walls are carried up to elevation 76.0, and the inside walls to elevation 70.0. After these piers were completed the outside walls were removed down to the 70.0.

The five single-wall caissons (Nos. 1, 2, 3, 12 and 13) are similar in shape to that shown in Fig. 3, the piers for which these caissons are used being located in comparatively shallow water. Figure 4 shows details of these piers and caissons. The elevation of the tops of these piers is 80.5, while the elevations of their bottoms are: Pier No. 1, 65.17; Pier No. 2, 64.34; Pier No. 3, 62.66; Pier No. 12, 61.83; Pier No. 13, 66.0. After these piers were built the caisson walls were removed down to elevation 68.5.

The caisson shoes were constructed on land and were launched from a skidway. Figure 5 is a view showing the launching of the shoe for an intermediate pier. The caissons were built up in place, as shown in Fig. 6, which shows a double-wall caisson under construction.

Figure 7 (a) shows a plan of the east abutment; Fig. 7 (b) shows a longitudinal section along the center line of the abutment; and Fig. 7 (c) shows a cross section of a wing wall.

In general the piers rest on slate rock, hardpan or compact gravel, except piers Nos. 12 and 13 and the east abutment, which required pile foundations. The compact material under pier No. 11 was overlaid with about 7 ft. of loose material, which was removed by an orange-peel bucket. Before the piles were driven for piers Nos. 12 and 13 about 5 ft. of the top soil was removed. Before placing concrete, a diver leveled off the foundation for each pier and also for the protection works. In addition to this work the diver, who was employed continuously on the job, assisted in landing the caissons, in blasting boulders from the cutting edge, and in blasting away the old crib protection works.

After the caissons reached bottom they were underpinned with burlap bags of concrete and were then filled with concrete, which was deposited by bottom-dump buckets. The water was then pumped from the caisson and the concreting continued in the dry.

The rest piers and the pivot piers were started after navigation had closed, and the work was sufficiently advanced to permit the swing span to be erected in time for the opening of navigation. Some severe weather was encountered, and the temperature was as low as 28° below zero when the upper part of the pivot pier was concreted.

All of the protection piles and cribs of the old bridge required replacing, the new work consisting of six cribs (for location see Fig. 2b), built of 10x10-in. timbers and loaded with rubble stone. The three cribs near each rest pier are connected and are joined to the rest pier by floating booms. These booms consist of 12x12-in. vertical timbers bolted to the cribs and rest piers. The old center protection work below low water was left in place, and, after being strengthened by the addition of five new cribs, a new top, consisting of a double row of walings, was constructed.

Superstructure.—The bridge, which is a single-track structure, was designed for Cooper's E-50 loading. The unit stresses used are those given in the 1910 specifications of the Grand Trunk Ry., the impact allowances being those given in the Dominion Government's 1908 specifications. The twelve deck plate girder spans possess no unusual features.

The swing span has a length, center to center of end floorbeams, of 244 ft. 7½ ins., and a center height, center to center of chords, of 36 ft. 0 in. It is of the center-bearing type, the ends being raised and lowered by wedges operated by hand power. The trusses of the swing span are spaced 18 ft. on centers, each

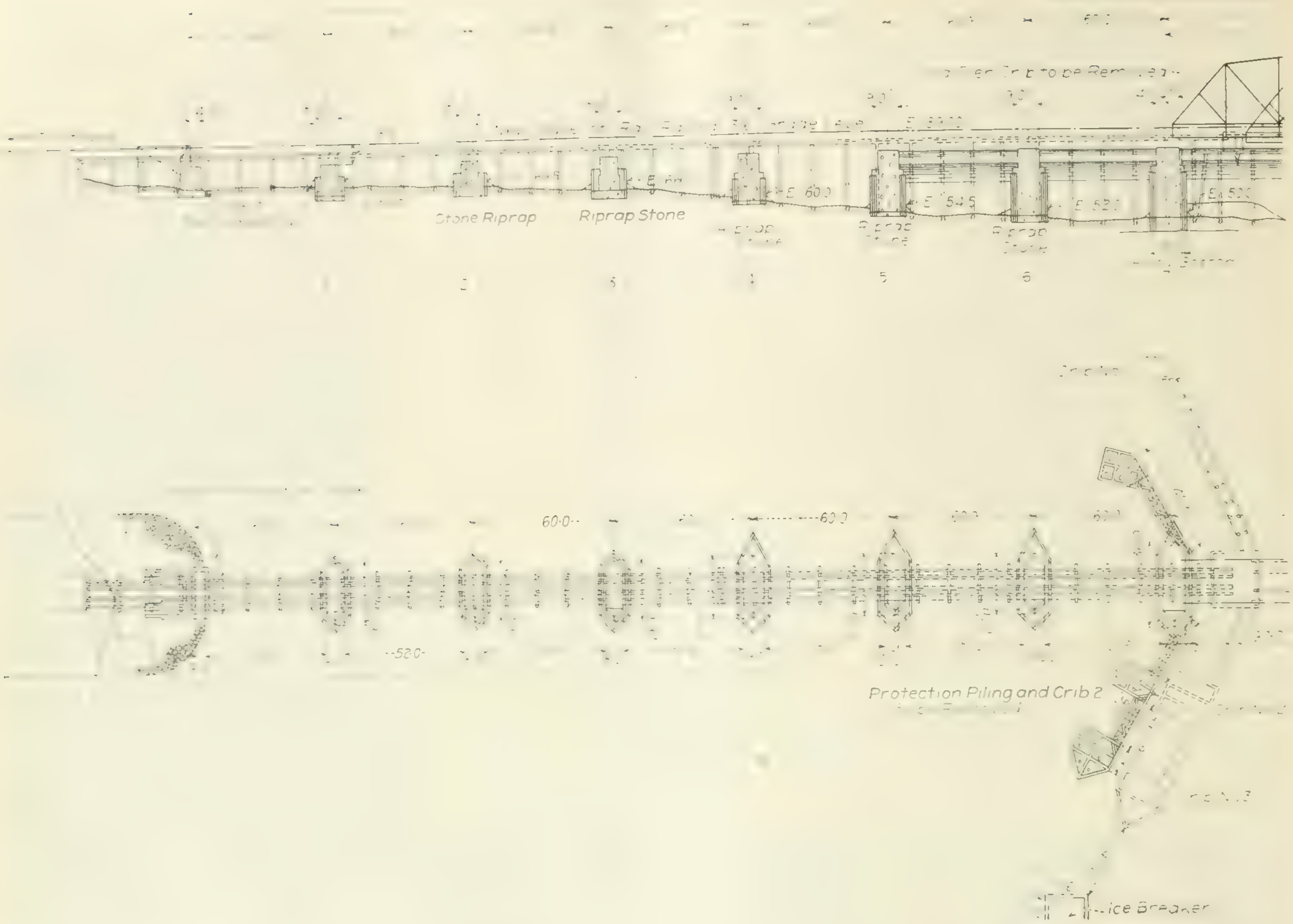


Fig. 2. Plan and Longitudinal Section Along Center Line of Richelieu River Bridge, Lacolle Junction,

truss consisting of eight 28 ft 6-in panels and a center panel having a length of 16 ft. 7½ ins. The distance from top of pivot pier to base of rail is 11 ft. 0 in.

Instead of using concentrated wheel loads in computing the stresses in the swing span equivalent uniform loads were used. The equivalent uniform load used for one arm was 6,400 lbs. per linear foot, and that for the entire span, 5,700 lbs. per linear foot. The dead load assumed for this span was: floor, 735 lbs. and steel, 2,365 lbs., a total of 3,100 lbs. per linear foot. The following cases were considered in determining the stresses in the trusses:

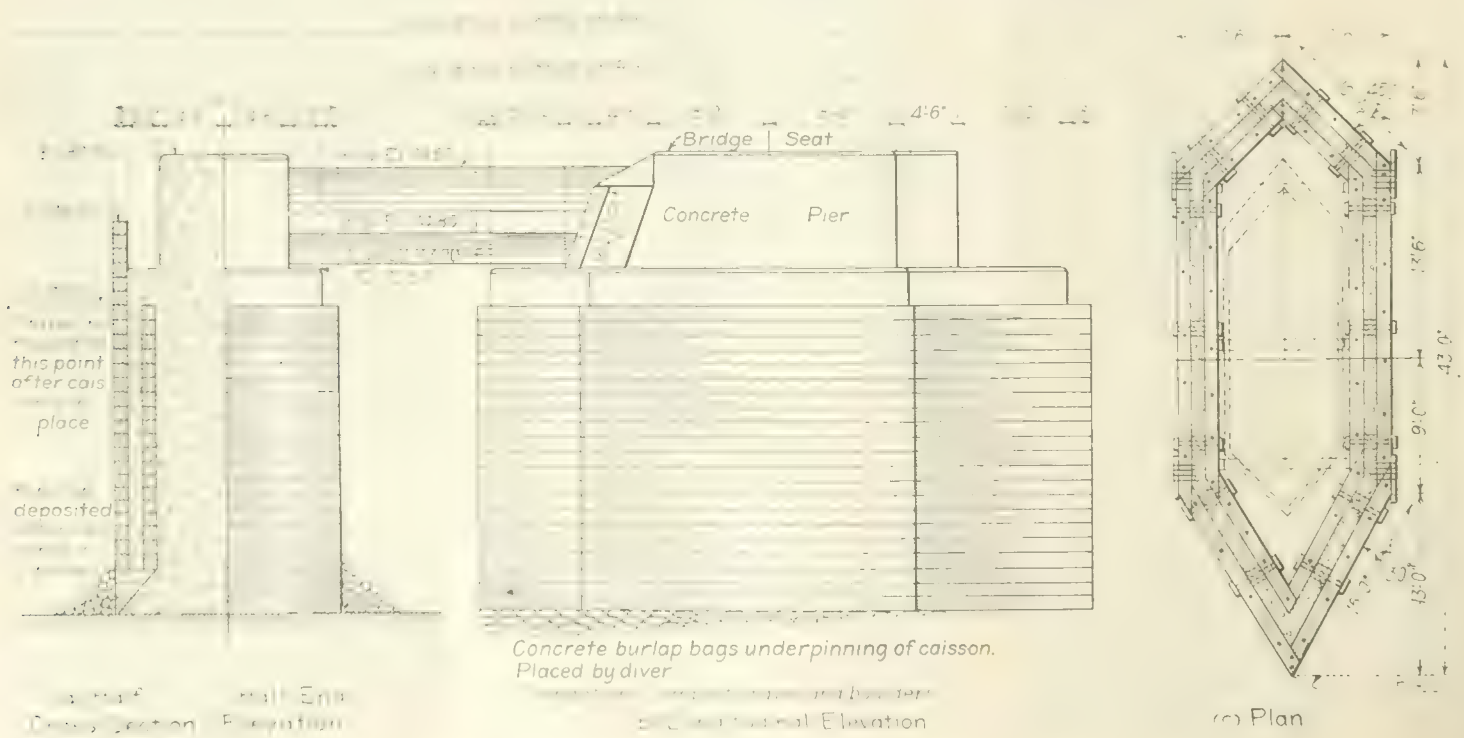
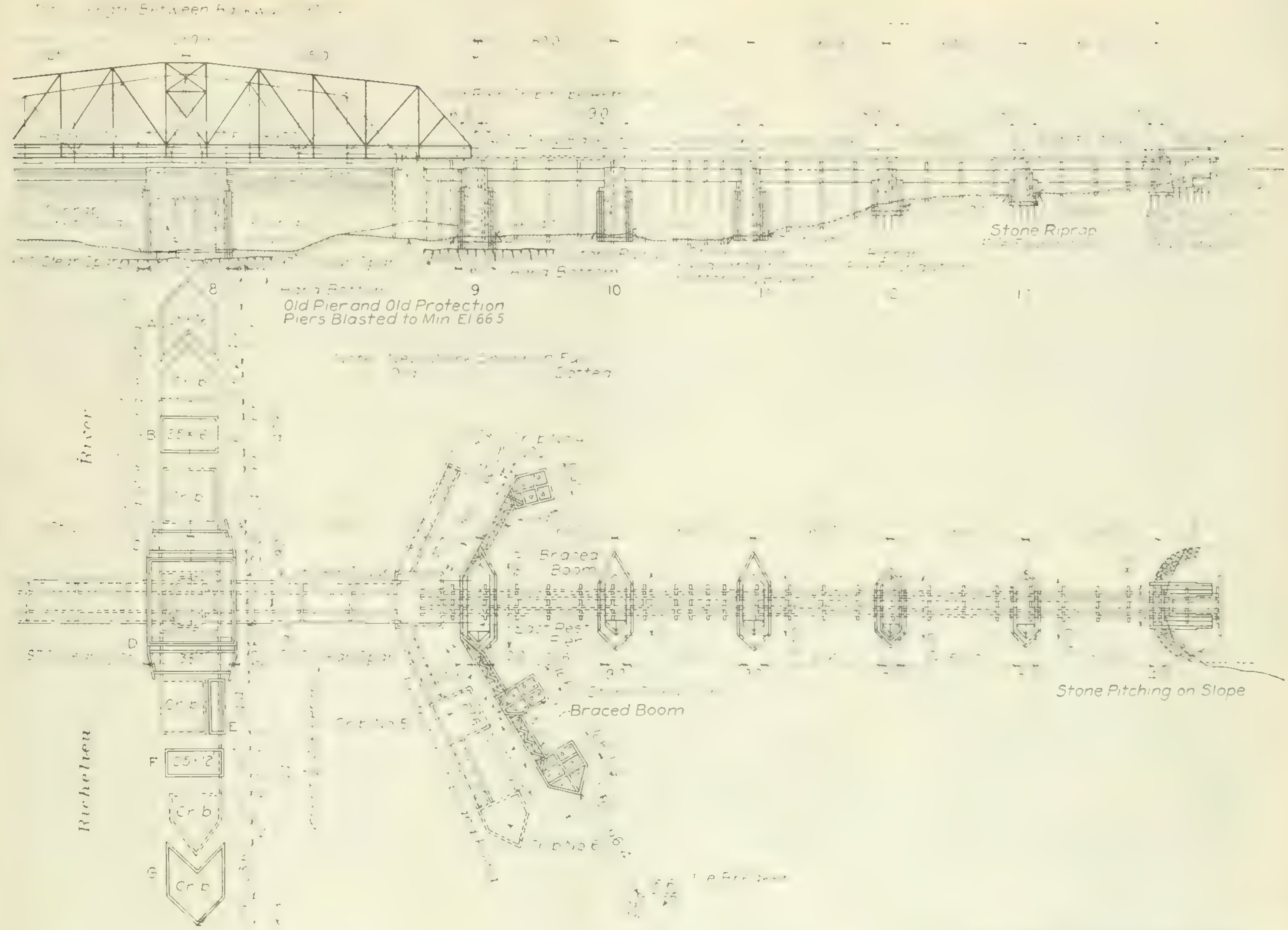


Fig. 3. Details of Typical Intermediate Pier and Double-Wall Caisson of Richelieu River Bridge.



Quebec, Showing New and Old Construction—New Plate Girders Not Shown.

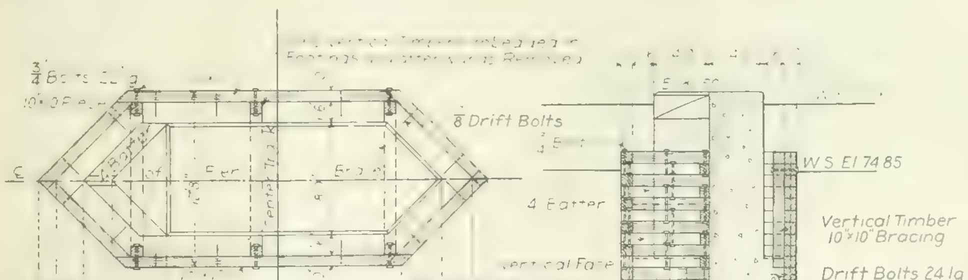


Fig. 4. Details of Pier and Single Wall Caisson Used In Shallow Water—Richelieu Bridge.

Case I.—Dead load, bridge as cantilever.
Case II.—Dead load, bridge on three supports.
Case III (a).—Live load, bridge on three supports, loads at b. c, and d.
Case III (b).—Live load, bridge on three supports, loads at c and d.
Case III (c).—Live load, bridge on three supports, load at d.
Case III (d).—Live load, bridge on three supports, loads at b and c.
Case III (e).—Live load, bridge on three supports, load at b.
Case IV.—Live load, bridge on three supports, fully loaded.
Case V.—Live load, bridge as simple span, one arm on two supports.

Figure 8 shows an outline of the trusses of the swing span, and gives the stresses in the truss numbers, and their composition.

The circular girder, which has a diameter of 18 ft., consists of four 6x4x⅝-in. angles and a 24x¾-in. web. The wheels are 12 ins. in diameter and 8 ins. wide. The center bearing steel casting is the Grand Trunk Ry.'s patent No. [illegible].

Fig. 5. View Showing Launching of Shoe of Caisson for Intermediate Pier—Richelieu River Bridge.

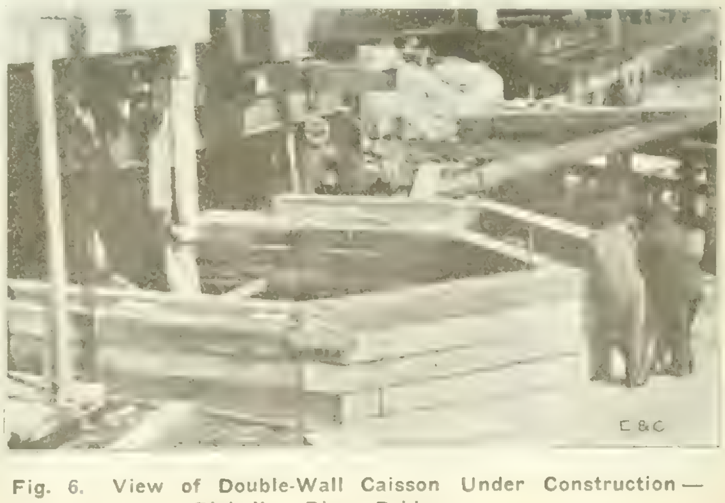
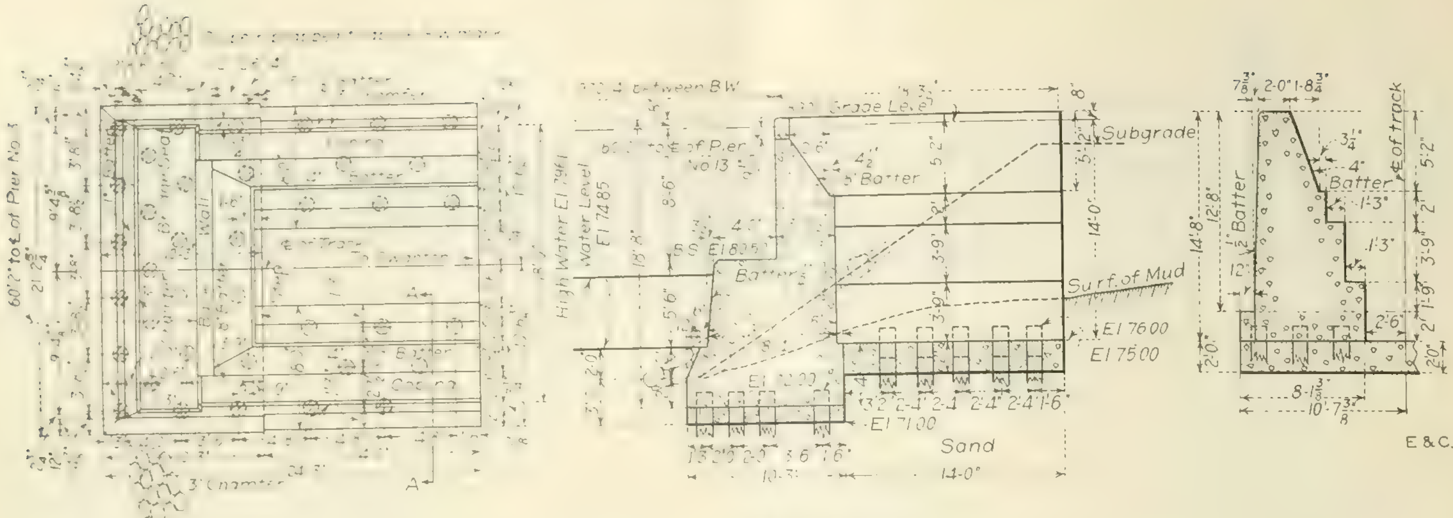


Fig. 6. View of Double-Wall Caisson Under Construction—Richelieu River Bridge.



(a) Plan (b) Section on ℄ (c) Section A-A

Fig. 7. Plan, Longitudinal Section, and Cross Section of Wing Wall of East Abutment of Richelieu River Bridge.

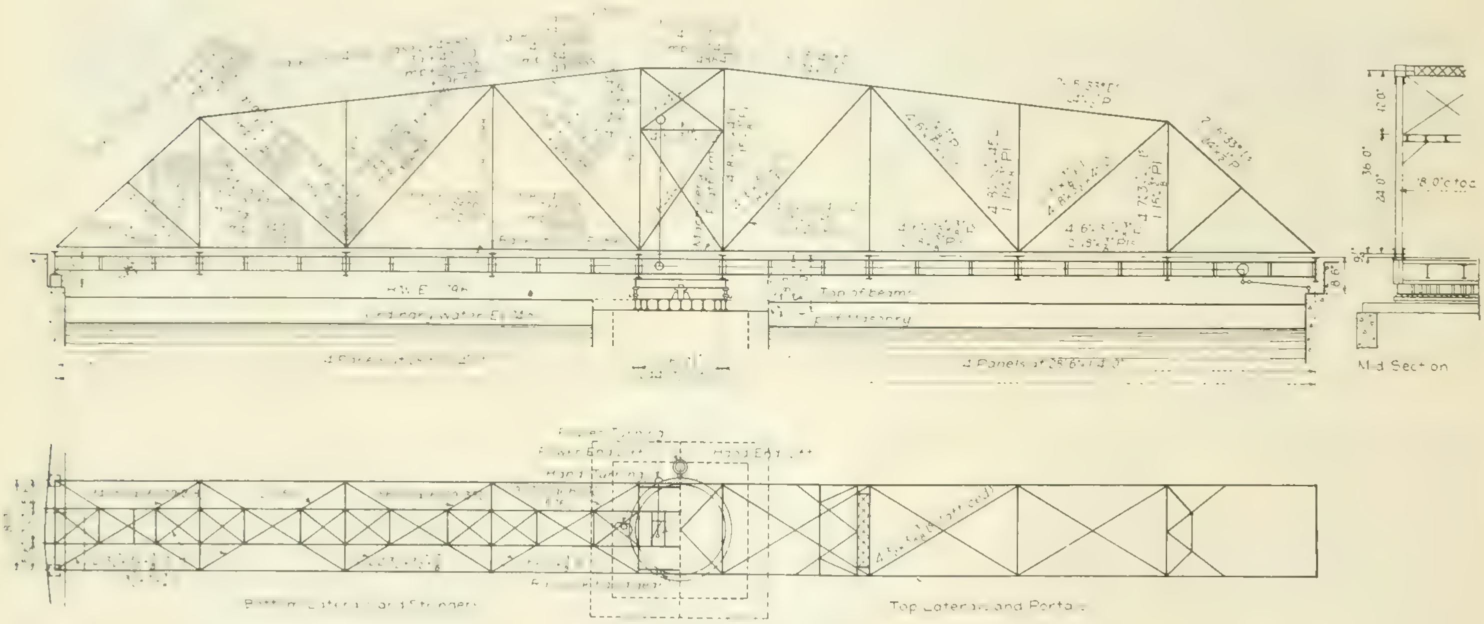


Fig. 8. Stresses and Composition of Truss Members of New Swing Span of Richelieu River Bridge.

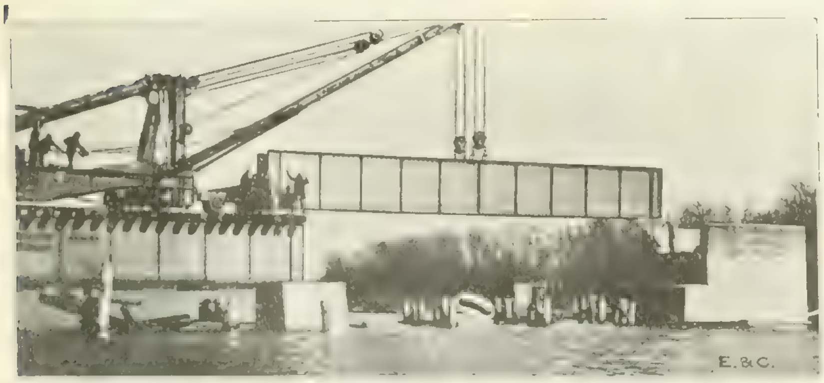


Fig. 9. View Showing Manner of Erecting 60-ft. Plate Girder Spans of Richelieu River Bridge—Note Type of Hooks Used.

Fig. 10. View of Swing Span, Protection Works and Adjacent Girder Spans of Completed Richelieu River Bridge.

The 60-ft. plate girder spans were erected, completely riveted, by means of a derrick car. Figure 9 shows the manner of erecting these spans. This view shows the front of the derrick car and the double hooks and blocks used. The twelve girder spans were erected in 5½ days, the traffic having been diverted during their erection.

Figure 10 is a view of the central portion of the completed bridge showing the swing span, the protection works, and an adjacent plate girder span.

Piling Used in Honolulu Harbor—Owing to the destructiveness of the marine life in the harbor of Honolulu, concrete has been resorted to in recent years as a substitute for timber piling. Some Douglas fir piling is still being used, in 70 and 80-ft. lengths, but it is necessary to sheath the piles with copper to protect them from the teredo. Piles when bought on the Pacific coast reach the islands as part of a deck load of lumber. With the opening of the Panama Canal an impetus is expected to be given to the shipping business of Honolulu, and the question has been agitated of enlarging the port facilities through the improvement of Kalihi Harbor and Channel, which lies between Honolulu Harbor and Pearl Harbor. Should this scheme materialize, a decided demand for lumber would follow for dock purposes.

Exports of Philippine hardwoods to the markets of the Orient and the United States are steadily increasing, and the demands are much greater than the current production can fill.

CONSTRUCTION PLANT

MACHINES DEVICES MATERIALS

The Largest Electric Dragline Excavator.

The largest electric dragline excavator yet made, which also carries the longest boom, is now being employed by Mr. James O. Heyworth of Chicago in excavating on Section 11 of the Calumet-Sag Canal at Blue Island, Ill. The machine itself, exclusive of the track sections, weighs in the neighborhood of 225 tons. On time tests it was able to make a complete digging cycle in slightly under 50 seconds with the bucket filled to overflowing and holding from 3½ to 4 cu. yds. This rate cannot be maintained continuously, of course, as there are numerous delays for moving track, trimming slopes, throwing the bucket out to reach material on the far side of the ditch, etc., which lowers the machine output somewhat. On the Heyworth contract this machine is working two 10-hour shifts per day. On the night shift preceding the day of the writer's visit the machine made 580 swings in 10 hours. The accompanying view shows this machine at work on the Calumet-Sag Canal.

The turntable is 24 ft. in diameter, the boom is 125 ft. long and the nominal capacity of the bucket is 3½ cu. yds., though when heaping full it holds fully 4 cu. yds. The hoist motor is 250 and the swing motor 135 HP., respectively. The sub-base, revolving frame, boom and A-frame are of structural steel. The base is of extremely heavy design and is built entirely of structural steel. All its principal members are reinforced by heavy cover and gusset plates. The base carries on its bottom supports for the propelling mechanism, and center castings for the trucks and equalizing beam.

The trucks are of the 4-wheel type and are exceptionally heavy. The wheels are double flanged. Two driving trucks support the base directly. All four wheels of these trucks are drive wheels. The other two trucks are also drivers and are connected by means of an equalizing beam extending the full width of the base. This beam insures an even load distribution on the two front trucks and on the two rear trucks when standing on uneven ground. The beam carries a jackscrew at each end; these are over the center of the trucks. These screws when set bear against plates on the bottom of the base and transmit the load directly from base to trucks. When the machine is being moved the jackscrews are released and the trucks are free to adjust themselves to irregularities in the track. Before starting to dig these jackscrews are set to relieve the equalizing beam from excessive loads when the machine is operating. Each truck is so designed that all wheels are bearing on the track, even when the track is uneven—that is, one axle is pivoted on the center.

The turntable consists of 40 open-hearth steel rollers revolving between two 90-lb. rail circles, 24 ft. in diameter, one attached to the bottom of the revolving frame and one to the top of the base. The revolving frame, of very heavy construction, consists of four longitudinal beams connected by transverse beams and separators. All the heavy forces at the front end of the revolving frame, caused by the boom foot reactions, the reaction from the front gantry and from the swing pinion are carried directly into the heavy, annealed, steel casting base plate, and then through the turntable rollers to the heavy sub-base.

Power for driving the trucks is transmitted through a vertical shaft passing through the main center castings of the machine and geared directly by means of a double reduction to the main hoisting shaft. A jaw clutch is provided on this shaft for disengaging the entire propelling mechanism. The vertical shaft drives a horizontal transverse shaft beneath the base. The latter shaft carries at each end a double chain sprocket with a jaw coupling, thus permitting the driving of all four trucks or the independent driving of any two side trucks. Heavy propelling chains connect these sprockets with two large split driving sprockets located on intermediate shafts of the driving trucks. Before beginning to dig, after moving, four 90-lb. Bates rail clamps, with wedges, are attached to the rails so as to bear against the trucks, to hold the machine firmly in position during operation. Dirt and iron punchings are used on the machine as ballast or counterweight.

The design of the long boom is rather unusual, inasmuch as it is only 3 ft. wide for its full length. It is held in place by very heavy guy cables. This construction combines lightness with strength. This reduces the amount of counterweight and the power required to swing the machine.

Another special feature in this machine is the automatic electrical control. The main motor, which operates the dragline and hoisting line, is under full automatic control and the operator can push his control lever directly to full speed position without injury to the motor, which will be given current as fast as it can take care of it. The swing motor is similar and the acceleration is entirely automatic. The control handle can be thrown from full speed forward to full speed reverse and the motor will only be stopped and started in the opposite direction by an automatically regulated amount of current. This method is regularly used to stop the swinging of the machine, although a hand-operated brake is also provided in case of emergency.

The dumping arrangement of the bucket is such that it may be picked up in a carrying position directly under the point of the boom, or even beyond. This means that when the bucket fills quickly after being lowered straight down or thrown out beyond the end of the boom it can be picked up at once and no delay nor waste of power is necessary to haul it in some distance before picking it up.

The track sections on which the machine is operating were furnished by Mr. Heyworth to his own designs. They are built up of heavy structural steel shapes. The track sections are made in the form of partial sectors of a circle. The chord of the outside arc is 12 ft. and that of the inside arc is 8 ft. long, respectively. The sections are 30 ft. wide. The rails are mounted on I-beam ties which rest on parallel pairs of I-beam stringers. The stringers rest on 4x6-in. timbers which lie on the ground. In going around curves the sections are assembled long side to long side, but in working on a straight section of the canal the sections are jointed so that the 8-ft. side of one abuts against the 12-ft. side of another. This causes the machine to travel along a line which approximates a straight line parallel to the canal center line. The rails are curved, of course, but in the short sections used the variation from tangent is small when they are assembled for straight portions of the work. The sections are held together by means of keyed pins. Each section weighs 2½ tons.

Each section has a main diagonal composed of four angles, lattice connected. The stringers under the two tracks are held together by heavy I-beams which extend along the two radii at the ends of the section. To the centers of these I-beams, on their inner surfaces, are riveted bent plates in which are punched holes for the attachment of the lifting hooks. A section is moved from one end of the track to the other by means of a bridle chain which hooks into the two bent plates described and to any convenient third point on the section. This chain is then attached to the dragline bucket and the section is swung into its new position.

Current is obtained from the Public Service Commission of Northern Illinois. The voltage to the transformer wagon is 2,000. Here it is stepped down to 440. The current is 3-phase, 60-cycle. The transformer wagon is a very heavy eight-wheeled truck formed of two 4-wheel sections coupled together. The wagon is moved forward by means of a cable which is attached to a hook on the base of the dragline machine.

The electric dragline excavator here described was manufactured by Bucyrus Co., South Milwaukee, Wis. It is a special machine, but is known as type 175 B.

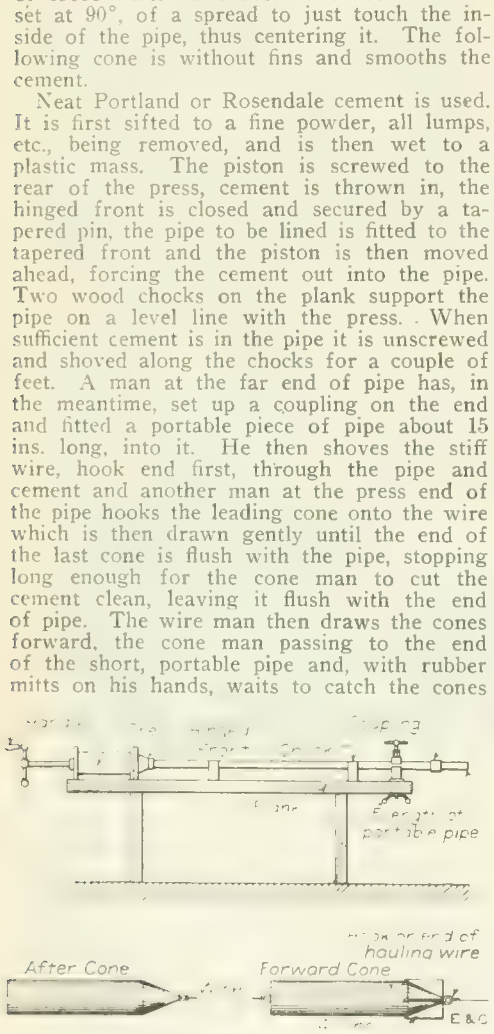
View of Largest Electric Dragline Excavator, at Work on Calumet-Sag Canal, Blue Island, Illinois.

Method of Lining Wrought, Steel and Cast Iron Pipe with Cement.

For over 40 years water pipe has been lined with cement in New England. This work was ordinarily done, heretofore, by water department employes. The Macbee Cement Lined Pipe Co. of Boston, Mass., is now equipped to line wrought iron, steel and cast iron pipes with cement. The first two types of pipe are lined as a protection against corrosion and cast iron pipe is lined to prevent the formation of tubercules. The process of cement lining is not patented, but requires some skill, especially in mixing the cement to the proper consistency. This varies, it is stated, with the size of pipe to be lined, with the length of pipe and with the temperature of the air. The Macbee methods and apparatus for placing the cement lining are here illustrated and described.

In general the pipe is filled two-thirds full with soft cement and bored out while the cement is still soft. A press about 5 ins. in diameter and 18 ins. long, shown in the accompanying cut, equipped with a piston on a threaded rod, has its front end tapered and hinged to swing open to one side. The front of this tapered end is tapped for a 2-in. pipe (other sizes are fitted to a bushing). The press is secured by bolts to one end of a plank about 2 ins. x 10 ins. x 18 ft. long. This plank is on supports 3 ft. high and at the far end a pipe vise is sometimes fitted. The vise is to hold the pipe firm, although it may be held by hand. A stiff wire, longer than the pipe, with a hook on one end and two cones are also provided. These cones are in a set of two and sets are made of different diameters to suit the sizes of pipe, always being ¼ in. less than inside of the pipe to be lined, for wrought iron and steel pipe up to 3 ins. and should be ½ in. smaller diameter for cast iron pipe of larger sizes. The cones are pointed at the forward ends, and are joined together by a chain or loose wire. The first cone has four fins joined together by a chain or loose wire. The first cone has four fins set at 90°, of a spread to just touch the inside of the pipe, thus centering it. The following cone is without fins and smooths the cement.

Neat Portland or Rosendale cement is used. It is first sifted to a fine powder, all lumps, etc., being removed, and is then wet to a plastic mass. The piston is screwed to the rear of the press, cement is thrown in, the hinged front is closed and secured by a tapered pin, the pipe to be lined is fitted to the tapered front and the piston is then moved ahead, forcing the cement out into the pipe. Two wood chocks on the plank support the pipe on a level line with the press. When sufficient cement is in the pipe it is unscrewed and shoved along the chocks for a couple of feet. A man at the far end of pipe has, in the meantime, set up a coupling on the end and fitted a portable piece of pipe about 15 ins. long, into it. He then shoves the stiff wire, hook end first, through the pipe and cement and another man at the press end of the pipe hooks the leading cone onto the wire which is then drawn gently until the end of the last cone is flush with the pipe, stopping long enough for the cone man to cut the cement clean, leaving it flush with the end of pipe. The wire man then draws the cones forward, the cone man passing to the end of the short, portable pipe and, with rubber mitts on his hands, waits to catch the cones



Sketch of Elements of Macbee Apparatus Lining Wrought Steel and Cast Iron with Cement.

as they come out. A box on the floor catches the excess cement, which is forced out ahead of the cones.

After the cones have bored the cement the portable pipe is then unscrewed, leaving about 5 threads clear for the next pipe when laying in the trench. The cones are then put into a pail of water to wash off any sticky cement and the pipe is examined. Some operators claim that daylight is enough for this, but two portable electric lights are preferable, since unless the men look through the pipe alternately while one holds a light, the rough places, cone marks, tearing off of the cement, etc., can not be easily detected. The pipe should be in 16-ft. lengths, but it is possible to make a good job on a 22-ft. length of 1-in. pipe. Lining that does not show smooth from both ends of the pipe is run through again with the cones.

It is stated that pipe may be shipped one week after lining without danger from breaking of the cement, although this time may be shortened or lengthened according to the location. The pipe must have plenty of air, a draft is good (the ends of course set up first), and a warm sun to heat the pipe will often hurry the set. Some operators wet the inside of pipe before lining, but it is not necessary, as a good bond is obtained without doing so. Care is taken to avoid much oil in the pipe (used when cutting the threads). Pipe may be cement lined when not absolutely straight, but it is better to straighten it before working. Pipe should be well supported, generally on the floor, to keep straight when the cement is setting. It is claimed that one barrel of Rosendale cement will line about 1,000 ft. of 1-pipe. The Company has lined 800 ft. of 1-in. pipe with one man to help, in a little less than 4 hours or better than 1,600 ft., in an 8-hour day; but this is said to be fast work. Generally 1,200 ft. to 1,400 ft. per day is good for two men.

Galvanized pipe is lined as well as plain pipe. All pipe to be cement lined must first be "plugged"; that is, selected for clear inside surface. Many times the weld in wrought iron pipe will have small pieces projecting inside which catch on the cones, making a clear bore impossible. This "plugging" is often done by pulling a 1-in. diameter steel ball, pierced by a wire, turned into an eye at each side, to which is attached a long wire through the pipe. If the ball passes through the pipe without catching the surface it is all right, otherwise it is discarded and another pipe is tested in its turn. Some operators keep a steel die, mounted on a small pipe, to force in and break off these bits of metal if there are only a few, rather than to return the pipe to the mill.

Continuity of surface is obtained by carrying the cement out into the coupling, as already described, and by lining tees, elbows, etc., having the bore same as the pipe, a smooth, even surface for flow of the water is secured. The cement in the fittings being kept back for a few clear threads to engage the pipe end. Although neat Rosendale cement is advocated for the pipe lining, Portland is better for the fittings, as the body of material is so small in some that strength is required to maintain a proper hold.

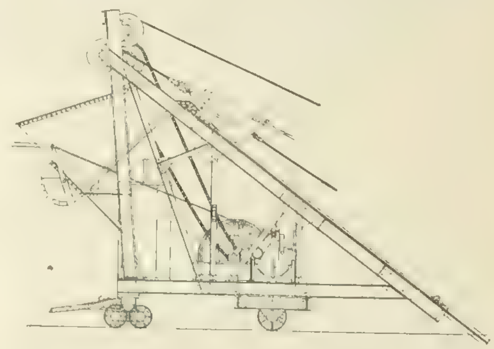
Cement lined pipe is cut by first marking to a depth of 1/16 in. with a roll cutter and finishing with a hack saw. The resulting end is said to be square and smooth, metal and cement being flush. It is claimed that small-size cement lined pipe can be bent sufficiently to clear ordinary obstructions without an appreciable breaking down of the lining. It is also claimed that 1-in. pipe when cement lined will freeze solid without splitting the wall of the pipe.

Cement lined cast iron pipe is a new application of an old method. Tests made on "undipped" pipe of 6, 8 and 10-in. sizes, with a cement lining ¼ in. thick, indicate that the lining adhers close up to the fracture when the pipe is cracked and broken. Such pipe is readily tapped for service connections. When lining bell and spigot cast iron pipe the cement is carried about ⅜ in. into the bell to give a continuous cement surface when the spigot end is fitted. The joint is caulked in the usual manner. Cast pipe is lined with one cone having a tapered after end. The centering fins are carried back of the center of gravity to avoid tipping.

Drag Scraper Excavator and Wagon Loader.

A machine used for excavating and loading sand and gravel from dry pit, excavating for foundations, digging ditches and sewers where fairly dry conditions obtain, and other purposes where materials are to be excavated, carried for some distance and automatically elevated and loaded is shown in the accompanying illustration.

The machine consists of an inclined runway mounted on a truck, together with a drag scraper. The scraper is hauled back into the excavation 100 to 500 ft. for its load, which it carries up the runway of the incline and dumps automatically into a hopper at the top to be loaded into wagons. The scraper is dragged by a continuous drag line running over a pulley at the top of the machine and another pulley anchored at any convenient point in the excavation. Power is supplied by a gasoline engine, or an electric motor. The hopper from which the wagons are loaded have a capacity of 1½ cu. yds. and the gate, placed 6 ft. above the ground, may be operated by the engineman. A 125-gal. water tank is mounted on the truck. The front wheels under the machine are in pairs, permitting the



Drag Scraper Excavator and Wagon Loader.

easy rotation of the apparatus to load from any position of the excavation.

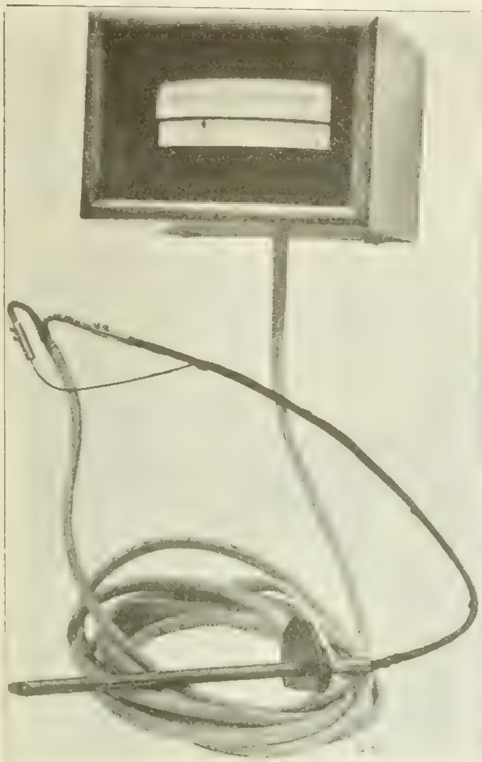
The outfit is manufactured in three sizes of 6, 10 and 20 cu. ft. scraper capacity and has a rated output at 100-ft. haul of 15, 25 and 40 cu. yds. per hour for each size, respectively. The maximum heights vary from 14 ft. to 16 ft.; length from 20 ft. to 22 ft.; widths from 5½ ft. to 8 ft.; and shipping weights from 7,000 lbs. to 12,100 lbs. Engines vary from 10 to 25 hp. Scraper speed may be varied from 150 to 350 ft. per minute. Under ordinary conditions a cost of 4 cts. per cubic yard for excavating and loading is claimed for their machines.

The apparatus is manufactured and sold by the Insley Mfg. Co., Indianapolis, Ind.

A Recording Temperature Device for Bituminous Paving Work.

In many of the larger cities it has been imperative that the contractors equip their plants with recording temperature devices, or the city will not accept their asphalt. If it is of value to the city, it is of much more value to the contractor to know that the asphalt is going through right, because he is the loser, no matter which way things break. He may get some asphalt past the inspector, but if the road proves faulty, as it will with improper asphalt, he has the work of tearing up the road and relaying it. If the inspector refuses the load, his loss is the certain one, and while not amounting to as much as replacing the road, still it is a considerable one, and one which can be avoided.

The indicating pyrometer, the details of which are clearly shown in the illustration, consists of a thermo-couple which is inserted in the material the temperature of which it is desired to maintain or determine continuously and a recording device with an indicating hand. The temperature of sand or asphalt is determined equally well with this instrument. In detail it may be explained that the thermo-couple is a steel tube containing an insulated wire of a different composition. The junction of any two unlike metals, if exposed to heat, will generate a current of electricity proportional to the temperature; not a current which can be felt, like a 1,000-volt charge, but a very light current, which must be measured by a millivoltmeter, as the millivolts generated seldom ever exceed 60. By building a sensitive millivoltmeter, and graduating it to read directly in temperature values rather than millivolts, it is apparent that a very simple temperature measuring device has been obtained, one that is robust, so far as the thermocouple is concerned, yet very quick in action, more so than a mercurial thermometer. The thermocouple and the indicator are connected by copper wires, and the distance between the two is immaterial,



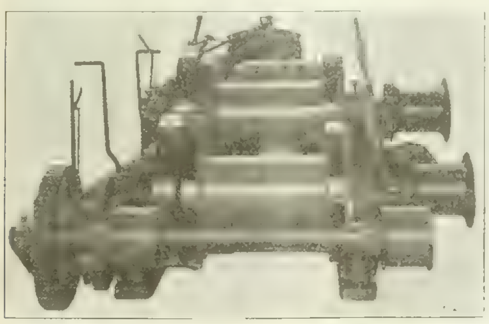
Continuous Indicating Device for Determining the Temperature of Bitumen and Sand.

so that the indicator may be placed where it is most convenient for the person responsible for the sand or stone. Of course, all plants are not arranged alike, and in some instances the thermocouple is at one end, the indicator at the other, while in other cases they are at the same end of the dryer. The cost of such an outfit, for a single unit, is $46.00.

The present method employed by those who have not adopted pyrometers is to dip out a cupful of sand and insert a thermometer and secure a reading in that way. This is crude, requires time and, it is claimed, is not as accurate as the pyrometer method. A somewhat similar method is used for bitumen. The pyrometer is manufactured by the Thwing Instrument Co., 456 N. Fifth St., Philadelphia, Pa.

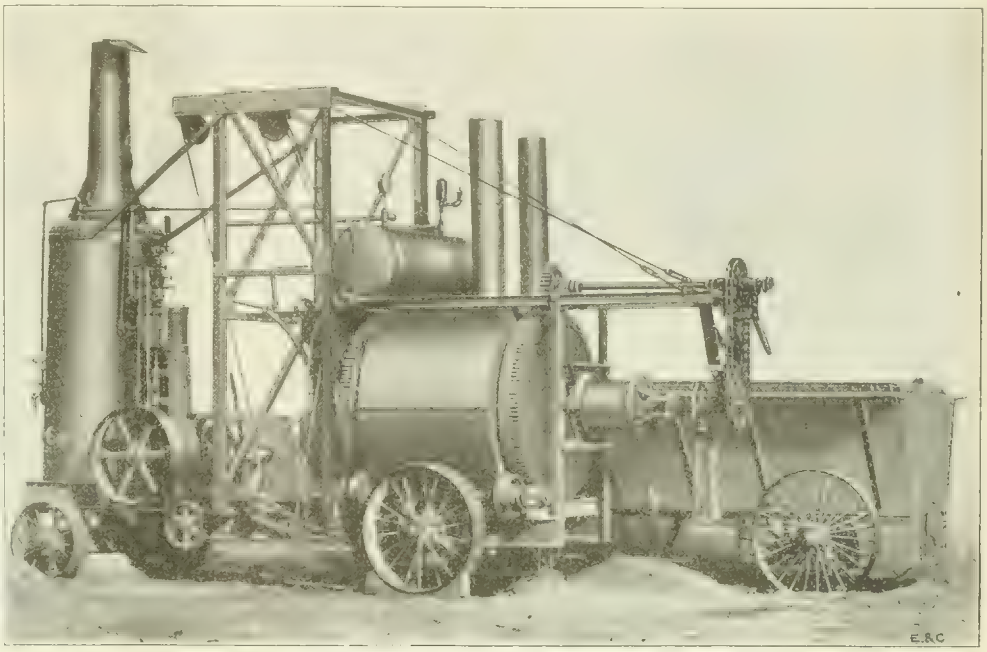
Electric Hoist Designed for Heavy Service.

The accompanying illustration shows the type of electric hoist which is to be used by



Electric Hoist for Heavy Service.

Post & McCord, New York, in the construction of the new Rapid Transit System in Brooklyn. These hoists are of the band friction type, with niggerheads of the clutch design, 100-HP. motors, 2-phase, 60-cycle, 200-volt controllers of the graphite carbon disc type, and steel swingers throughout. The swingers are of the two-drum type and are so designed as not to interfere with the lines which lead to the drums. The two-speed device permits a shift from low to high speed, or vice versa, without stopping the rotation of the motor or drum under load. Due to the severity of service on which these hoists are to be used the brakes are located on the opposite side of the drum from the friction, so that the heat generated by slipping the brakes cannot affect the efficiency of the friction.



Small Portable Plant for Preparing Bituminous Concrete.

The boom swinger is equipped with an automatic lock which prevents the boom from creeping with the wind strain when the control lever is in the neutral position. The hoists are furnished by the Thomas Elevator Co., 20-22 S. Hoyne Ave., Chicago, Ill.

A Machine for Preparing Bituminous Concrete for Country Road Construction.

(Contributed.)

The accompanying illustration shows a new combination concrete and hot mixing machine designed and built by Hetherington & Berner Co. of Indianapolis, Ind., and intended to be placed upon the market in the season of 1915 along with the 1915 models of asphalt paving plants designed by that company. This machine may be used for mixing ordinary concrete or, without any material alteration, may be used effectively for the manufacture of bituminous concrete or sheet asphalt mixtures for road building, or for street or road repair work.

The principle involved in the design of this mixer is that common to all hot mixing and concrete mixing machines. In making mixtures for bituminous concrete paving work the mineral ingredients are first charged into the revolving drum by means of a skip bucket operating at one side of the machine, in the usual manner. A hot blast caused by the consumption of common fuel oil is then introduced into the drum and the minerals are heated to the desired temperature; the asphaltic material, which has been previously melted and made ready in a separate melting kettle which stands alongside the mixer, is then introduced into the drum and the whole mass is thoroughly intermingled by means of a system of shovels or mixing blades attached to the interior of the drum shell. When the mixture has been completed it is automatically discharged into the waiting carts or barrows and is taken to the street, spread, tamped and rolled in the usual manner.

This outfit is self-propelling from power supplied by its own steam engine. It is geared for two speeds, slow and faster, the slow speed being used for grade climbing and the faster when traveling upon a fairly level road. The machine is provided with an arrangement of differential gearing similar to that used in automobiles, thus rendering it easy and practicable for turning corners and negotiating curves.

The makers claim for this machine that there is less liability of damage to materials from burning or overheating than any other machine on the market and that it has a capacity sufficient to make it a practical outfit for use in small paving contracts or for country road building.

The Monahan Pipe Layer.

A useful device for laying large sewer pipe is shown in the accompanying illustration. The machine is simple of construction consisting of a frame of 2-in. pipe the rear of which is mounted on wheels to facilitate handling. On top of the frame a hand winch and pulley tripod are mounted. The frame is securely braced with steel angles.

In operation the machine is placed astride the sewer trench one wheel on either bank. The width of tread of the machine may be varied by using different length cross pipes. The section of pipe to be layed is rolled upon two boards placed across the trench directly ahead of the pipe layer. The machine is then pulled forward until the pulley is over the pipe and the look hook illustrated inserted in the end of the pipe. The pipe is

The Monahan Sewer Pipe Layer.

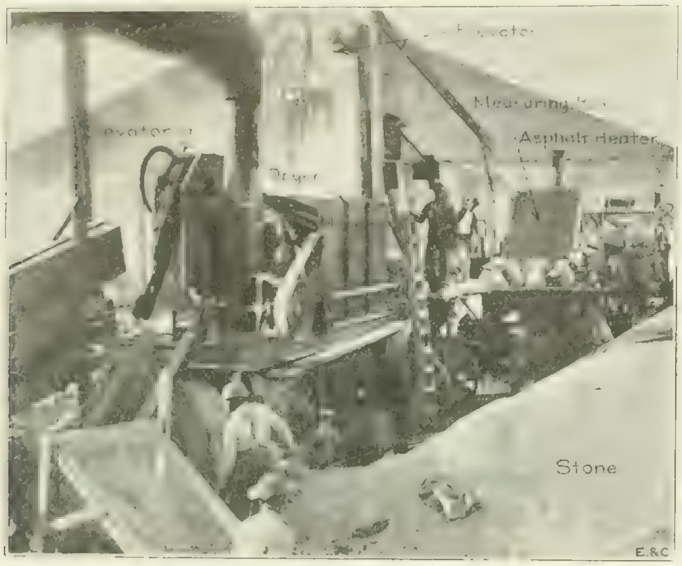
lifted slightly, the boards removed and the pipe lowered into place.

This machine will lower pipe varying from 18 to 36 ins. in diameter rapidly and with no breakage. On a sewer job in Winnetka, Ill., two men placed 175 ft. of 27-in. vitrified pipe in a 13-ft trench in 4 hrs. No effort was made to speed up, the men operating the [illegible] unaware of the fact the work was being timed.

The [illegible] is manufactured and sold by Wm. Hegie, Joliet, Ill.

A Portable Plant Adapted to the Use of Contractors on Bituminous Road Construction.

The requirements of a mixing plant adapted to the use of contractors engaged in bituminous road construction are, first, ease in assembling, moving over country roads and shipping by rail; second, simplicity of design obviating repairs and breakdowns and, third, large output. The plant illustrated is claimed to meet these requirements.



Dryer and Mixer Unit of Small Cummer Asphalt Plant

The plant consists of three units: a drying and mixing mill, a power unit, and a melting kettle. The drying and mixing unit is made in two sizes of 750 and 1,000 sq. yds. of sheet asphalt, or asphaltic concrete, laid 2 ins. thick in a day. The other units are the same for both plants. A brief description of the 750 sq. yd. capacity plant is as follows:

Aggregate is elevated by the bucket conveyor at the end of the machine and dumped into a steel air heated drum of 8 tons per hour capacity. Passing from this drum it is elevated in the vertical bucket elevator and discharged into a rotary screen, the openings in which are so arranged that sheet asphalt topping, binder or asphaltic concrete may be prepared without changing the screen. The screened material falls into a 5½-ton capacity bin, thence each ingredient passes to a measuring box on a beam scale arranged so that each ingredient may be weighed separately or combined, or a 5-cu. ft. measuring box may be used. The aggregate then passes into the mixer, where the bitumen is added from a bucket connected with the melting kettle and after mixing the finished material is dumped into wagons and hauled to the road. The apparatus is mounted on steel trucks the front wheels of which are 30 ins. by 12 ins. and the rear wheels 36 ins. by 12 ins.

Power is supplied by a belt drive from the power unit. This unit consists of a 30-hp. locomotive type portable boiler and a 25-hp. engine, mounted together on a steel truck. The melting kettle unit consists of a 10-ton melting kettle mounted on a steel truck, or two 5-ton kettle.

Each unit is mounted on separate trucks permitting easy hauling over roads and facilitating loading on flat cars for transportation to another city. In shipping the only adjustment necessary consists of removing the top of the bin containing the screen and folding down the upper 4 ft. of the elevator, which is hinged for that purpose.

The plant is modeled after a large plant manufactured for 14 years by the F. D. Cummer & Son Co., Cleveland, Ohio, and is manufactured and sold by that company.

A New Motor Street Sweeper.

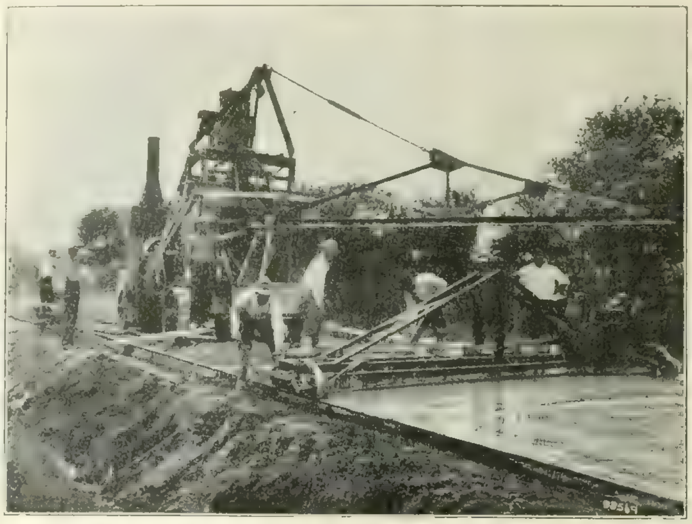
The ideal method of street cleaning must provide means of picking up refuse. The old method of sweeping or washing the street dirt into the gutters is gradually being abandoned for reasons of sanitation and economy. The new type of sweeper, of which the one illustrated is an example, is motor driven, as a rule, provides for the picking up the refuse, and frequently sprays water in front of the broom to settle the dust and prevent its flying when sweeping.

The sweeper illustrated covers a strip 8 ft. wide and has a refuse receptacle with a capacity of 3 cu. yds. The apparatus is of the light motor type and is operated by one man. Water is forced through atomizing sprays placed directly in front of the broom at a pressure of 40 lbs. per square inch. The water and air pressure are contained in a steel tank with a capacity of about 200 gals. and the pressure is automatically maintained by a compressor driven by the motor. One filling of this tank will run from 3 to 4 miles. The atomizers extend the whole width of the broom and are all operated in unison by a hand wheel convenient to the driver. The density of the spray to be thrown upon the street is under the direct control of the driver and is determined by a changing of the opening in the atomizers and not by increasing or decreasing the flow of water through the feed pipe. This spray is capable of such fine adjustment that the sweepings may just be dampened and as the machine moves along it leaves a clean moistened path without a drop of free water upon the street. A record of continuous operation, it is claimed by the makers, shows that the machine will sweep over 16,000 sq. yds. of pavement per hour of operating time at an average cost of 4 cents per 1,000 sq. yds., for the operation of the sweeper. The sweeper is made by the Elgin Motor Sweeper Co., Oak St., Elgin, Ill.

The Elgin Motor Street Sweeper.

The Baker Concrete Road-Finishing Machine.

The road-finishing machine illustrated consists of a trussed frame spanning the entire paved surface. The ends of the frame rest on double-flanged wheels which run on the side forms of the concrete roadway. The finishing parts of the machine are supported on this rigid frame and consist of two steel striking plates extending across the roadway and which are adjustable to form any crown desired. The under side of the striking plates is a smooth surface 5 ins. wide, the forward strike having in addition a steel plate set at an angle of about 60° with the horizontal that serves to shove ahead the excess concrete.



The Baker Concrete Road Finishing Machine.

In operation the machine travels by its own power, the striking plates moving across the road in opposite directions, each with a throw of 1½ ins. Traveling at a rate of 5 or 6 ft. a minute, the machine strikes the surface, shapes it to the desired crown and compresses it firmly. The degree of compression applied may be varied by changing the inclination of the lower surface of the plates, relatively more compression being required for dry concrete.

This machine is manufactured and sold by R. D. Baker Co., 73 Home Bank Bldg., Detroit, Mich.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design and to Methods and Cost of Construction

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Early Difficulties Encountered Under the Commission-Manager Form of Municipal Government.

The commission-manager form of municipal government has now been in effect in several cities for a length of time sufficient to bring to light some of the obstacles to its complete success. That these obstacles will finally be surmounted we do not doubt, for the form of government is sound in its essential principles, and requires only competent administration to prove its worth. However, the first step in overcoming obstacles is to recognize them as such and to study methods available for surmounting them. Some of the papers read by city managers before engineering and other societies have served to bring out some of the initial difficulties encountered both by the form of government and by the city manager as its most important official. Several of these points were brought out in papers read by city managers at the first annual convention of the Association of City Managers which was held at Springfield, Ohio, recently.

The greatest difficulty seems to arise from the impatience of the public with reform administrations unless more or less spectacular reforms are quickly brought about. The tendency of the people to revert to conservatism after a short season of extreme radicalism is a phenomenon often witnessed both in national and local politics. When the average man goes in for reform of any kind he expects almost immediate results. Immediate results can seldom be shown, for time is consumed in the learning of duties pertaining to positions, and in the perfecting of plans and legislation, often by the process of trial and error. During such periods of adjustment the citizen grows exceedingly impatient and is prone to renounce the new order of things. One city manager in commenting on this impatience expressed the fear that in no city as yet has there been a sufficient breaking away from the old practice of turning men out of office to give either the commission-manager form of government or the city manager a trial of sufficient length to enable either to demonstrate fitness.

The best way to quiet the impatient critic is to let him know what is going on. He will seldom remain unreasonable longer than he remains uninformed. The co-operation of citizens must be secured, and this can be effected only by a carefully planned publicity scheme. The good will of newspapers must be sought in all legitimate ways, and matter for publication in them should be prepared by a properly qualified representative of the administration. Talks before local societies and clubs are also effective measures to be employed in this connection.

Another and very serious difficulty arises from the pernicious activities of the politicians who were relegated to private life by the establishing of the new form of government. These men are hard and efficient workers in their own behalf, and their interests are wholly opposed to those of the officials of the commission-manager administration. They make it their business to misrepresent things and in every way possible seek to reflect discredit on the administration. The activities of this element must be equaled by that of the officials, who, if competent to perform their duties and diligent in their publicity measures, can offset untruth with truth.

Another obstacle is the traditional belief that in working for a city the employe is justified in rendering inefficient service. This obstacle is quickly surmounted by a firm but judicious exercise of the right to discharge the incompetent.

So far we have considered the difficulties encountered in the successful application of the form of government. Turning directly to the city manager as virtually the chief official we find that his lot is indeed a hard one, particularly in the early months of his engagement. In general, also, we believe it is more difficult and vastly more trying to be city manager of a small city than of a large one. The manager in the small city has problems fully as difficult to solve as those with which the big city manager is confronted, and has less competent help in their solution. While big city undertakings call for the expenditure of great sums of money, such sums are usually more easily secured than are the much smaller sums expended by the small cities. Municipal public improvements costing hundreds of thousands of dollars seldom present more engineering difficulties than arise in connection with works of the same character constructed on a smaller scale.

In the small city the manager must train his assistants. He cannot get them in any other way, on account of the necessity of keeping down salaries. The number of assistants he can employ must necessarily be small, and the result is that he must personally give his attention to many matters of detail. Many of the latter are exceedingly trivial, but they must be attended to or the manager will be criticised for lack of attention to his duties. One such manager says that while studying a vexing problem of law enforcement an urgent call for help will come to him on account of a stopped sewer. This is merely a suggestion of how the manager of a small city must jump from one thing to another throughout the day.

The theoretical qualifications of the successful city manager have been well emphasized. It has been said that he must have engineering and legal knowledge, a knowledge of business procedure, executive ability and a judicial mind. It should be borne in mind, however, that in addition to these qualifications the successful city manager must also be a man of even temper and one who is fearless in the face of criticism and not oversensitive about it. Again, may it be said, the additional qualifications mentioned are especially desirable in managers of small cities.

The same speaker previously mentioned said that he had done some hard work during 15 years of engineering before becoming a city manager. This was especially true when he was working in the tropics, handicapped by a strange language, inexperienced labor, an enervating climate, and long hours of work. Added to these impediments were the responsibility of the work and the necessity of keeping the cost at a minimum. "But," he said, "I have never held a position requiring such an outlay of energy, patience and varied accomplishment as does the one of city manager."

The commission-manager form of government, as a general proposition, seems to have caught the public fancy, and it is spreading rapidly. Doubtless many engineers will be offered positions as city managers. For their own good and for the good of a movement which is deserving of success they are cautioned to examine themselves thoroughly as to their possession of the requisite qualifications before accepting positions of this character. Many men possess all the mental qualifications, but lack the temperamental. Their success as city managers would therefore be highly problematical.

The Engineer and the Law.

Due to his knowledge of engineering materials and of construction methods, coupled with his reputation for honesty and fairness, the engineer has, in the past, been permitted by the public and by the courts to exercise certain judicial functions as between owner and contractor. This assumption of authority has caused the engineer to embody in his construction contracts and specifications clauses intended to give him final decision both as to the intent of his specifications and as to the character of the completed work. This often has led him to be too little concerned with the wording of the specifications; and there is little wonder that contractors are sometimes unable to grasp the exact meaning intended. Believing that he is to act as interpreter and also as judge the engineer is inclined to insert general clauses which in themselves have little or no meaning. The present tendency of courts to deny the right of individuals to assume judicial powers (which is in line with the broad policy of the state to guard more closely the rights of its citizens) has caused, in at least one state, a law to be passed which denies to the engineer or architect the right of final decision in case of disputes between owner and contractor. The fact that engineers are engaging more and more in work not heretofore considered within their province emphasizes the need of a better understanding on their part of those principles of law affecting their work.

In this issue we are publishing extracts from a paper presented by William L. Bowman before the Harvard Engineering Society of New York in which the author discusses certain legal principles and decisions of interest to engineers. In this paper Mr. Bowman criticises very severely the present general form of construction contracts and specifications, and says: "From my personal knowledge of the contracts of the city of New York, and of the interpretations of the corporation counsel and some of the engineers, I am of the opinion that in signing such a contract the contractor, architect, or engineer gambles worse than on the stock market." That many contracts and specifications are imperfect and unsatisfactory cannot be denied, and we believe that in the future the courts will require that such instruments be written with greater precision and with a better knowledge of legal principles if they are to be binding upon both parties to the contract. As is pointed out by Mr. Bowman such requirements as: "That the work must be to the satisfaction of the owner and engineer;" "that the contractor shall follow the plans and specifications and do the work under the direction of the engineer," and then to call for a perfect job or make the contractor guarantee a certain result; "that the contractor shall assume all underground risks," even though the engineer gives borings of the sub-strata; and clauses of a similar character are likely to be given little consideration if taken into court. Too often "big stick" clauses, which are unfair to all parties and which create the impression that the engineer himself does not know definitely what he wants, are inserted in specifications.

The need of exercising great care on the part of the engineer in contracting to do work for corporations and municipalities is emphasized, and the following advice is given to engineers who contemplate such employment:

(a) Know that there is an appropriation for your work;

(b) Know that the official employing has the power to employ.

(c) Keep the cost of the work within the appropriation.

(d) Get [illegible] money as soon as you can.

Unless the engineer gives full consideration to such items as the foregoing it is extremely probable that he will find himself unable to collect for his service, even though he has acted in good faith in every particular. The legal principles discussed by Mr. Bowman may well be given serious study by engineers, and even though some of his statements may not be accepted literally they are, nevertheless, pertinent and timely.

Residential Sewage Disposal an Engineering Problem.

The design of sewage disposal plants for residences is a problem for the sanitary engineer. This work is now too generally entrusted to architects or plumbers who are incompetent in this field. Those who have made a special study of residential sewage disposal realize that here as in municipal sewage disposal the design should be governed by local conditions. Moreover, if the design is not governed by local conditions the owner is practically certain to find that in paying for his sewage plant he has paid for something which will produce rather than prevent nuisance to his senses of sight and smell.

In this issue we publish an article which illustrates and describes the essential features of designs prepared, by a sanitary engineer, to govern the construction of sewage treatment plants for two estates situated near Chicago. These designs well illustrate the desirability of employing a sanitary engineer to solve residential sewage disposal problems. Engineers will be interested to note the use of two-story tanks of the Imhoff type in these two installations. These tanks are as large as some which have been successfully operated at sewage treatment experimental stations and their successful operation is anticipated. This application of the Imhoff principle is in line with the progress of the art.

Architects and plumbers are prone to use plain septic tanks, in all cases, without any form of secondary treatment and without regard to local conditions of flow, nature of sewage or ultimate disposal. These men are among those who were convinced some years ago that the plain septic tank presented the solution of all sewage disposal problems. They still hold to that view. Owners, when they have any ideas on the subject, are likely to agree with the architect and plumbers in this matter. These men have not learned that sewage disposal is a much more complex problem than it was generally thought to be 15 years ago. The fault is not theirs so much as it is that of the engineers who ten or fifteen years ago were so exceedingly enthusiastic over septic tanks. The sanitary engineers of the present must correct the existing conditions. Probably this can be best accomplished by co-operation between the engineers and architects practicing in a given locality. Engineers should take the initiative in securing this co-operation.

The Management of City Paving Problems: The Result of an Investigation in Syracuse, N. Y.

The conclusions reached by a committee of the Chamber of Commerce of Syracuse, N. Y., after a careful study of statistics from the 68 cities noted in the article on another page, including their own city, were as follows:

(1) Property owners should not be permitted to choose types of pavements to be used without the approval of some more expert authority.

(2) The city at large should pay one-third of the cost of all new pavements, as well as street intersections.

(3) The city at large should pay the entire cost of repaving.

While there may be cities in which special conditions exist that would modify the application of the method of paving management outlined in the conclusions stated, their use in practically every city of the United States would greatly simplify engineering problems and serve, in many cases, to clarify the political atmosphere. That the type of pavement should be decided by a vote of the citizens is a method of pavement selection inherited from the days of town meetings. It is an excellent example of "let the people rule" run into the ground.

The apportionment of the cost in proportion to the benefit derived is a principle of local assessment for public improvements that is applied in many forms of taxation. Those directly benefited, such as abutting property owners, are assessed for a larger proportion of the cost than those indirectly benefited. The basis for paving work of two-thirds against the property and one-third against the city appears to be an excellent ratio of apportionment.

That the city should bear the cost of repaving is perhaps the most practical solution of a rather involved problem. A pavement is usually worn out by traffic by which the interests of the abutting property owners are but little bettered. Furthermore, repaving usually consists merely of replacing a wearing surface and it is difficult to arrive at the cost of such work accurately before the repaving is undertaken.

In connection with the financing of paving work it might appear desirable, in some cases, to issue bonds for the first paving of a street, but for repaving such procedure is usually not desirable. The pavement is worn out by traffic, then why should not traffic be assessed for the maintenance of the pavement? In Chicago a wheel tax is imposed upon all vehicles using the streets, the proceeds of which is used exclusively for repaving purposes. Without entering into a discussion of the various features of the wheel tax it appears, to say the least, to be equitable.

Statistics of methods of managing paving problems are needed. From them most valuable information of utility in drafting ordinances and the systematizing of civic works may be secured. We know of no more valuable addition that could be made with little additional work to every city engineer's report, than a concise statement of the methods employed in pavement management. The method of presenting this information might well follow that outlined in the investigation conducted by the citizens of Syracuse.

The Bonus System in Paying Meter Readers.

In this issue we publish an account of the introduction of the bonus system by the Milwaukee Gas Light Co. in paying its meter readers. Under the old plan readers were paid by the hour, regardless of the number of meters read. Now a minimum number of readings per hour is required and for all readings in excess of the minimum a bonus is paid. The plan has worked out to the advantage of company and readers. The number of readers has been reduced from 26 to 22, the cost per reading has been reduced two-tenths of a cent and the average monthly wages of the readers increased from $55.21 to $61.48.

In the introductory paragraph of the article mentioned we state that the plan described might be adopted to advantage in the handling of water meter readers. Of the fact that the plan is not so generally applicable to the reading of water meters as to the reading of gas meters we are well aware. Gas services are always metered and the meters are usually read at monthly intervals. On the other hand, there are but comparatively few cities in which all water services are metered and water meters are, in many cities, read but quarterly. In many cities of even 25,000 population, partially metered, one man working part time can readily read the water meters as often as the local custom requires.

Notwithstanding these considerable points of difference, we think it likely that in the larger and secondary cities which are fairly well metered the introduction of the bonus system in compensating readers would promote both economy and efficiency. "Sidewalk" reading is readily eliminated by withholding from the reader the last reported readings of the meters.

GENERAL

A Discussion of Some Legal Principles of Interest to Engineers.

In the past the engineer has been permitted to exercise broad judicial functions as between owner and contractor, and this assumption of authority has led him to embody in his specifications clauses which have little or no legal standing. A better understanding of those laws which touch upon the rights and limitations of engineers is essential if engineers are to avoid expensive litigations in the future. The following article gives some pertinent data on this subject, the data being abstracted from a paper presented before the Harvard Engineering Society of New York by William L. Bowman.

With our present methods of building construction the burden of all such construction has fallen chiefly upon the engineer, and it will continue so to do more and more in the future. The architect still plans and outlines the work, but it is the engineer who is responsible for its stability, usefulness and permanence. With this in mind it seems strange that there are as yet no state statutes restricting the practice of engineering. Why have the engineers escaped while architects are controlled in Illinois, California and New Jersey? The only answer which seems satisfactory is politics. If that is so, then the engineer must be prepared for practice statutes in the near future similar to those governing architects in the states mentioned, as it has been held time and again that such statutes do not contravene the fourteenth amendment of the United States constitution.

City or town laws or ordinances taxing the practice of engineering in the city or town are frequent and common. The taking out of these licenses, as they are usually called, is often required not only in the place where the engineer has his office or offices, but in the place where he is doing the construction work.

These brief considerations call for the following recommendations:

(a) Practicing engineers should annually ascertain whether any state laws have been passed governing their profession, and if such laws have been passed they should carefully follow each and every provision.

(b) Ascertain whether or not there is any municipal or town license tax on the engineering profession where they have offices or jobs.

(c) Before taking employment in another state ascertain whether there are any state restrictions governing non-resident engineers.

(d) Many engineers practice their profession under the form of partnership, while the partnership named uses some designation or name other than the names of the partners. In such cases they should take care to file the proper certificates as required by law.

(e) If the form for doing business is a corporation be sure to comply with the requirements of the state in which you are incorporated, and also with the requirements for foreign corporations in those states where construction work is undertaken. This is very important, as otherwise you may do work and have no remedy to secure your pay if the same is refused, and especially if your fees are under the $3,000 limit of the federal court.

Let us now briefly take up the subject of the engineer's employment. It can be subdivided as: employment by individuals, by corporations, by municipal corporations, and by a state or the United States. No matter who your employer is, be sure to have the full and complete terms of your employment in writing. Then don't forget that such a written contract excludes in a court of law all consideration of preliminary negotiations or agreements or documents pertaining thereto prior to the written contract and not made a part of the contract. Compliance with this simple suggestion will prevent many disputes and also troubles with the statute of frauds. Know with whom you are dealing, and the extent of their authority if they are acting for others, and beware of conditional contracts, or, as it might better be stated, contracts with conditional payment.

Naturally there are less complications in employment by individuals than in the other classes stated. It will probably be sufficient to dismiss this class with merely a warning to look out for agents, committees of organizations, and irresponsible real estate dealers and operators.

When considering employment by a corporation it must be noted that a corporation is a legal person endowed with life by a charter. That charter rules and governs its powers, and hence the corporation can do nothing which is not specified or essentially implied by the charter. It is therefore possible, though not probable, that some corporations might not have the power to employ an engineer, and if one were employed and refused compensation he could not recover at law for such services. The chief difficulty, however, in dealing with a corporation is to know whether or not the officer with whom you are dealing has the power to act or has been granted authority by the board of directors to employ an engineer. Generally speaking, the employment by the president or general manager of a corporation is sufficient to charge the corporation with the employment, so that you are assured that the plea of "no authority" will not be used against you; but the only safeguard is a resolution of the board of directors.

When we think of the dealings of engineers with municipal corporations and their departments we usually think of the salaried engineers protected by the civil service. While there are many interesting questions that constantly arise in regard to the civil service as regards grading, salaries, etc., yet they are not important enough to dwell upon, especially as they are dependent upon the construction of special statutes. We are more interested with the consulting engineer in practice who is called upon to take charge of special or important work for the municipality. Early in my connection with municipal cases I made the recommendation, which later experience has shown should be a subject for a desk card, "Never do any work for a public corporation without first consulting expert legal advice." That sounds as though there might be a great deal of personal interest involved, but it is probably the best advice I shall give you in this paper. Let me tell you about an extreme case, which involved an architect, but the same principles would govern a contract of services for an engineer. In one of the New York City contracts a provision is found as follows:

The architect shall on or before the first day of November, 1904, furnish a set of preliminary studies and specifications, together with an estimate of the cost of said building or structure. If the said preliminary drawings and specifications and estimates are not satisfactory to and approved by the Commissioner, then the architect shall and will revise and correct the same so as to conform to the suggestions and criticisms and requirements of the Commissioner, and so that the estimated cost, including the architect's fees, of the cost and service and inspection shall be well within the sum of $48,000, the funds available for said building or structure.

It has been judicially determined that under such a clause it was not necessary that the preliminary drawings, specifications and estimate should be within the sum mentioned, and it was intimated that if the estimated cost, including architect's fees, ran above that figure it would become a question for the jury to determine whether or not there had been a substantial performance of the contract by the architect. In the latest case, however, the same court held that the architect was bound to furnish not only preliminary plans, drawings and specifications, which, with the architect's compensation, should be within the limit of cost of $48,000, but that the final drawings, which must include the suggestions and criticisms of the commissioner, should also be within the $48,000.

Another clause of the same contract provided for the payment of 1 per cent upon the completion of the drawings and specifications called for by the clause hereinbefore set forth, and it was contended that the architect was at least entitled to that 1 per cent. The court held to the contrary, on the ground that the approved revised plans did not come within the limit of cost of $48,000.

This case and others seem for the present to establish a new rule of law in such employment that where an architect contracts to furnish plans not to exceed a certain cost and the bids being largely in excess of said cost, so that the plans are abandoned, the architect loses his right to any compensation for the work done. This seems an exceedingly harsh and unfair requirement to put upon an architect, especially when one notes the range of prices bid by contractors upon the same plans pursuant to the ordinary advertisements for bids. How can a municipality conscientiously and fairly ask an architect to plan a building and make his remuneration dependent upon his following not only the requirements and suggestions of the official in charge, but also refuse to give him any credit in case there is delay in advertising for bids, especially when the contractors' bids will vary 30 per cent on the same plans? However, the courts have so held and it behooves architects and engineers to take it into consideration in this class of work.

In such employment the engineer, to be assured of pay for his services, must know that each and every provision and requirement regarding appropriation and employment have been complied with by the legal and proper officials or official. The board can usually only act by action at a board meeting. Hence employment by the commissioner of health of this city would not be valid since it is the board of health which must act. It might be said that the engineers and other professional men are the subjects of exception from the many requirements of the "Charter of the City of New York" regarding written contracts and competitive bidding for their jobs. There are even instances where the court has said that, after plans and specifications have been prepared for a large municipal building (and where no express provision was made for a supervising or superintending architect or engineer), it might be considered as one of the things essential to carrying in effect the main purpose of the appropriation for the erection, furnishing and equipment of the building that he could be paid from the general appropriation. It may be noted that it is now general in most of our state statutes to find a clause which shows that the appropriation for the engineering work must include the engineer's charges. Employments of this nature are usually largely political, and I hardly need warn you that you should at all times get any earned money as soon as possible, since the change of officials or new administrations cause many unpaid bills to be forever unpaid. As a brief summary the following is offered:

(1) Know there [illegible] work.

(2) Know that the official employing has the power to employ.

(3) Keep the cost of the work within the appropriation.

(4) Get your money as soon as you can.

Employment by the state involves all the troubles suggested in employment by municipalities, and also the fact that in many states you are absolutely at the mercy of the contracting official because there is no way in which you can compel payment for your services except through legislative consent to sue the state, which road is usually blocked if you have troubles with the officials. This statement is, of course, only applicable to those states like Pennsylvania, which have no court or board of claims.

In employment by the United States, although you have to exercise much the same care that you do in taking employment from municipalities, yet you have a chance for your compensation in the court of claims.

MECHANICS' LIENS.

There is no need to suggest that the engineer seeking employment should know that the party is good for his compensation. The engineer is protected very often by the so-called "lien laws." As these laws are statutory, there are few of them that are worded or construed alike. Generally they apply to all improvements of real property. Of course that will hold on any building erected (although it has been held not to apply to a mausoleum); and again in other statutes it has held good as to a dam and not to drilling oil wells; still another state allows a lien for planting trees, while another will not allow one for sidewalk improvements. These instances show that you must first find out what improvements or structures are within the provisions of the lien law.

Is an engineer entitled to such protection for his services in preparing the plans and specifications for and in superintending the construction of work? No general answer can be given, as each state has a lien law which in its wording usually differs somewhat from every other such statute. All such statutes, however, can usually be divided into two general classes—one which protects "any person performing labor" upon building construction, and the other which specifies certain persons as "mechanics, laborers," etc. This classification at the same time will give a general idea of the usual decisions on the subject as regards architects, since in the first instance they are protected and in the second they are not. Thus it would seem from the recorded cases that at present, where the architect has an entire contract both for preparation of plans and specifications, and also for superintendence, and fulfills both of these services, he is entitled to a mechanics' lien in Alabama, California, Colorado, Illinois, Louisiana, Minnesota, Nebraska, New Jersey, New Mexico, North Dakota, Pennsylvania, Rhode Island, Washington, and probably New York. On the other hand, Iowa, Kentucky, Massachusetts, Missouri, North Carolina and Tennessee deny an architect the aid of the lien law under such circumstances.

Suppose the architect merely prepares plans and specifications. Is he protected by the lien law for such services? It would be expected that if the buildings are not erected there could be no lien, but Wisconsin seems to allow a lien even then. Where the construction work is actually done there is about an equal difference of opinion in the reported cases, Illinois, Iowa, Minnesota, Nebraska and Wisconsin aiding the architect, and Delaware, Maine, Massachusetts, Missouri, New York and Pennsylvania refusing him the advantages of the statute.

Where the architect merely superintends the construction work it is certain that he would be protected in those states where he is benefited when he draws the plans and superintends the construction. New York, Rhode Island and Oregon seem to help the architect in such a case upon the theory that such services are actually performed upon the construction work in contradistinction to the work done in making the plans and specifications, which is all performed in his office.

[illegible]

When we realize that upon a single piece of work the engineer may not only be agent for the owner, but also for the contractor, and at the same time arbitrator or judge between the same parties, we must recognize his unique position. In practically all construction contracts the engineer today is paid by the owner or party having the work done, and at the same time acts in a quasi-judicial capacity in determining what the plans and specifications mean, what they call for, what work has been done at certain periods, whether or not the work is according to contract, whether or not it is properly and satisfactorily done, and the amount of the quantities for which the contractor shall receive payment. It is a precarious position for the strongest human character. Imagine the judges on our benches in the pay of the plaintiffs or defendants! Yet that is practically what has been for some time and still exists in engineering work. The fact that the engineer has stood the test so well is, to my mind, remarkable. How much longer will he stand it? That will depend largely upon the college graduate engineers who are now being turned out by the hundreds. I might add that the courts in Pennsylvania got to supporting the engineers so strongly in their decisions and certificates that the state legislature took a hand in the matter and passed a statute providing that no contract clause making an architect's or engineer's award or certificate final or conclusive should oust the courts of their jurisdiction, and that any controversy arising on such a contract should be determined in due course of law with the same effect as if such provision were not in the contract.

This was not applicable to corporations having the power of eminent domain, which, of course, made the statute ineffective in the cases where it should have had its greatest effect.

In my opinion the entire trouble is with the present-day construction contracts and their specifications, or at least the general conditions of the specifications. In discussing the present-day railroad construction contracts the following has been stated: "It is safe to say that in no other business relation between men are such one-sided agreements customary. In no other relation is a man conceived to be clothed, by reason of a written instrument, in a mantle of infallibility as is the engineer in customary railroad contracts." The same is true of all United States, state or municipal contracts, and even of many private contracts. From personal knowledge of the contracts of the city of New York and of the interpretations by the corporation counsel and some of the engineers, I am of the opinion that in signing such a contract the contractor, architect or engineer gambles worse than on the stock market. He frequently becomes an insurer and an exponent of faith, hope and charity. The unrest caused by these classes of contracts has kept all the contractors and many of the architectural and engineering organizations in this country and in England busy discussing the subject and trying to better the conditions. In England they have done much better and gone much further than we have. Lately the new standard forms of invitation to submit a proposal, the contract bond and the general conditions of the contract of the American Institute of Architects are quite an advance upon the old-style so-called uniform contract. In private work the use of the cost-plus-percentage, or cost-plus-a-fixed-sum, has made great advances, and seems generally to prevent many of the common disputes of the past. You as engineers can help this situation by never presenting to a contractor a contract in a form which you would not be willing to sign were the situations changed. Be sure that the general conditions of the specifications are not contrary to or different from the contract requirements. Do not call upon the contractor to do everything which the engineer deems part of the complete performance if it is not shown on the drawings or mentioned in the specifications; that this or that part of the work, or method of working, etc., shall be determined by the "discretion or judgment of the engineer"; that any and all changes can be made at the demand of the engineer; that the work must be to the satisfaction of the owner and engineer; that the contractor shall follow the plans and specifications, and do the work under the directions of the engineer, and then call for a perfect job or make him warrant a result; that the contractor shall assume all underground risks, even though the engineer gives borings of the sub-strata; that the contractor shall tear down any portion of the work at any time before complete acceptance at the direction of the engineer, etc.

This brings us to one of the most frequent causes of trouble between owners, engineers and contractors—namely, the inability of some engineers to express their requirements clearly, concisely and in plain unequivocal English, so that all concerned may read and know what their specifications mean and call for. Most of this trouble can be ascribed to the practice of copying specification provisions from some other person's work or from some ancient specifications with no regard or consideration as to whether the class of materials is the present market classification or whether even obtainable except at an exorbitant price. Such specifications usually contain ambiguous phrases which have been rightly named "club or big stick clauses," unfair to all parties, and which create the impression that the engineer himself does not know what he wants, and that he expects to cover up his deficiency by other common phrases, such as "the decision of the architect as to the true construction and meaning of the drawings and specifications shall be final"; "that all work and materials must be to the entire satisfaction of the engineer"; "that all materials must be of the best quality"; "that all work must be done in the best manner as the engineer shall direct," etc. Nor do these expressions always accomplish the expected result. For example, where a contract for a heating plant provided for a "complete and perfect job, even though every item required to make it such, is not specially noted in the drawings or these specifications"; also that the contractor "shall furnish all labor, tools and appliances necessary to complete his work according to these specifications, and shall perform his work in a true workmanlike manner in every particular, and thus provide the building with a durable and mechanically perfect system," it was held that the contractor was not required to improve upon the plans in order to make a mechanically perfect system.

Similarly, when a contract requires the construction of a cellar according to specifications it was held that an additional requirement that "the whole to be perfectly water-tight and guaranteed" only bound the contractor so far as his own work was concerned, and that he was not held to guarantee that the plans would produce a water-tight job. In another instance, where a tin roof of the "best quality" was called for the trial justice in charging the jury held that such a requirement was satisfied when the roof as finished "was equal to the standard contemplated by the contract." These examples are given because of the tendency on the part of some architects and engineers to reject work under such circumstances, should be avoided, because it often causes a contractor to increase his estimate, and because it opens the door for questioning the engineer's motives. If the specifications are made liberal in this respect, and call for material of a certain make or equal, the engineer is, of course, the judge as to what is equal, and the owner is thus protected in this respect. It might be here noted that if the specifications do call for a particular brand or equal the contractor may use the equal material in the first instance, and it has been held that such use could not be made to depend upon the question as to whether the material specified was procurable or not. I quote the following excerpts from a late written discussion of the subject:

> The engineer or professional adviser who draws up the specifications is too lazy to write out the details of the paragraph, and so he says we will leave that to the judgment of the architect or the engineer. It is the result of his own mental laziness. Now, then, if you go to the opposite extreme and specify everything, there is nothing left for the engineer to decide, and there is nothing left for the arbitration to decide, . . . leaving also much less to fight about than if you left the things to the discretion of the engineer or put in 'big stick' compulsion clauses, which do not belong there.
>
> Let the professional advisers work entirely for the man who employs them, and nobody else, and not have him a judge of any kind whatever. When he is not acting as a judge, he will write specifications that will explain themselves. . . . It is morally wrong to have a judge in litigation paid by one of the litigants. If our judges on the bench were paid that way . . . you would get wrong decisions: and this is a case where you propose to have the judge paid by one of them, the owner, and expect him to judge fairly between the owner and the contractor.

In connection with this subject, however, it should be noted that the satisfaction of an architect does not permit the engineer to force his personal idiosyncrasies or personal tastes upon a contractor. To require work to be done "in the best workmanlike manner," or "with material of the best quality," does not permit the engineer to declare arbitrarily and unreasonably that work or materials are not such as called for in the body of the specifications. The legal rule for these instances is "that which the law will say a contracting party or engineer ought in reason to be satisfied with, that the law will say he is satisfied with"; or, in other words, all that is required is materials and workmanship which would satisfy that legal creation named a reasonable man.

Thus it is that materials and workmanship for a building cannot be compared with portraits, statuary, clothing, etc., which require the absolute satisfaction of personal taste.

Just lately I have been engaged in looking over and criticising a proposed amendment to our state municipal law. This amendment was proposed chiefly to prevent the abuses of narrow-minded, straight-laced or dishonest engineers and other officials. It has been criticised by some as foolish, on the ground that you cannot legislate such a difficulty, but that it all lays in the education of the engineer; that we must educate him more broadly, give him more common sense and not so much book knowledge and technical theory. While I agree with the education proposition, I am of the opinion that we can prevent by laws some of the wrongs committed by the engineer.

Take all of our big engineering work: The engineers are required to estimate on the quantities and fix the kinds of work. Why should we not take the next step and make that estimate a bill of quantities which shall become a part of the contract and fix the contract work? Would not that save many disputes and help to drive away that specter of extra work for the owner or the nightmare of work without pay for the contractor? In England and somewhat in this country, especially with engineer's building construction, this is the practice, and it also usually enables the owner to get a lower figure for his work, since the contractor does not have to charge for the time spent determining the quantities and then adding a big percentage over his cost to cover contingencies hidden in the club clauses and evasions in the specifications which are going to permit the engineer to make or break him. Our methods of submitting plans and specifications for estimates have much need of improvement. It must come. Our economic waste in this our present method is too great to last. (This phase of the subject was discussed in detail in our June 10, 1914, issue.—Editors.)

A Strauss lift bridge was put into operation recently across the Fraser river at Fort George B. C. This bridge is 2,654 ft. long and required 6,500 tons of steel in its construction. The total cost was about $1,500,000.

Cost of a Kilowatt-Hour.

The final summing up of the results of operation of any power plant is the cost per "kilowatt-hour." This term is certainly brief and looks as if it might be easily understood. But when it is considered that there are more than a score of items involved in power plant operation, and that every one of these items is more or less variable, and finally that each item has its individual effect upon the cost of a kilowatt-hour, the term looks somewhat more formidable. The present article, which is from an article by Albert F. Strouse in The Electric Journal, explains the terms and elements involved in power generation.

In taking up the "kilowatt" we are dealing with a term, which at first seems to have no relation to anything connected with horse-power. A kilowatt is primarily an electrical term, while a horse-power is considered a mechanical term. The fundamental electrical

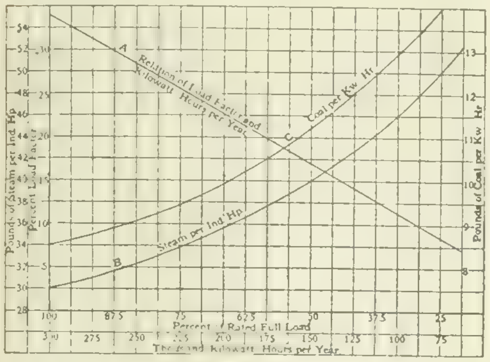

Fig. 1. Load Characteristics of a 100 KW. Plant Operating 3,000 Hours Per Year.

unit of power is the "watt," one thousand of which make one kilowatt, from the prefix "kilo" denoting 1,000. A watt (in direct current circuits) is the product of one volt and one ampere.

It has been determined experimentally that one watt is the power equivalent of 1/746 of a horse-power. Therefore 1,000 watts or a kilowatt is 1,000/746 or 1.34 times a horse-power. Thus to convert kilowatts to horse-power multiply by 1.34, and to convert horse-power to kilowatts divide by 1.34. A kilowatt used for one hour constitutes a "kilowatt-hour."

The cost of power may be separated into two main divisions, "fixed charges" and "operating charges," their sum making the "total operating expense." The total expense divided by the number of kilowatt-hours used, gives the cost of a kilowatt-hour. It is evident that it is necessary to have an accurate record both of the total operating expense, and of the number of kilowatt-hours. This latter can only be obtained by accurate measurement with suitable integrating or recording instruments.

In summing up the fixed charges we have as the main items, interest, depreciation, insurance and taxes, obsolescence and fair profit. The fixed charges considered as a lump sum remain the same when taken as a yearly, monthly, or daily expense, but this method is neither satisfactory nor correct, because it prohibits an accurate determination of the cost of power per kilowatt-hour. Obviously, a greater number of kilowatt-hours makes the cost per unit less and vice versa. This is the thought that is expressed by the term "load factor."

Load factor is a measure of the ratio of the actual power generated (or consumed) by an installation to the maximum that may be generated (or consumed). Unfortunately, there are several interpretations of the term, so that wherever it is used it becomes necessary to explain what particular meaning is intended. The differences are caused by a lack of uniformity in choosing the maximum power. It may be taken as the full-load rating of the apparatus times the total hours in a given period or times the actual operating hours in a given period. Also it may be considered as the maximum demand for power over any given period and multiplied by the total possible hours or by actual running hours in another period. In order that the reader be not confused, we will hold to one idea, viz., the load factor is the actual power consumed per annum divided by the product of the installed rated capacity times 8,760 (hours in one year).

The average load factors in various industries are pretty well known. A few of them are as follows:

	Per cent.
Boiler shops	10 to 20
Shoe factories	15 to 25
Breweries	about 45
Cement plants	60 to 90
Coal mines	15 to 30
Cotton mills	20 to 30
Flour mills	20 to 25
Foundries	5 to 15
Knitting mills	about 25
Machine shops	5 to 25
Clay products	15 to 20
Tanneries	10 to 20
Textiles (general)	about 25
Woodworking	5 to 30

These figures apply broadly to the industries as a class; certain cases will not lie within the given limits.

It is frequently necessary to arrive at a preliminary approximation of the total power that a plant would generate or a factory would consume. This can be done by figuring the maximum possible power from the rated capacity of the installation and multiplying by a load factor selected to suit the case. For instance, in a certain machine shop, the conditions indicate that the existing load factor would be nearer to the high than to the low limit of the average figures given in the preceding list. Suppose we say it will be about 20 per cent and suppose the shop power plant to have 100 kilowatts of rated generating capacity. The maximum power that could be generated in a year would be $100 \times 8,760 = 876,000$ kilowatt-hours. Then with a load factor of 20 per cent, the actual power would be 20 per cent of 876,000 or 175,200 kilowatt-hours.

Money invested in a power plant incurs a permanent yearly expense. This expense is no respecter of business conditions. Unlike the cost of labor, coal, etc., it does not decrease when the plant is running slack or cease when the plant is shut down. Therefore, the magnitude of the fixed charges can only be realized when based upon the net power. Again take the example of the 100 kilowatt-hour plant and let its original value be $10,000 and the fixed charges as follows: Interest 5 per cent, depreciation 5 per cent, insurance and taxes 2 per cent, total 12 per cent. The annual fixed charges are 12 per cent of $10,000 or $1,200. If this plant has a load factor of 100 per cent, the total power generated would be 876,000 kilowatt-hours, and the fixed charges per kilowatt-hour would be $1,200 divided by 876,000 equals $0.00137, but if the load factor is only 25 per cent, the fixed charges are four times as great, or over half a cent.

The amount of fuel used per kilowatt-hour depends upon two things, viz., the amount of steam generated per pound of fuel fired and the amount of steam consumed per kilowatt-hour by the various power plant machines. Each of these items is dependent upon other variables. The steam product per pound of coal depends upon the heat value of the fuel, the manner in which it is fired, the quality and temperature of feed water, the general boiler conditions and the load. The steam consumed by prime movers per unit of delivered power depends upon the efficiency of the machinery, the method of drive between engines and generators and driven machinery, upon the many details of transmission up to the point where power is used, and lastly upon load conditions.

In order to give more complete information regarding the variation of steam and coal per unit of power, attention is called to Figs. 1 and 2. The curves on Fig. 1 are based on the assumed 100 kilowatt plant operating 3,000 hours per year, and the assumed efficiencies, steam consumption, and steam generated per pound of coal. Curve A, shows the relation between load factor and kilowatt-hours generated per year. Curve B shows the pounds of steam per indicated horse-power at the engine at various operating loads, the loads being simply another expression of the power outfit. Curve C shows the coal consumed per kilowatt-hour corresponding to the conditions of steam per indicated horse-power. This curve involves the efficiencies of the machinery and coal rate.

Figure 2 shows the cost of coal only per kilowatt-hour with coal at different prices per ton, and based on the coal consumed per kilowatt-hour as given in Curve C of Fig. 1. Next

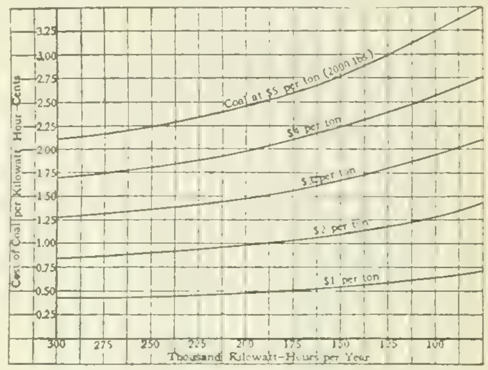

Fig. 2. Cost of Coal per KW.-Hr. at Various Loads on a 100 KW. Plant.

to fuel, the labor is the highest expense in operating costs. Inasmuch as it is a constant expense it affects the kilowatt-hour cost in the same manner as the fixed charges.

Miscellaneous expense will include feed water, repairs, oil, waste, packing, etc. These expenses vary with the size, type and care of the plant.

In a manufacturing plant, having several departments which not only require power, but also use steam for various processes requiring heat, in addition to the general heating required in cold weather for buildings, the cost of generating live steam must be separated from the cost of power. It is obvious that to get the correct cost of power, the entire expense of the power plant, especially the boiler plant, must be analyzed and proportioned accurately. This is particularly important in negotiations between central stations and industrial power users. The solution of these points must come from actual test.

Method of Planking for Drag Line Work Over Soft Ground.

The sketch shows a method of planking for drag line excavation work for drainage ditch near Viro, Fla. The ground consists of a top layer of vegetable fiber on which a man can stand in most places, but which will not carry a team. A 6-ft. bar can be shoved down its whole length with one hand. On 6x6-in. stringers laid parallel to the direction of movement are laid platforms of 3x12-in. x 12-ft. planks set close, and at the center of each platform is laid a roller track of three 6x12-in. x 14-ft. timbers set close. These track timbers are staggered better to distribute the load to the plank. The stringers are pressed down into the ground by the weight of the

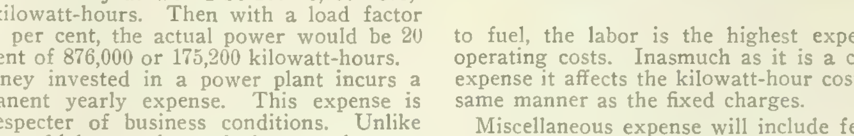

Fig. 1. Sketch of Planking for Drag Line Excavation Work Over Soft Ground.

excavator and apparently so confined the material as to prevent it from squashing out sideways under the ends of the planks. The stringers have to be dug out to be shifted ahead, but the planks can be easily picked up. Four pitmen pile the plank in bundles behind the machine which with a chain hooked to the bucket picks up the bundles and swings them ahead for the pitmen to relay. The four pitmen, with the use of the machine as described, pick up and relay the stringers, plank and track timbers as fast as the machine can work. The excavator used on the work being noticed is a Bucyrus Co., Class 14; the contractor is the Marshall Contracting Co., and Mr. Geo. Stanberry describes the system of planking in the "Excavating Engineer" for September, 1914.

WATER WORKS

Some Observations on Water Works Valuation.

Among the points on which there is a divergence of views in the details of procedure and analysis in water works valuation the following may be mentioned: Is the value the same whether the case be one of condemnation or of rates determination? Should the value be based on cost, net income or estimated cost of reproduction less depreciation? How much should be allowed for the various items which are generally known as "intangibles"? Should depreciation be deducted from the established value whether the company has been earning enough to pay it or not? Should the cost of pavements laid by the city after the works were completed be included in the valuation? In rates, should the company always be allowed a reasonable return on the investment, or should rates be based on what other cities are paying for similar service, regardless of cost? Is the term "going value" synonymous with "development cost" or is it the present worth of an income, the amount of which is changing from year to year. If "going value" or development cost is to be included, how is it to be determined? Should depreciation be figured by what is known as the "straight line" or the "sinking fund" method, and if the latter, what rate per cent should be taken? Should the company be allowed for items once useful but now obsolete? If "going value" or development cost measured principally by past losses is to be added to the physical value, should not undue returns in the shape of past dividends be deducted therefrom? Considering all these difficult questions it must be evident that even where the tribunal is actuated by principles of pure justice and equity, it must frequently be impossible to decide questions of valuation without serious loss to innocent parties. The present article, which is based on a paper by J. W. Ledoux before the Pennsylvania Water Works Association is a plea for fairness in making valuations. Mr. Ledoux's response to the discussions drawn out by his paper is also given since in it he amplifies many points touched upon in the paper.

The courts may be in a position to make decisions that conform in the highest degree to fairness because they are presented with all the data with the most elaborate completeness in detail, but each party is endeavoring not to instruct the court in the truth, but to show his side of the case in the most plausible and attractive manner, and it becomes a battle wherein sharp practice and duplicity are common weapons. Even engineers commonly hold that when they are employed on one side it is not their duty to expose the weak points in their own case, and in this phase of the matter, according to the writer's opinion, lies the weakness of the courts, which might as a partial remedy employ their own witnesses.

If a municipality were to negotiate for the purchase of a public utility operating therein, and if the rates were fixed and the laws did not provide for condemnation, it is evident that not the cost but the value of the utility would be the main consideration, and in determining this value the net revenue, present and prospective, would be the controlling factor.

If the municipality has the right to condemn and the laws of condemnation prescribe only that the value of the property is to be determined and paid for, it appears reasonable that all the elements entering into the value of the plant should be given consideration—the cost of reproduction, the net revenue and the rate at which it is varying.

If a public commission has in hand the establishment of rates for a utility it would seem that what it should have cost to date would be the proper valuation on which to base the fixed charges. This can sometimes be estimated with more reliability than the books and records of the utiilty indicate, for the reason that the works may have cost less or more than reasonable, dependent on the relative efficiency of the constructing and operating organization. The tribunal should concern itself with average and not unusual conditions.

Among experienced builders of water works, who have kept close accounts of the cost, there is a firm conviction that estimates are generally very much too low, unless a liberal allowance be made for items apart from the actual physical cost.

Such things which cannot generally be avoided in average or ordinary practice are: The cost of preliminary engineering, legal, financial and other expert reports, cost of promotion and organization; and after these are effected, the detailed surveys and designs have to be made and the purchase of land, rights of way, water rights, the examination of titles and the general management of all these matters have to be attended to before actual construction is begun. Then besides the cost of construction we have such items as engineering, architectural, chemical and legal expert services, liability and other insurance, cost of law suits and damages; interest on money during construction, general contractor's profit, commissions on financing or discount on bonds; and after all these are reasonably allowed for there should be added an item of contingencies to cover things which cannot be foreseen, but which in some form or other are sure to occur, such for instance, as floods, washouts, wind, frost, errors, accidents to the plant or materials of construction, delays and extra expense due to various causes, such as non-arrival of the material, injunctions, trouble in obtaining permits, changes of grade, caving, settling of embankment and foundations, strikes, failure of sub-contractors, and many others which should be allowed for in a general percentage based on experience.

Take the item of General Contractor's profit, which is usually objected to in valuations and is seldom seen in the estimates of men looked upon as authority in this line of work, but no one knows better than the experienced manufacturer how necessary it is to include not only such an item but overhead charges in estimating the price to be charged for his commodities. Take such a plant as the Baldwin Locomotive Works, and suppose they figured the actual wages of labor and the cost of material and added to that only such ordinary sums as are commonly used in expert water works valuations, it would be a question of but a short time before they would be in the hands of a receiver, but, as a matter of fact, the salaries of the officers of the company, the rentals of the property, the cost of advertising and procuring business, taxes, insurance, bookkeeping, depreciation and interest on the plant and property, the cost of making good defective work and taking care of all fixed charges during periods of depression, and many other items, are a part of the actual cost of the sale articles, to which must be added a sum for profit, if the business is to be successful. The same principle applies to a general contractor of water works; pipe, trenching, lead, yarn and labor, are only part of the actual cost to the contractor. He has his plant to maintain, his managing force, his office, and practically must consider the same items as are included in the estimate of a skillful manufacturer, and that he does not include such items is proved by the fact that it is only a question of time before nearly all these water works contractors become bankrupt.

When the works are finally finished, capable of supplying service to the community, it usually takes from 10 to 30 years before there can be obtained enough contracts to pay operating expenses and fixed charges. During this period there will be further improvements and extensions, and all this time the owners will have to pay out much more than they receive, and this deficiency of receipts is a necessary item, which should be allowed for in the development cost of the plant.

There is an opinion, which is probably held by the majority of engineers, that the valuation of a plant should be the same whether the question of rates or purchase be considered. This view, it would seem to the writer, is theoretically unsound. There is no such thing as true value; it depends on the human temperament and a myriad of other conditions which are constantly changing; for instance, the truest measure of value of our great commodities, such as wheat, corn, cotton, etc., is obtained in the quotations made from hour to hour in the boards of trade and even the value of a great property like the Pennsylvania Railroad will vary by millions of dollars in a day without any change whatever in the physical elements; therefore, there is no such thing as a fixed yard-stick with which to measure values. We can measure the length of pipe and the dimensions of structures with certainty and can determine with some degree of approximation what these items have or should cost, but when we come to determine value without definitely fixing the meaning of the term, irreconcilable disagreement is almost sure to result. Take the simple case of a property which is to be sold; the price or value will depend upon many elements, such as the earnings of the property and the relative desires of the owner and the prospective purchaser, and these may not have much to do with the actual cost of producing property. In the same manner in the case of the purchaser of a water works, there is a given property with a given revenue to determine its value, or relative desirability to the purchaser, and it is evident that this value is influenced more by the net income and its permanence than by the cost of the property.

In a case of determining valuation for the establishment of rates, the tribunal must decide whether the venture is to yield a profit or loss to the owner. As profit or loss is dependent upon the investment, and as the revenues which go to make profit or loss are dependent upon the rates, it is evident that the cost or investment and not value can be the only logical criterion for determining rates, and therefore the moment it is decreed that the state through a commission or tribunal has the right to establish or regulate rates, its power to fix value follows as an inevitable consequence.

It will generally be conceded that the state should recognize as just such revenues as will pay the owners of the utility a reasonable return on the investment. But the investment of the present owners of the utility may have been based on value instead of cost. They may have paid for the utility by reason of its existing or prospective net revenue, more or less, than the cost of the actual items entering into the creation of the property. Where there is no contract it would be desirable and equitable if the value of all public utilities were identical with the proper and necessary investment, so if the rate-making commission were presented a case in which the rates were so high or low as to produce revenues which would make the value different from that necessary cost, it would seem proper that these rates should be readjusted. The selling price of the stocks and bonds of the company should have little or no weight in determining the valuation. The heretofore existence of injustice to the people or to the utility does not establish the right to its continuance.

In the abstract, value varies as revenue; revenue varies as rates, and if rates should vary as cost then value should vary likewise, but if the commission does not have the right to establish rates, then value varies as revenue; revenue varies as rates, but rates depend upon what the traffic will bear and not upon cost; therefore, it is seen that for an unregulated utility cost and value have no necessarily direct relation.

Heretofore in cases of condemnation by the municipality or in arbitration of the value of the utility, it has been held with justification that an important element of valuation is "going value," which represented what a purchaser could afford to pay for a given plant with revenue above what the same plant was worth if the purchaser were obliged to wait for the acquisition of that revenue. This item is in such cases a true element of value, but it cannot be conceived of in the same manner in rate cases, because the rates have not yet been established upon which to figure the growth of income. In any case it would be more proper to call this item the cost of development, which may be defined as the necessary net outlay over and above the cost of the physical plant up to the time it is earning an assumed reasonable return on the total investment.

There have been many ingenious attempts to determine these elements of cost by rational computation, but as these calculations have been based more or less upon future prediction, the results when obtained by different computers of equal ability have varied materially.

It would seem to the writer that the logical method is to take actual experience with the property under consideration, a method that has been recommended by the Wisconsin Commission. The practical difficulty about the adoption of this method is that the plant might have been grossly mismanaged or more than ordinarily well managed. In the former case the company would obtain a premium for inefficiency; in the latter case a penalty for displaying more than ordinary ability, and it is difficult to figure what the results would have been under average conditions.

There is some justification for a method which will take the plant as it exists and calculate what it would reasonably cost to construct and develop up to the present time, but, of course, this would depend entirely upon the appraiser's judgment. If he were a man of wide experience, he would probably take the exact accounts of the company as far as practicable, if available, and modify such items as he found to be grossly different from average practice. In many cases, however, the accounts of the utility are so kept that they would afford the appraiser very little assistance and are frequently misleading on account of the illogical system in use. In such cases it is necessary to estimate the cost by the aid of experience in other more or less similar conditions. A man of large experience in constructing and operating water plants can make as close an estimate of the cost of say a million-dollar plant in two or three days' work as he could in six months. Nevertheless, in order to satisfy the courts it has been considered necessary to fortify all the estimates with the most painstaking detail, which, while not adding in the least to the reliability of the final result is intended to impress the court with the profound accuracy and reliability of the work. A given piece of construction, such, for instance, as the steam and exhaust piping of a pumping station, may cost anywhere from $5,000 to $10,000 dependent on the conditions and skill with which the work was carried out, but this large element of uncertainty cannot be eliminated by counting all the fittings and minutia entering into the work. Ask for bids on a given pumping station or piping job and let ten skilled contractors estimate. Their figures will vary anywhere from 10 to 50 per cent, and this variation concerns items about which there are no difficulties in calculation, but when we come to estimate the sum that should be added for development cost (what is frequently known as "going value") we are confronted with such great difficulties and uncertainties that unless the exact experience of the plant is to be taken as a basis, it would seem that the engineering profession should agree upon some uniform allowance to use in every case. To determine what this allowance should be it is desirable that a careful history of a large number of representative plants should be made for towns having populations from 2,000 to 1,000,000 and the data so obtained could be used for calculating the difficult and uncertain elements of the plant under consideration. The modus operandi of such a procedure would be first to estimate the cost of the physical plant and property as it exists, taking average prices that have been current for the past ten years. To the sum so obtained would be added certain percentages for each of the following items:

Promotion, organization expenses, engineering, financing, legal expenses, interest on the cost during construction, rental or cost of construction plant and housing of employes, contingencies, and general contractor's profit. From this total sum should be deducted the cost of constructions which are obsolete or of no value in the plant; the remainder would be the cost to reproduce the useful physical plant as it stands, to which should be added a general percentage for errors and constructions that are useless but which are practically sure to occur and impossible to avoid in the majority of plants. Then there should be added a general uniform percentage for development cost, which is determined from the analysis of the history of the representative plants above referred to. From the writer's own experience he is of the opinion that the following allowances would not be far from average practice, for a plant costing about $1,000,000; physical cost, 100 (material and labor only); promotion, 2½; organization expenses, 2½; engineering, 3; financing, 18; legal expenses, 2; rental or cost of construction plant and housing of employes, 2; contingencies, 10; interest during construction, 5; general contractor's profit, 15; development cost, 20; allowance for useless items, 3; total 183 per cent of the physical cost.

These percentages, of course, are not proposed with the idea of recommending their use, but they are the result of offhand judgment based on a considerable amount of practical experience. At any rate when such percentages are once determined and recognized as reasonable, the method would seem to be the most satisfactory that could be developed for arriving at the proper cost, and it will in a majority of cases be fair to the owner of the utility and to the people. Many engineers, of course, would object to this method on account of the arbitrary fixing of certain percentages to allow for uncertain elements of cost, but no matter what allowances are made they must necessarily be based largely on speculation, and it is an advantage where speculation cannot be avoided for all engineers to speculate uniformly, and then we will not be confronted with such cases as came to the writer's notice some time ago, when three eminent valuation engineers differed in their estimates of the "going value" by 300 per cent.

At first sight the method would appear to give the recently established water company an undue advantage by reason of its not having incurred the expenses of development belonging to one established for over ten years, but these expenses are sure to occur later and the rates should be such as would yield the desired revenue when the consumers are connected up at the end of the development period, or the time at which the water works would begin to yield a reasonable profit on the investment.

In recent valuations it has been the custom to calculate the accrued depreciation and deduct this from the determined valuation of the plant. In determining *value* this is no doubt an intelligent procedure, but it would appear unjust to make this deduction in case of fixing rates or condemnation where it is known that the water company has neither earned a profit nor a sufficient amount to cover depreciation. The only source the water company has to meet its expenditures is the people or municipality, and, therefore, it would seem that the people must at least pay the cost of operation, fixed charges, and a sufficient sum to maintain the plant in a 100 per cent condition, so that if the water company has not as yet received any profit the valuation should certainly not fall short of the depreciation. On the other hand, if the water company has been earning a sufficient amount to pay all operating expenses and fixed charges, a reasonable profit and a sufficient sum to cover accrued depreciation, and this depreciation fund is not available for the use of the water works, then it would seem proper to deduct it.

The question of street pavements also leads to a confusion of thought. If the works were built when no pavements existed and which were afterwards constructed by the city, should the rates be increased thereby, and in case of purchase or condemnation should these pavements be an element to increase the value of the utility? In the case of rates, the municipality has added a feature which increases the cost of maintenance and the final duplication of the piping system, and to that extent the revenues and rates should be increased. In case of purchase by barter, where the municipality has no right to condemn, these pavements certainly add to the value of the utility on account of the greater cost to the municipality of duplicating the piping system. In case of condemnation by the municipality or the establishment by the commission of a sale valuation it is a serious question whether the court or commission should recognize as just an element of value that is created by and at the expense of the people.

Summarizing, it will be seen that where the property of a public utility is under regulation by the state through commissions or courts, the writer is strongly inclined to a belief in the equity of basing the valuation upon the necessary investment instead of value.

The public should not expect people to make investments in regulated utilities and afterwards have part of their investment confiscated by so-called determination of value.

Where an investment is made in an unregulated business or manufactory the owner must necessarily take the chances on the vicissitudes of competition, supply and demand, and he never can reasonably expect to receive more for his property than its market value; and he will do his utmost to see that his investment shall never be greater than its market value.

But, in the public utility or water works plant whose rates are now to be determined, based on the so-called "true value" of the property, the owner of the utility at once knows that his investment is in jeopardy, and unless the court or tribunal which is to pass upon this matter is broad enough to protect the rights of the owner in the same degree as it protects the rights of the public, and is uninfluenced by the public clamor for depreciation, the owner is sure to suffer. The only course open which will do justice to the water company is to base the valuation on the investment, and while there are some exceptional cases in which this principle would seem to fail, they are almost rare enough to be negligible, and it is believed that even now the commissions and courts are leaning toward the idea that valuation should in every case, where possible, be based on the proper, legitimate and necessary investment required to bring the property to its present condition.

When an engineer undertakes an inquiry into any problem whose solution depends upon intelligence and justice he makes no permanent headway by basing his conclusions upon the opinions or pronunciamentos of others, no matter how reputable. We say the product of labor should belong to its producer, not because great authorities like John Stuart Mill, Adam Smith, Richard Cobden or Henry George said so, but because it is an axiom of justice; hence when a water works is appraised at less than it properly and legitimately cost, the laborer or owner is compelled to give up part of his own handiwork to others who have not produced it.

DISCUSSION.

(The preceding paper was discussed by Messrs. Geo. W. Fuller, Morris Knowles and John Chester. In the following paragraphs Mr. Ledoux replied to points raised by these engineers and further discussed the subject in some matters of detail.)

Mr. Chester thinks that the writer will have difficulty in convincing the commissions and courts that the so-called "intangibles" will amount to something over 83 per cent of the physical cost. In the first place it must be understood that what is meant by physical cost is the bare cost of labor and material, for instance, if the cast iron pipe weighs 50 lbs. per foot and costs laid down 1.2 cts. per pound, and the labor, lead and yarn necessary to lay, cost 40 cts., the physical cost of pipe is $1 per foot to which the percentage items enumerated must be added to get the complete cost of the finished proposition. This estimate was based on the conception of plants costing in the neighborhood of $1,000,000.

Promotion represents about what a man of diplomatic and social mixing ability, combined with a general knowledge of water works undertakings, would modestly expect for his services in working up a proposition and pushing it through the various municipal bodies to the point where it becomes attractive to water works investors, and then comes the by no means easy task of finding not only an investor, but a reliable concern to organize and manage the undertaking. Out of the allowance for his services he would have to pay for more or less expert engineering advice and be under a considerable burden of expense. Some may assume that the writer has in mind that graft or bribery will constitute a certain portion of this unavoidable promotion expense. While we fully realize the widespread existence of human frailty it must not for one moment be considered in an engineer's allowance and properly merits his disgust and condemnation, as would any other species of criminality.

Organization expenses, which we estimate at 2½ per cent, would probably be greatly exceeded in a small undertaking. These costs are real and material—the preparation of the mortgage and the franchise—legislative expenses, legal and business services, and the compensation in some way or other of people of worth and influence, who, in a perfectly legitimate manner, consent to be identified with the enterprise.

After hearing the admirable criticism of Messrs. Geo. W. Fuller, Morris Knowles and J. N. Chester, I feel convinced that my allowance of 3 per cent for engineering was too low as an average, but was justified for the cases I had in mind, where for works costing in the neighborhood of $1,000,000 3 per cent would generally be sufficient. However, as this estimate was based on the final developed plant after a considerable number of years, and as there are engineering expenses required in piecemeal, the final total would generally be greater; and then, again, for works costing less than $100,000, 3 per cent would be entirely inadequate. There are sometimes as much engineering expenses in a small works as in one costing many times as much, and probably it would be better to cut out engineering altogether from the item of intangibles and estimate it as a part of the physical cost based on the amount of skillful engineering that is required in the specific plant under consideration.

Financing.—If the final cost of a plant without making any allowance for the cost of financing is, say, 65 per cent higher than the actual price of labor and materials on the ground, then, when we consider the ordinary average practice of selling bonds to obtain money for construction, which is the usual method of financing water works, we find that about 90 is as good a price as water companies can expect to obtain. Therefore, if we divide 165 by 90 we would find that the allowance for financing, as measured by the discount on bonds, should be about 18 per cent. In other words, 18 per cent on the physical cost of labor and material is equal to about 11 per cent on the total cash cost.

Many valuators omit this item on the ground that the water company should have the total amount of cash in hand before undertaking the work, but this is almost never done, the general method being to sell bonds. In some of the court cases there has been a distinction made between the investment by the stockholders and that made by the people who have loaned the stockholders money, and in that case the return on the money loaned was to be paid back by a sinking fund, but that seems to be complicated and not the business of the tribunals. The total amount of money invested according to the prevailing methods of raising money in such enterprises would seem to be the logical view. If the water company has to pay 8 per cent on the money borrowed when the prevailing rates are 6 per cent, it is their own fault and if they are able to borrow money for 4 per cent when the prevailing rates are 6 per cent they are entitled to the difference.

Legal expenses at 2 per cent is certainly not too high, and does not cover the annual legal expenses which are assumed to be charged to operation.

The rental or cost of construction plant and housing of employes is a very material item in an average water works, and many contractors, after building a plant, find themselves in possession of an immense amount of plant upon which it is extremely difficult to realize. There are derricks, pile-drivers, steam shovels, pumps, boilers and pipes used in construction, cofferdams, shanties, office buildings, tram roads, cars, commissaries, wagons, dump carts and tools of all kinds, which are best allowed for in a separate item rather than to estimate such in the cost of labor and material. Two per cent is certainly not too high an allowance. Take a storage reservoir, and the writer is in charge of one at the present time which is about completed. It is done by contract and the contract price is approximately $120,000. There are steam shovels, derricks, pumps, boilers, dump wagons, drilling machines and small tools of all kinds. The entire plant account is $35,000 including horses. Without the horses and wagons the plant account is about $25,000. The contractor estimates a depreciation of 25 per cent for the job. If he can go right from this work to another job with the same plant this 25 per cent may be considered reasonable, but the best that he can do he will be subject to a loss in plant of over $6,000, which is 5 per cent of his contract price. However, in a general water works proposition the plant and tool account bears a lower proportion than on the reservoir, but it is an item that is nearly always lost sight of in expert appraisals.

Some would object to putting in this item at all on the ground that it has been allowed for in contractor's profit. In other words when the contractor is paid his 15 per cent he is supposed to furnish his own plant, but 15 per cent on the cost of labor and material is a lower price than most general contractors would be willing to accept unless they had continuously a constant volume of work, or unless the plant was paid for in addition.

Contingencies.—Every engineer who has built many water plants, after making as careful an estimate as possible of their cost in each case, finds that an allowance of 10 per cent is not excessive.

Interest During Construction.—The item of interest during construction is, of course, dependent upon the length of time required to construct the works and the method by which it is financed. Five per cent is a common allowance and as a general proposition is not excessive.

General Contractor's Profit.—The argument in favor of general contractor's profit at some such figure as 15 per cent has been covered in the body of the paper. Many engineers for various reasons prefer to figure this allowance in the cost of labor and material, and in the same manner hide other unavoidable items, but this performance will in no way affect the final result, unless it is the intention to scale down the estimate as much as possible, which the process facilitates.

Development Cost, 20 Per Cent.—In actual experience this will be more often exceeded than found in excess. It is easy enough for estimators to present a hypothetical case and scale down this cost to an amount that is away below the possibilities under the circumstances in which the plant has been built and developed, but if they could only actually go through the financial vicissitudes of a dozen or more important and ordinary cases, their viewpoint would be materially modified, and it is not a sufficient excuse to say that the "courts or commissions will not stand for such a high allowance."

Allowance for Useless Items, 3 Per Cent.—This, of course, is based on cutting out all such items found in the plant under consideration and then adding a reasonable percentage. On the whole, I am fully convinced that the allowance of 183 per cent is not only fully justified, but greatly exceeded in practice.

Successful Application of the Bonus System in Paying Meter Readers at Milwaukee, Wis.

Until about a year ago the Milwaukee Gas Light Co. paid its meter readers at a certain rate per hour, irrespective of the number of readings taken. At that time the basis of payme t to readers was changed to the plan of paying a certain rate per hour with a minimum return of a special number of readings per hour, and for readings over this minimum return a bonus is paid. The successful working of this plan at Milwaukee is here brought to the attention of water departments, in the larger of which the bonus system of paying meter readers seems equally applicable. Our information is from a paper by Mr. H. C. Schaper, chief clerk of the company named, before the American Gas Institute:

The reason for making this change was primarily to establish a system that would more equitably compensate meter readers for actual work done. Some basis of what should constitute a reasonable hour's work had been in a manner established through years of experience, although districts vary greatly, some localities offering opportunities of reading meters easily, while others are harder to get and having a smaller number to read. Furthermore, conditions are likely to change in the same district, so that an average per hour basis does not in every case truly decide whether or not a reader has done an honest hour's work.

In 1907 our company abolished separate rates for fuel and illuminating purposes and began to remove duplicate meters. It was natural that this should reduce the average returns per hour, and when this work was practically completed it was necessary to establish some new basis of an hourly return. To determine this, we picked five men at random from our reading force and placed them upon a monthly basis. They were paid $60 per month, irrespective of the number of hours required to work each day, but we did make a provision that no more than eight hours were to be put in in any one day. We had been getting an average of 25 readings per hour, but by making this change we increased this average to 30 per hour. This average was obtained not only from the monthly men but from the day men as well. After several months' trial we concluded that the average of 30 readings per hour represented a reasonable return, and this was used as a basis for our present method: a merit basis of compensation, which is as follows:

New men are paid 24 cts. per hour and with continued service this rate is gradually increased to 27 cts. per hour. For this a minimum return of 30 readings per hour is expected; so that our primary labor cost is from \$0.008 to \$0.009 per reading. For readings taken in excess of 30 per hour the men receive extra compensation at the rate of \$0.008 per reading, this rate being paid to old men and new men alike. This has made it possible to do our work with 22 men, who are in charge of one supervisor, and at an average cost per reading of \$0.0105, including the cost of a supervisor and miscellaneous expenses. Under the old system it required 26 readers, with one supervisor, at an average cost of \$0.0125 per reading.

We have in no case so far found it necessary to fine a man for not bringing in the minimum number of readings per hour, and whether we would feel inclined to do this or not I question very much, as a man who could not bring in the minimum number of readings per hour would hardly be considered competent for the work and would be discharged.

At the end of each of the three reading periods each man reads his own stragglers and retakes, for which he is paid a flat rate of \$1.25, irrespective of how many or how few he may have. Our purpose of having this flat rate is to encourage bringing in as few stragglers with the regular readings as possible, and the fewer stragglers a reader has, the easier he will earn his \$1.25. I find that men will now make a second call to get the reading at the regular period rather than bring in a report of "not at home," as it lessens his work on straggler day, besides paying him the bonus on regular reading days. As evidence, I submit the number of stragglers we had for March, 1913, and March, 1914, which were 4,536 and 2,698, respectively. Another reason, and a more material one, is to avoid the making of stragglers purposely.

Stragglers and retakes naturally are scattered, and it would be unreasonable to expect these read at the regular reading rate. If we were to pay some equitable rate per reading, and which of necessity would have to be considerably higher than the regular reading rate, the meter reader could then purposely fail to read some meters at the regular reading time and report them as stragglers, and receive the higher straggler rate of pay. With this flat rate of \$1.25 this objection is overcome.

We have approximately 200 meters that are read weekly and, as these are considerably scattered, we pay a flat rate of 30 cts. per hour. This rate is also paid when special readings are taken.

Our work is so arranged that the meter readers also deliver gas bills, and this work is likewise paid for on the merit basis. The men are paid the same as for meter reading, from 24 cts. to 27 cts. per hour, and for which a minimum delivery of 65 bills per hour is expected. For each bill delivered over this average they receive extra pay at the rate of \$0.004 per delivery. This has resulted in reducing the cost per delivery from \$0.0062 to \$0.0053.

It occasionallly occurs that we have special work for the readers which can be done in connection with their regular work, such as delivering advertising matter or making canvasses. Frequently we have advertising stickers attached to our gas bills, and for attaching these stickers we pay them 5 cts. per 100 and for delivery an additional 5 cts. per 100. Some months ago we made a canvass of large dwellings having certain types of heating apparatus, and for each prospect accepted by us the reader was paid 5 cts.

The average wages paid during the past eleven months, the period covered by the present system, has been \$61.48, compared with a previous average of \$55.21. The maximum average earned by one reader during this period was \$71.45, and the maximum for any one month by one reader was \$75.89.

Another result that we have accomplished with this system, and one that will readily commend itself, is that our men always appear busy, as it is natural that loitering will pay them nothing. To see a busy man read his meter inspires a consumer with more confidence than to see a boy, and sometimes a pair of them, fooling away part of their time.

Our readers apparently are very well satisfied with this merit system, and probably the best evidence of this is the fact that we have had no men leave us since it was inaugurated. Our men are taken from all walks of life, and we have amongst our number of readers mechanics, former school teachers, artists, musicians, ball players, etc. Whenever necessary to call in an extra man to help out we usually do so from students of the high schools, normal school or the university.

Our men are uniformed, the uniform being of approved design, and is made for us by one tailoring concern. The uniforms are charged to the men, and they pay for them in installments of \$3 per month. No objections have been raised by any of our men because they are obliged to wear uniforms, and by having them uniformed they present a neater appearance and are readily recognized by consumers as gas meter readers.

In addition to this we aim to keep our readers in the same district as long as we can, as greater confidence in the reader is gained as he becomes known to the consumer. We are opposed to any system of trying to check up the readers that will require frequent changes of reading districts, as this surely is an annoyance to the consumer. We endeavor to get reliable men and then trust them, and our trust in them has so far not been misused.

We use a coupon reading slip and cut off the previous month's reading before the slips are turned over to the readers for the next month's reading, so that no reader can tell from the slip what the previous reading was. We also get a check on the readers with our meter changes and, while this may not check up a reader for periods which may sometimes cover more than a year, the reader is constantly in the dark as to just when such a change will be made, and he will not risk his position, knowing that any time he might be found out.

Method of Testing Verticality of a Deep Well at Audubon, Iowa.

During the years 1912 and 1913 the city of Audubon, Ia., put down an artesian well 2,400 ft. deep and 14 ins. in diameter at the top. Inside the well an 8-in. drop pipe 400 ft. long was suspended. The brass well barrel or deep well pumping cylinder is hung on the lower end of this drop pipe. The pump installed in this well caused the city great annoyance and considerable expense due to frequent breaking of pump rods. The pump manufacturers contended that the well bore was crooked and so induced stresses in the rods which caused them to fail. The city maintained that the well was substantially straight and that the rods failed because they were overloaded. To determine the true condition of the well the tests here described were conducted by Prof. W. H. Meeker of the Iowa State College. Our information is from an article by P. F. Hopkins in The Iowa Engineer for November, 1914. The procedure was as follows:

The center of the well head flange was accurately located by stretching two fine cords at right angles across it. A point was then located, on the well house, 26 ft. 11¼ ins. vertically above the flange center. A three-bladed gage, which was a snug sliding fit on the inside of the 8-in. pipe, was attached to a reel of fine steel piano wire. The gage was lowered into the drop pipe, the wire leading up and through the point which had been established in the tower of the pump house above the well-head flange. From this point the wire was led over a pulley and down to a drum near the floor. A 10-lb. sash weight was hung to the gage, thus insuring a proper tension to the wire. By unwinding the drum carrying the wire, the gage could be lowered to any desired point in the drop pipe. The amount that the center of the drop pipe was out of plumb at any position could be determined by calculations based on measurements of the distance of the wire at the pump-head flange from the north and south and east and west lines, which were stretched across the face of the flange to determine its center. Readings were taken at each 11 ft. of depth. The actual readings taken from the positions of the wire at the face of the flange were recorded in inches as "top deflections." The radial deflection is the actual distance of the wire from the center of the flange. Its magnitude was the hypotenuse of the right triangle of which the two measured top coordinates were the legs. Under "total deflection" were recorded the corresponding distances of the position of the gage in the drop pipe, also in inches. These were obtained, for each position of the gage by the solution of proportions based on similar triangles. For example: at a depth of 242 ft. the "top deflections" or co-ordinates of the point where the wire intersected the plane of the top of the casing were, by actual measurement, found to be 0.75 in. and 1.06 ins., for the north and west deflections, respectively. The north deflection, x, of the center of the gage was then solved for from the following proportion:

26 ft. 11¼ ins. :0.75 in. : : 242 + 26 ft. 11¼ ins. :x.

In a similar manner the west deflection, y, was solved for by replacing the figure 0.75, in the above proportion, with the measured west top deflection of 1.06. In this case the "total deflections" x and y equaled 7.48 ins. and 10.56 ins., respectively.

Profiles of the projections of the center line of the drop pipe on east and west and north and south planes were platted after the measurements and computations were made. The horizontal scale used in plotting the profiles was very much larger than the vertical scale. On the horizontal scale 1 in. equals 20 ins., while on the vertical scale 1 in. equals 20 ft. Had the same scale been used for the vertical dimensions as for the horizontal, the profile would have been very long and the apparent sudden change in direction of the north and south profile at 150 ft. and at 194 ft., in the Audubon well, would have been so reduced as to be hardly appreciable.

At Audubon the profiles showed that the center line of the drop pipe was not absolutely plumb. The profiles, however, did not show any sudden changes in direction and the investigator concluded that there is no reason why so flexible a construction as the rods which are being used in this well should not work freely without undue binding or cramping.

Railway Ties in Canada.—A total of 19,881,714 cross-ties were purchased by the steam and electric railways of Canada during 1913, according to a Forestry Branch Bulletin. In 1912, a total of 21,308,571 ties were purchased. Of the 1913 total, 39.1 per cent were of jack pine; 12.3 per cent, white cedar; 12.2 per cent, Douglas fir; 6.2 per cent, western larch; 6 per cent, hemlock; 5.7 per cent hard pine; 4.9 per cent, oak; 4.4 per cent tamarack; and small percentages of western hemlock, spruce, chestnut, red cedar, red pine, beech, birch, maple, elm, ash and cherry.

The Welland Canal, for which all contracts have been let, is to be finished by 1918 at a cost of \$50,000,000, all of which is to be borne by the Dominion Government. At present about 2,000 men are at work excavating the northern end of the canal.

BUILDINGS

Comparative Cost Data on the Various Units of the Arizona Copper Co.'s Smelter, Clifton, Ariz.

The construction of new smelter of the Arizona Copper Co., at Clifton, Ariz., which was built at a total cost of $2,105,020.07, was started in February, 1912, and completed in February, 1914. The plant is completely equipped for the smelting of copper, and modern facilities are provided in all particulars. In our Sept. 16, 1914, issue we gave a layout of the smelter showing to scale the size and location of each unit of the plant, and described the design, construction and costs of the power house. In this issue we shall give cost data applicable to the various parts of the work, the data being arranged in such a manner as to facilitate a comparison of the costs of the various units of the plant. Data will also be given on the construction methods used and on the general features of the buildings. The article is based on a paper by E. Horton Jones, in Bulletin for July, 1914, American Institute of Mining Engineers. The "Dewey" decimal system was used to classify the cost data. The costs, as given, are actual costs and were compiled by an efficient cost keeping organization. Each cost item has been judged solely on its merits for use by an estimator. All delays due to changes in plans, delayed shipments, labor troubles, changes in working hours and rates, and variable weather conditions have been included. The cost data are given first, followed by an explanation of them. Many of the data are applicable to other kinds of plants.

Cost Data.

TOTAL COSTS OF VARIOUS UNITS.

The accompanying table gives the total cost of each unit of the plant:

Ref. No.	Item.	Total cost.
7100	Engineering expense	$ 100,649.88
7300	Yard tracks and industrial system	156,326.43
7400	Receiving bins	44,185.06
7700	Crushing plant	9,268.62
7800	Sampling plant	34,108.74
7900	Bedding plant and bunker bins	150,939.05
8100	Roasting plant	136,734.87
8120	Roaster dust chamber	49,664.76
8300	Reverberatory plant	328,945.02
8400	Converter plant	216,033.37
8420	Converter dust chamber	27,813.58
8500	Conveying system	45,411.15
8600	Chimney	45,471.34
8610	Reverberatory flue	13,453.70
8620	Converter flue	7,602.88
8625	Roaster dust chamber flue	12,859.10
8700	Boiler and blacksmith shop	21,449.23
8714	Machine and carpenter shop	27,356.27
8800	General office	1,394.95
8809	Warehouse	13,602.71
8819	Laboratory	6,144.02
8840	Sample room	2,826.11
8900	Miscellaneous accounts	37,186.48
8999	Indirect expense	140,277 72
9000	Power plant	434,703.15
9060	Oil supply sump and pump house	40,611.88
	Total cost	$2,105,020.07

Table I gives the area of each building, its total cost, and its cost per square foot of floor space. These costs are for the buildings alone.

Table II gives the volume of each building, its total cost, and its cost per cubic foot. These costs are for the buildings alone.

Table III gives the area of each building, its total costs, fully equipped, and its cost per square foot of floor space, fully equipped.

Table IV gives the volume of each building, in cubic feet, its total cost, fully equipped, and its cost per cubic foot, fully equipped.

MISCELLANEOUS COSTS.

The cost of the cooling tower per thousand gallons per minute (capacity 12,000 gals. per minute) was $2,189.42, its total cost being $26,273.01.

TABLE I.—COST OF EACH BUILDING PER SQUARE FOOT.

Ref. No.	Name of building.	Floor space, sq. ft.	Total cost.	Cost per sq. ft.
7700	Crushing plant	1,650	$ 5,968.32	$3.62
7800	Sampling plant	6,140	16,299.16	2.65
8100	Roasting plant	28,740	43,322.75	1.51
8300	Reverberatory plant	20,370	50,687.28	2.49
8307	Reverberatory boiler building	14,310	36,887.67	2.58
8400	Converter building	26,084	87,231.14	3.34
8700	Boiler and blacksmith shop	4,424	11,320.58	2.56
8714	Machine and carpenter shop	5,144	14,905.56	2.90
8809	Warehouse	5,040	11,512.93	2.28
8819	Laboratory	1,492	11,363.77	2.92
8840	Sample room	600	991.46	1.65
9000	Power plant	32,096	77,452.56	2.41

TABLE II.—COST OF EACH BUILDING PER CUBIC FOOT.

Ref. No.	Name of building.	Volume, cu. ft.	Total cost.	Cost per cu. ft.
7700	Crushing plant	27,040	$ 5,968.32	$0.22
7800	Sampling plant	80,547	16,299.16	0.20
8100	Roasting plant	410,140	43,322.75	0.11
8300	Reverberatory plant	474,350	50,687.28	0.11
8307	Reverberatory boiler building	500,850	36,887.67	0.07
8400	Converter building	1,529,636	87,231.14	0.06
8700	Boiler and blacksmith shop	86,268	11,320.58	0.15
8714	Machine and carpenter shop	100,308	14,905.56	0.15
8809	Warehouse	83,160	11,512.93	0.14
8819	Laboratory	16,140	4,363.77	0.27
8840	Sample room	6,000	991.46	0.16
9000	Power house	784,000	77,452.56	0.10

TABLE III.—COST OF EACH BUILDING EQUIPPED PER SQUARE FOOT.

Ref. No.	Name of building.	Floor space, sq. ft.	Total cost, equipped.	Cost per sq. ft., equipped.
7700	Crushing plant	1,650	$ 9,268.62	$5.62
7800	Sampling plant	6,140	34,108.74	5.56
8100	Roasting plant	28,740	136,734.87	4.76
8300	Reverberatory plant	20,370	172,171.55	8.45
8307	Reverberatory boiler building	14,310	159,716.26	11.16
8400	Converter building	26,084	216,033.37	8.28
8700	Boiler and blacksmith shop	4,424	21,449.23	4.85
8714	Machine and carpenter shop	5,144	27,356.27	5.32
8809	Warehouse	5,040	13,602.71	2.70
8819	Laboratory	1,492	6,144.02	4.12
8840	Sample room	600	2,826.11	4.71
9000	Power house	32,096	359,590.10	11.20

TABLE IV.—COST OF EACH BUILDING EQUIPPED PER CUBIC FOOT.

Ref. No.	Name of building.	Volume, cu. ft.	Total cost, equipped.	Cost per cu. ft., equipped.
7700	Crushing plant	27,040	$ 9,268.62	$0.34
7800	Sampling plant	80,547	34,108.74	0.42
8100	Roasting plant	410,140	136,734.87	0.33
8300	Reverberatory plant	474,350	172,171.55	0.36
8307	Reverberatory boiler building	500,850	159,716.26	0.32
8400	Converter building	1,529,636	216,033.37	0.14
8700	Boiler and blacksmith shop	86,286	21,449.23	0.24
8714	Machine and carpenter shop	100,308	27,356.27	0.27
8809	Warehouse	83,160	13,602.71	0.16
8819	Laboratory	16,140	6,144.02	0.38
8840	Sample room	6,000	2,826.11	0.47
9000	Power house	784,000	359,590.10	0.46

TABLE V.—TOTAL AND UNIT COSTS OF VARIOUS ITEMS OF BOILER AND BLACKSMITH SHOP.

Ref. No.	Item.	Labor cost.	Material cost.	Total cost.	Quantity.	Total unit cost.
8701	Excavation	$1,142.07	$ 44.81	$1,186.88	1,458 cu. yds.	$ 0.81
8702	Foundation	416.57	584.49	1,001.06	78.7 cu. yds.	12.71
8703	Steel structure	....	....	2,913.90	32.72 tons	89.06
.1	Doors, windows and frames	693.02	2,456.28	3,149.30	2,581 sq. ft. opening	1.22
.11	Concrete sills	119.80	50.87	170.67	251.5 lin. ft.	0.68
.2	Tile walls	477.95	612.62	1,090.57	2,297 cu. ft.	0.48
.21	Unloading tile	18.89	....	18.89	69.70 tons	0.27
.22	Coping	112.17	2.72	114.89	290 lin. ft.	0.40
.30	Roof	286 52	828.24	1,114.76	66.49 squares	16.77
8703.31	Ventilators	16.01	261.50	277.51	3 vents	92.50
.40	Dirt floor	59.73	1.25	60.98	....	.....
.50	Benches	87.83	49.43	137.26	....	.....
.60	Painting	92.53	60.73	153.26	1,574 sq. yds.	0.10
8704	Crane	119.60	438.41	558.01	1 crane	558.01
8705	Tools	798.51	7,859.36	8,655.87	....	.....
8706	Shafting, pulleys, belting	105.59	301.16	406.75	51 lin. ft.	7.98
8707	Motor	23.22	347.54	370.76	20 HP.	18.54
8708	Lighting	23.41	44.50	67.91	17 drops	4.00
	Total cost—Boiler and blacksmith shop			$21,449.23		

TABLE VI.—TOTAL AND UNIT COSTS OF VARIOUS ITEMS OF MACHINE AND CARPENTER SHOP.

Ref. No.	Item.	Labor cost.	Material cost.	Total cost.	Quantity.	Total unit cost.
8715	Excavation	$1,615.83	$ 325.28	$1,941.11	1,765 cu. yds.	$ 1.10
8716	Foundation	792.05	584.06	1,376.11	105.5 cu. yds.	13.04
8717	Steel structure	....	....	3,431.42	38.23 tons	89.76
.1	Doors, windows and frames	923.61	2,992.16	3,915.77	3,037 sq. ft. opening	1.29
.11	Concrete sills	111.65	67.70	179.35	295.3 lin. ft.	0.61
.2	Tile walls	531.45	571.28	1,102.73	2,397 cu. ft.	0.46
.21	Unloading tile	42.06	2.00	44.06	58.80 tons	0.75
.22	Wall coping	121.67	23.70	145.37	320 lin. ft.	0.45
.30	Roof	297.85	953.04	1,250.89	77.21 squares	16.20
.31	Ventilators	11.16	248.24	259.40	3 ventilators	86.45
.40	Floor	269.80	593.30	863.10	4,136 sq. ft.	0.21
.50	Benches	130.00	35.00	165.00	....	.....
.60	Painting	118.00	87.40	205.40	1,989 sq. yds.	0.10
8718	Crane	25.19	564.36	589.55	....	.....
8719	Tools	444.07	8,953.13	9,397.20	....	.....
8720	Shafting, pulleys and belting	289.29	1,513.36	1,802.65	152 lin. ft.	11.86
8721	Motor	18.34	477.97	496.31	40 HP.	12.40
8722	Lighting	55.84	135.01	190.85	20 drops	9.54
	Total cost—Machine and carpenter shop			$27,356.27		

The cost of the power plant, including boiler plant, per indicated horse power (capacity 10,660 I. HP.) was $55.32, its total cost being $589,717.16. The capacity, indicated horse power, of the three turbines was 9,460; that of the two Nordberg blowers (see Sept. 16, 1914, issue), 1,000; and that of the single air compressor, 200.

The cost of the power plant, exclusive of boiler plant, per indicated horse power, was $37.40, its total cost being $398,631.17.

The cost of the boiler plant per boiler horse power (capacity 6,143 HP.) was $31.11, its total cost being $191,085.99. The total capacity is given by seven waste heat units at 713 HP. each and three oil-fired units at 384 HP. each.

DETAILED COSTS OF BOILER AND BLACKSMITH SHOP, MACHINE AND CARPENTER SHOP, WAREHOUSE, AND CHIMNEY.

The detailed costs of some of the buildings, which are applicable to other kinds of plants, will now be given. The detailed costs of the power plant were given in our Sept. 16, 1914, issue.

Boiler and Blacksmith Shop.—Table V gives the separate costs of the various labor and material items, the total costs of these items, the actual quantities of materials, and the unit costs for the boiler and blacksmith shop.

Machine and Carpenter Shop.—Table VI gives the separate costs of the various labor and material items of the machine and carpenter shop, the total costs of these items, the actual quantities of materials, and the unit costs.

Warehouse.—Table VII gives the separate costs of the various labor and material items of the warehouse, the total costs of these items, the actual quantities of materials, and the unit costs.

Chimney.—The large chimney of the smelter plant is 300 ft. high, 26 ft. 8 ins. inside diameter at the base and 22 ft. at the top. The average thickness of the walls is 24½ ins. The chimney is corbeled out every 25 ft. to hold the lining of radial perforated fire brick laid in acid-proof mortar. The foundation of the chimney is constructed of concrete, the base of red brick, and the chimney proper of radial blocks. The upper 75 ft. of the chimney are pointed with acid-proof mortar. Table VIII gives the total and unit costs of the excavation, the foundation and the chimney proper.

The excavation for the foundation was a deep hexagonal cut through clay and caliche, and penetrating a considerable distance into sand and gravel containing large boulders. The material was loosened with picks, slipped out with fresnos, dumped through a trap into carts, and hauled 2,700 ft.

The foundation consists of a concrete block, cast in a hexagonal shape, with a depth of 20 ft. and a least diameter of 50 ft. The bottom of the block is reinforced with three layers of 1-in. rods spaced 1 ft. on centers. The concrete was machine mixed and consists of 1 part cement to 8 parts sand and gravel, with large stones embedded in it. Forms were built for about 40 per cent of the vertical surface. The concrete was transported about 100 ft. in cars.

The chimney proper was constructed by the Alphons Custodis Chimney Construction Co., the costs given including constant inspection by the Arizona Copper Co. There were used in the construction of the chimney proper 138,000 lbs. of lime, 290 lbs. cement, 1,638 tons of radial brick, 652 tons of wire-cut brick, 56 tons of wedge brick, and 100 bbls. of acid-proof mortar.

Data Explanatory of Cost Data.

WAGE SCALE.

As the construction work extended over a period of two years the rates of wage for various classes of labor changed somewhat. Table IX gives the labor scale throughout the job. It will be noted that some Mexican labor was used.

BOILER AND BLACKSMITH SHOP (SEE TABLE V.).

The following data give a good idea of the type of construction used in the boiler and blacksmith shop, the materials used, and the conditions under which the work was done. The steel frame consists of Fink trusses on 10-in. steel columns. The span of the trusses is 48 ft., and the clear height of the building is 20 ft. The building has 8-in. tile walls.

Account 8701—Excavation.—The excavation involved making a 6-ft. slice to get the proper grade for the building site, together with piers and small wall excavation. It was plowed and slipped away in fresnos 400 ft.

8702—Foundations.—These foundations were the small walls and piers for the brick and steel column supports. The concrete was plain, hand mixed in the proportions of 6 sand and gravel to 1 cement, and it was handled 100 ft. in wheelbarrows to the forms. Fifty per cent of the vertical surface was formed. This was the first concrete cast at the smelter.

8703—Steel Structure.—There were 32.72 tons of structural steel used in the framework of the building, consisting of 48-ft. span Fink trusses on 8-in. columns.

8703.1—Doors, Windows and Frames.—This

TABLE VII.—TOTAL AND UNIT COSTS OF VARIOUS ITEMS OF WAREHOUSE.

Ref. No.	Item.	Labor cost.	Material cost.	Total cost.	Quantity.	Total unit cost.
8810	Excavation	$944.59	$ 51.49	$ 996.08	1,287 cu. yds.	$ 0.77
8811	Foundation	878.16	856.09	1,734.25	123 cu. yds.	14.09
8812	Steel structure	....	....	3,734.08	39.76 tons	93.92
.1	Doors, windows and frames	533.02	1,056.31	1,589.33	1,982 sq. ft. opening	0.80
.11	Concrete sills	164.72	61.63	226.35	241.5 lin. ft.	0.94
.2	Tile walls	438.00	477.86	915.86	2,342 cu. ft.	0.39
.21	Unloading tile	15.50	1.00	16.50	74.20 tons	0.22
.22	Coping	176.60	36.58	213.13	320 lin. ft.	0.67
.3	Painting roof	81.16	65.66	146.82	813 sq. ft.	0.18
.31	Ventilators	30.38	207.12	237.50	3 vents	79.17
.4	Floor excavation	129.03	....	129.03	66 cu. yds.	1.96
.41	Floor concrete	558.04	721.60	1,279.64	8,298 sq. ft.	0.15
.5	Lighting	45.09	70.48	115.57	26 drops	4.45
8813	Warehouse fixtures	548.66	1,541.12	2,089.78	....	.....
.1	Painting	26.50	14.17	40.67	412 sq. yds.	0.10
.11	Painting sash	122.78	15.34	138.12	189 sash	0.73
	Total cost—Warehouse			$13,602.71		

TABLE VIII.—TOTAL AND UNIT COSTS OF CHIMNEY.

Ref. No.	Item.	Labor cost.	Material cost.	Total cost.	Quantity.	Total unit cost.
8601	Excavation	$337.44	$ 29.61	$ 367.05	597 cu. yds.	$0.61
8602	Foundation	654.42	4,199.65	4,854.07	872.7 cu. yds.	5.56
8603	Chimney proper	891.88	39,358.34	40,250.22	58,644 cu. ft.	0.69
	Total cost			$45,471.34		

TABLE IX.—WAGE SCALE DURING CONSTRUCTION OF SMELTER.

Occupation.	Feb. 28, 1912 Ten hours. A.	M.	April 1, 1912. Nine hours. A.	M.	July 24, 1912. Nine hours. A.	M.	July 1, 1913. Nine hours. A.	M.	Sept. 1, 1913. Eight hours. A.	M.
Blacksmiths	$....	$2.50	$4.00	$....	$4.50	$....	$4.50	$....	$4.25	$....
									4.50	
Blacksmiths' helpers				2.25	3.00	3.00	2.50	2.50		
							3.00	3.00	2.50	2.50
							3.50		3.25	3.25
Boilermaker boss							5.50		5.50	
Boilermakers					4.50		4.50		4.25	
Boilermakers' layer-out							4.75		4.75	
Boilermakers' helpers					3.00		3.00	3.00	3.00	3.00
Brick masons							*6.50		*6.50	
Brick masons' tenders								*2.25		*2.25
Carpenter boss					5.00		5.00		5.00	
Carpenters, first-class	[illegible]		4.00		4.50		4.50		4.25	
Carpenters, second-class	4.00	4.00								
Carpenters' helpers	3.00	3.00	3.00	3.00	3.00		3.00		2.50	
Carpenters (with tools)					3.50		3.50		3.00	
Carpenters' helpers				2.25						
Cart drivers					2.25	2.25				
Cement finisher boss									5.00	
Cement finisher boss					4.50			2.50		
							3.50	3.50		
							4.00	4.00		
Concrete boss					4.50		4.50			
Concrete mixers				2.25		2.25		2.25		
Corral boss			3.50		3.50		3.50		3.50	
Corral men					2.25	2.25				
Drillers		2.25		2.00						
Electrician boss							5.00		4.50	
									5.00	
Electrician							4.50		4.25	
Electrician helpers							3.00	2.25	2.75	
									3.00	
Engineers, locomotive	3.25									
Engineers, compressors					3.50		3.50		3.50	
Engineers, stationary					3.00					
Janitors				2.00		2.25				
Labor bosses			3.25	2.00	4.00		4.00		4.00	
			4.00	2.50			4.50			
				3.00						1.75
Laborers		1.75		1.75		2.00		2.00		2.00
Machine shop boss							5.00		5.00	
Machine shop boss (outside)							5.50			
Machinist			4.00		4.50		4.50		4.25	
Machinist helpers			3.00		3.00	3.00	3.00	2.50	3.00	3.00
Miners, underground				*2.75		2.75				
Miners, surface				2.25		2.25		2.00		†2.00
Office boys										2.25
Pipe fitter boss							5.50		5.50	
Pipe fitter		2.50					4.50		4.25	
Pipe fitter helpers							3.00	2.25	3.00	
Plow holders			2.25	2.25						
Plumbers					4.50					
Plumbers' helpers					3.00					
Rigger boss			4.50		5.00		3.50		4.25	
Rigger							4.50			
Rigger helpers			2.25	2.25			2.25	2.25	2.25	2.25
							2.50	2.50	3.50	3.50
							3.00	3.00		
Stone mason boss				3.50						
Stone mason		2.00		3.00		3.00				
Steam fitters					4.50					
Steam fitters' helpers					3.00					
Teamsters, 4 and 6 head			3.00		3.00					
Teamsters, 2 head			[illegible]	2.00	2.25	2.25	3.00		3.00	
Teamsters, fresnos and slips		2.25	2.00	2.00	2.50	2.50				
Teamsters, plow	2.75		2.25	2.25						
Tinners					4.00		4.50		4.25	
Tinners' helpers					2.50	2.50			3.00	
Tool room man				2.00	2.25					
Tool sharpeners				2.50	3.50		4.00			
Track boss					4.25					
Water boys		1.50		1.00	2.00	2.00	2.00	2.00	1.75	1.75
Warehouse help							3.00	2.50	3.00	

A—American. M—Mexican *Eight hours. †Nine hours.

account covers the purchase price of all doors, windows, their frames, lintels and glass. It also covers the labor of installing the steel lintels which run from building column to building column; the erection of the steel door and window frames; the erection of the steel sash and doors; and the glazing of these doors and windows. After the lintels had been framed in and the tile work brought up to the sill base and the sill set, the frames were put in place, bolted to the lintels, and tied by rods back to the building columns. When the frames had been entirely bricked in, the steel sashes were bolted in place and later glazed. A segregated material list is as follows:

13 11-ft. 7-in. x 12-ft. ⅞-in. steel sash, 63 lights, 2 mullions, with 3 to 6 light ventilators, not glazed.
1 10-ft. 3-in. x 12-ft. ⅞-in. steel sash, 56 lights, 1 mullion, no ventilators, not glazed.
2 10-ft. 3-in. x 12-ft. ⅞-in. steel sash, 48 lights, 1 mullion, no ventilators, not glazed.
1 4x9-ft. steel sliding door, with six 14x20-in. lights, not glazed, lower panels steel plate.
1 8x9 ft. steel sliding door, with eighteen 14x20-in. lights, not glazed, lower panels steel plate.
1 14x20-ft. Kinner steel rolling door.
1 10x10-ft. Kinner steel rolling door.
11 14-ft. 10-in. lintels built up of 8-in. channels.
2 13-ft. 4-in. lintels built up of 8-in. channels.
2 11-ft. 6-in. lintels built up of 8-in. channels.
1 10-ft. 4-in. lintels built up of 8-in. channels.
1 11-ft. 6-in. lintels built up of 8-in. channels.
[illegible] lights, 14x20 ins., ⅛-in. factory ribbed glass.
164 lights, 13⅛x19⅛ ins., factory ribbed glass.
82 lights, 14x19⅛ ins., factory ribbed glass.
44 lights, 13½x20 ins., factory ribbed glass.
Steel windows and door frames for above made of two 3½x2½x¼-in. angles.

8703.11—Concrete Sills.—This account covers the labor and material used to make the following list of concrete sills. The sills were made 3 parts sand and gravel to 1 cement, cast in collapsible moulds and later finished. Three ⅝-in. rods are used in each sill.

11 sills, 8¼x10 ins., 14 ft. 10 ins. long.
1 sill, 8¼x10 ins., 11 ft. 6 ins. long.
1 sill, 8¼x10 ins., 8 ft. 6 ins. long.
2 sills, 8¼x10 ins., 12 ft. 2 ins. long.

8703.2—Tile Walls.—This cost includes the cost of tile, mortar and scaffolds, together with the mason and carpenter labor used to build the walls. The walls were non-bearing 8 ins. thick, built of hollow tile, laid in between the steel building columns. The mortar used was 1 cement, 1 lime and 1 sand.

8703.21—Unloading Tile.—This covers the cost of preparing site, unloading, and checking quantity of tile.

8703.22.—Coping.—This covers the cost of labor and material incident to coping the walls at the top, beneath the roof. A 2x4-in. piece was bolted to the top course of tile and another to the underside of the roof. These were lathed across with metal lath and plastered with cement mortar.

8703.30.—Roof.—This account covers the cost of the material and labor incident to roofing the boiler and blacksmith shop. Oregon pine sheathing, 2x8-in., surfaced, tongued and grooved, was nailed to strips bolted to the purlins. Over this three-ply asbestos roofing paper was laid.

8703.31.—Ventilators.—This covers the cost of labor and material incident to installing three 48-in. "Burt" ventilators on the peak of the boiler and blacksmith shop roof. The ventilators were skidded up onto the roof with hand tackle along a runway, bolted to the purlins and flashed.

8703.4—Dirt Floor.—This account covers the labor incident to bringing the dirt floor of this building to the required grade. The dirt was wheeled in and tamped in 3-in. layers.

8703.5—Benches.—This account covers the labor and material of making from time to time benches, racks and the like used in this shop.

8703.6—Painting.—This covers the cost of painting all the steel sash one coat of "turkey red," and the woodwork, namely, the under side of the roof, two coats of white lead and linseed oil, cream color.

8704.—Crane.—This covers the purchase of the crane listed below, the labor of overhauling and erecting it.

One 3-ton hand-power traveling crane, chain block transfer type, 18-ft. span, complete with roller bushed geared trolley and provided with 3-ton triplex chain block for 13-ft. lift	$378.35
Miscellaneous	60.06
Total	$438.41

8705—Tools.—This account covers the purchase price of the tools enumerated below and the labor required to install them.

	Factory.	Freight.	Clifton.
1 No. 2 punch and shear, Hilles & Jones	$1,530.00	$435.00	$1,965.00
1 No. 0 bending rolls	580.00	75.00	655.00
1 1,100-lb. steam hammer, Niles-Bement-Pond Co., 1 blower, size 5, type D, American Blower Co.	1,015.00	408.00	1,423.00
1 5-hp. 440-volt, 3-phase, 60-cycle, 1,720-r.p.m. motor	160.00	19.90	179.90
1 No. 5 swage block	......	.....	35.08
1 Peter Wright anvil, weight 497 lbs.	..	.....	70.57
10-in. galv. iron pipe and connections	.	.....	106.63
3 sheets steel, ⅛ in. x 48 in. by 120 in.	......	.....	16.02
1 2-in. heading, upsetting and forging machine, Acme Machinery Co.	2,790.00	440.70	3,230.70
1 Sisco anvil, 407 lbs.	......	.....	46.60
1 Hay Budden anvil, 420 lbs.	......	.....	48.10
40 ft. of 6-in. I-beam	......	.....	12.62
Castings	......	....	41.00
Miscellaneous	......		29.14
Total			$7,859.36

8706—Shafting, Pulleys, Belting.—This account covers the purchase price of the list of material below and the labor of installing the same, and the necessary wooden bridge trees.

33 ft. of 2 7/16-in. and 18 ft. of 2 15/16-in. shafting.
5 pulleys, varying from 26-in. to 52-in., with bearings and hangers.
1 length of 8-in. double leather belt 104 ft. long.
1 length of 6-in. double leather belt 140 ft. long.

8707—Motor.—This account covers the purchase price of the material below and the labor of installing it. This motor furnished the power for the boiler and blacksmith shops.

One 20-HP., 440-volt, 3-phase, 60-cycle, 850-r.p.m. motor.

8708—Lighting.—This account covers the cost of the material below and the labor of its installation.

14 carbon lamps, 16 c.p.
3 tungstens, 250 watt.
260 ft. brewery cord.
300 ft. No. 12 wire.
100 ft. conduit.

MACHINE AND CARPENTER SHOP (SEE TABLE VI).

8715 and 8716—Excavation and Foundation.—Same as 8701 and 8702, respectively.

8717—Steel Structure.—This building is the same as boiler and blacksmith shop. There were used here 38.23 tons of structural steel.

8717.1—Doors, Windows and Frames.—This account is the same as 8703.10, with the following list of material:

13 11-ft. 7-in. x 12-ft. ⅞-in. steel sash, 63 lights, 2 mullions, with 3 to 6 light ventilators, unglazed.
2 10-ft. 3-in. x 12-ft. ⅞-in. steel sash, 56 lights, 1 mullion, no ventilators, unglazed.
2 10-ft. 3-in. x 12-ft. ⅞-in. steel sash, 48 lights, 1 mullion, no ventilators, unglazed.
2 4x9-ft. steel sliding doors, with six 14x20-in. lights, unglazed, lower panels steel.
1 11x12 ft. steel sliding door, with forty 14x20-in. lights, unglazed, lower panels steel.
2 14x20-ft. Kinner steel rolling doors.
14 14-ft. 10-in. lintels, made of 8-in. channels.
6 11-ft. 6-in. lintels, made of 8-in. channels.
740 14x20-in., ⅛-in. thick, factory ribbed glass window panes.
[illegible] 14x10-in., ⅛-in. thick, factory ribbed glass window panes.
20 15x20-in., ⅛-in. thick, factory ribbed glass window panes.

Steel window and door frames for the above list.

8717.11—Concrete Sills.—Same as account 8703.11, but the following product:

13 8¼x10-in., 14-ft. sills.
2 8¼x10-in., 11-ft. 6-in. sills.
2 8¼x10-in., 6-ft. 6-in. sills.

8717.20, 8717.21, 8717.22, 8717.30 and 8717.31.—These accounts, which are tile walls, unloading tile, wall coping and roof, are the same as accounts 8703.20, 8703.21, 8703.22, and 8703.30 and 8703.31, respectively. The roof contains 77.21 squares, equal to 14,543 ft. B. M. of 2x8-in. lumber.

8717.40—Floor.—This account covers the cost of the material and labor required to lay this floor. The 6x8-in. stringers were laid 2 ft. 6 ins. on centers with earth tamped in between them. On the stringers No. 3 grade, 3x12-in. white cedar planking of various lengths was spiked.

8717.50 and 8717.60—Benches and Painting.—Same as accounts 8703.50 and 8703.60, respectively.

8718—Crane.—This is the same as account 8704, with the exception that the crane here used is of 5-ton capacity.

8719—Tools.—This account covers the purchase price of all the material listed below and the labor cost of installing it:

	Factory.	Freight.	Clifton
1 Prentiss machine bench vise, No. 2	......	......	$ 20.15
1 machine bench vise, No. 21	.	....	20.16
1 machine bench vise, No. 22	......	......	28.85
1 machine pipe vise, No. 2A	. .	. . . .	2.38
1 machine pipe vise, No. 4A	......	......	7.77
1 stationary bench vise, No. 56	......	......	20.72
40 ft. of 1½-in. pipe	......	......	2.97
1 No. 48 power grindstone		.. ..	56.62
2 emery wheels	......	......	8.90
1 emery wheel grinder	......	......	17.90
1 No. 40 special turning machine	......	......	36.22
1 set faces for wiring machine	......	......	5.56
1 gauge	......	......	2.35
1 burr machine and stand	......	......	9.92
1 No. 17 S. P. crimper and stand	......	......	10.77
1 No. 3 beading machine	......	......	26.79
1 No. 0236 squaring shears	......	......	180.86
1 stake-holder and stakes	......	......	42.15
1 rivet set	......	......	2.65
1 No. 101 tinner's rule	......	......	2.73
1 power hack saw No. 3	......	......	29.63
1 radial drill press, 42-in.	......	......	752.20
Miscellaneous	......	......	21.92
1 50-in. cornice brake	......	......	155.96
1 16-in. rip saw	......	......	4.30
Castings	......	......	10.10
1 No. 1 drill chuck	......	......	5.61
1 No. 2½ drill chuck	......	......	7.02
72 hack saw blades	......	......	5.55
1 surfacer, 20 in. x 6 in.	$180.00	$26.70	206.70
1 No. 50 hand saw	175.00	27.45	202.45
1 lathe, 14 in. x 8 ft.	563.75	81.40	645.15
1 lathe, McCabe patented double spindle	2,111.00	277.15	2,388.15
1 Crescent saw table	168.75	51.34	220.09
1 20-in. Rockford shaper	425.00	175.07	600.07
1 2-in. bolt cutter	355.00	47.10	402.10
1 Crane pipe machine, 2-in.	192.00	16.56	208.56
1 Crane pipe machine, 4-in.	480.00	44.10	524.10
1 Crane pipe machine, 12-in.	1,500.00	163.59	1,663.59
Small tools, miscellaneous equipment	......	......	349.36
Total			$8,953.13

8720—Shafting, Pulleys and Belting.—This account covers the purchase price of the material below and its cost of installation:

1 pc. 30 ft., 2 15/16 in. diameter shafting.
1 pc. 60 ft. 2 15/16 in. diameter shafting.
1 pc. 18 ft., 2 15/16 in. diameter shafting.
1 pc. 18 ft., 2 7/16 in. diameter shafting.
1 pc., 22 ft., 2 7/16 in. diameter shafting.
1 pc. 10 ft., 2 7/16 in. diameter shafting.
1 pc. 4 ft., 2 7/16 in. diameter shafting.

Many pulleys ranging from 10 ins. to 68 ins. in diameter, with necessary hangers, collars, boxes, etc., were used.

8721—Motor.—This account covers the purchase price and cost of installing the following motor:

One 40-HP., 440-volt, 3-phase, 60-cycle, 850-r.p.m. motor.

8722—Lighting.—This covers the cost of the following material and the labor of installing it:

17 carbon lamps, 16 c.p.
2 tungstens, 250 watt.
240 ft. brewery cord.
100 ft. conduit.
360 ft. No. 12 weatherproof wire.

WAREHOUSE (SEE TABLE VII).

This building is the same as boiler and blacksmith shop, except that a corrugated iron roof was used. There were 26.5 tons of structural steel and 13.26 tons of corrugated iron used. All accounts except those given below are the same as for the boiler and blacksmith shop.

8812.1—Doors, Windows and Frames.—The doors and all lintels for the warehouse were similar to those for the boiler and blacksmith shop. The small doors and all window frames were wood. This account covers the cost of the door and window material listed below and the labor of installing the same.

51 windows, 3 ft. 9⅞ ins. x 7 ft. 8¾ ins. x 1⅝ ins. (these were grouped 16 in triple frames), all glazed	$ 305.99
16 wood frames for 48 of above windows	170.32
Lumber for 3 window frames, all door frames and all hardware	83.78
1 O. G. 1-light glazed door, 3 ft. 6 ins. x 7 ft. 1⅜ ins.	7.91
2 9-ft. 10-in. x 7 ft. 6½-in. Kinner rolling doors	157.30
Steel lintels	331.01
Total	$1,056.31

8812.3—Painting Roof.—This covers the labor and material for painting the underside of the corrugated iron roof two coats of lead and linseed oil, cream color.

8812.40—Floor Excavation.—This entailed cutting down the front in the warehouse 6 to 8 ins. and backfilling in places.

8812.41—Floor Concrete.—This concrete floor was cast in large 6-ft. to 8-ft. blocks, 4 ins. thick, with sand joints between blocks. The concrete was hand mixed in the proportions of 6 sand and gravel to 1 cement. It was transported in wheelbarrows 100 ft. The top finish, ¾ in. thick, was composed of 2 sand to 1 cement. This top was troweled smooth.

8812.50—Lighting.—This account covers the cost of the following material and the labor of installation:

26 carbon lamps, 16 c.p.
365 ft. brewery cord.
170 ft. No. 12 weatherproof wire

8813—Fixtures.—This account covers the purchase price of the steel bins, shelving, counter scales, office partition and furniture, as listed below; also the erection cost:

197 ft. Berger's sectional steel bins and shelving (bins received knocked down, gage of material 16 to 20)	$1,116.82
1 No. 1046 dormant warehouse scales, weighing 5,000 lbs. to ½ lb.	141.42
Furniture, material for office partition, etc.	282.88
Total	$1,541.12

8813.10—Painting.—This account covers the cost of material and labor of miscellaneous painting at the warehouse. The steel doors were given one coat of turkey red. The iron lintels were given one coat of lamp black in linseed oil. The counter was stained and oiled.

8813.11—Painting Sash.—This account covers the cost of material and labor used to paint all the warehouse sash. They were given two coats of white lead and linseed oil, cream color.

The Design of Railroad Ice Storage Houses.

There are few published data on the design, construction and effectiveness of different types of ice storage houses. There also is considerable misunderstanding as to the relative efficiencies of various systems of insulation. To meet a need for data on railroad ice storage houses the American Railway Bridge and Building Association appointed a committee to investigate and report upon this subject, the following data being taken from the comprehensive report of this committee. Although the report refers particularly to railroad ice storage houses most of the data are applicable to ice storage houses in general. A general discussion of the subject, based on the data received from the railroad companies, follows:

The conditions governing the supply and demand of ice are probably different from those pertaining to any other material used in railroad operation. During a very considerable portion of the year the demand is largely in excess of the average, while during the remaining months the demand is always less than the average, usually very slight, and sometimes there is no demand at all. Considerable supplies are needed during the hot months at points far removed from points of supply and it is of greatest advantage in such cases to transport the ice during the winter months in order that the loss during transmission may be minimized and the ice houses filled at a time when they have been cooled by the low temperature of the air.

In the case of natural ice, advantage must be taken of the seasons, and a supply must be obtained when available during the winter, being distributed at that time to fill the ice houses at the various points where a demand will arise during the following months. In the case of artificial ice only small houses at outlying points must be supplied by ice hauled over a considerable distance, as points where a large amount of ice is used can be supplied by the installation of an ice plant.

The points of supply of natural ice are controlled by nature, and in the use of natural ice all other things must be made subordinate to its availability. The supply of artificial ice, however, can be controlled by artificial means, both as to amount and location. To keep the ice machinery going with reasonable uniformity the surplus supply in the winter months is usually stored.

The demand is controlled entirely by the use that is made of the ice. At points where ice is used only for supplying local needs, the demand is fairly uniform through the months of any season, but less, of course, in the winter season than in the summer; this, however, makes the smallest demand, as the greatest demand is created by the handling of perishable freight, such as fruit, meats, etc. For the latter purpose supplies of ice must be furnished at points controlled by the location, traffic and operating conditions existing on each railroad. In some cases, as in the handling of meat, the number of refrigerator cars that must be iced continues fairly uniform throughout the year, the amount of ice required, of course, being greater in summer than in winter, while in other cases, as in the handling of fruit, there is practically no demand through many months of the year, but there is a great demand during a short period. These features must be studied in order that the house may be designed to meet the conditions to which it will be subjected.

TYPE OF CONSTRUCTION.

The type of construction which can be followed is merely a question of expense, one large item of expense being the cost of construction, maintenance and operation of the house, and the other large item of expense the loss suffered through shrinkage of the ice. The problem is somewhat different from that encountered by cold storage companies that make a specialty of storing perishable goods of considerable value. In the latter case the most modern methods of construction and means of refrigeration must be employed in order to guard absolutely against a failure of the plant that would cause a tremendous loss through the decay of the perishable materials. In the case of railroad ice houses, however, the only thing that is stored is the ice, and defects in construction, failure in the plant, etc., will merely result in the loss of ice, which, while not so serious as the loss of perishable goods of greater value, is still so important as to necessitate the most careful construction at reasonable expense.

The percentage of shrinkage in stored ice is dependent on the efficiency of the insulation of the ice house and can be reduced, but not entirely eliminated, by proper insulation. Where only natural ice is stored, and where artificial ice is stored at points removed from the refrigerating plant, it is necessary that all reasonable precautions be taken to keep the shrinkage at a minimum.

At points where refrigerating plants are operated it is possible to employ the additional precaution of keeping the temperature of the house below freezing by equipping each room within the ice house with cooling pipes that will keep the temperature so low as to prevent entirely the melting of the stored ice. The amount of power that must be applied to prevent the melting of the stored ice will be controlled largely by the type and efficiency of the insulation of the house. In any event it is important to construct ice houses so that the passage of heat from the outside to the inside of the house will be reduced to a minimum.

Many types of construction have been followed, and great loss has been suffered on account of improper understanding of the conditions to be met. In some cases it has been considered sufficient to store ice directly on the ground and build four walls and a roof over it, decreasing the shrinkage by covering the ice with hay, sawdust or other similar materials. The extent to which such practice can be followed and the amount of shrinkage that may be suffered depends entirely on the cost and the supply of ice and the condition of its use. The cheapest type of construction will probably suffice at northern points immediately adjoining large lakes that freeze over every winter, where the cost of ice is very small; while in hot southern countries, where natural ice must be hauled in during the winter, or artificial ice manufactured at all times, very expensive types of construction are justified.

INSULATION.

It is not generally understood that wood is a better insulator than brick or concrete and the problem of insulation cannot be solved by merely changing from a wood frame to brick or concrete construction without employing other means of insulation. Brick or concrete walls will prevent the circulation of air to a greater extent than some types of frame construction, but it is entirely possible to build wood frame ice houses so that the walls will resist the circulation of air through them just as well as brick or concrete walls. In either case, insulation must be provided regardless of the type of construction; and when it is considered that brick and concrete houses cost from $4 to $6 per ton of ice storage capacity as compared with well constructed and insulated wooden houses costing $2 to $3 per ton, it is questionable if, under all conditions, the more expensive type of construction is justified.

When brick or concrete houses are constructed the walls are of ordinary construction, but in addition their interior surfaces must be thoroughly insulated to provide against the passage of heat from the warm exterior into the storage rooms, and this insulation can usually be provided in a wood frame house at no greater cost than in a brick or concrete house. At the present time the wood frame ice house is universally used, there being only a very few exceptions in the way of brick and concrete houses.

It is most important to prevent absolutely the circulation of air within the house, for if any circulation occurs the warm air will continually come in contact with the ice and cause its rapid shrinkage, while if the air is kept quiet, either by confining it to small spaces or filling it with porous materials, it is the best insulation that can be found.

FOUNDATION DETAILS.

A feature that has been given only slight consideration is the insulation of the floor. A great majority of the designers have thought that it would be sufficient to level off the ground and place the ice directly upon it, sometimes covering the surface with cinders, boards or concrete, but it has not been uniformly recognized that the floor must be thoroughly insulated. Heavy losses due to shrinkage at the floor line in a number of houses indicate that the stored ice will not overcome the ground heat which continually rises to the surface. In more recent houses the shrinkage from this cause has been largely reduced by providing air spaces between the ice and the ground, usually by placing joists and covering them with a slat floor for the support of the ice. These air spaces are from 6 ins. to 12 ins. between the bottom of the ice and the surface of the ground. With other construction, so handled as to keep this air stationary, this forms a good bottom insulation.

The foundations for the walls may follow the usual types of construction with proper regard to insulation. Foundations of concrete, brick, wood blocking, posts, pile stubs and other designs are much used. Especially where masonry foundations are used care must be taken to secure proper insulation. A concrete foundation wall projecting 2 or 3 ft. above the ground line, without inside insulation, has been known to be the cause of heavy losses by the transmission of heat from the ground and outer air through the walls to the air near the floor, which rises and comes in contact with the ice.

In porous soils the water from the melted ice is permitted to go directly into the ground, but at other points drains must be installed. These are no different from other drains, but special precautions must be taken to trap them so securely that no air currents can find their way into the houses through them. The amount of air entering a room containing 1,000 tons of ice through a 4-in. pipe can easily melt all the ice in a single season.

SIZE AND CONSTRUCTION.

The information received from many sources indicates that ice storage rooms vary from 50 to 1,500 tons, the average being about 1,000 tons, the usual dimensions being 30 ft. wide, 40 ft. deep and 30 ft. high, although many other proportions are used.

While many different arrangements, thicknesses and designs of wall construction are used they all follow the same general trend; i. e., vertical studs 6 ins. to 12 ins. wide, covered on both sides with wood sheeting, building paper, etc. The best construction requires double sheeting both inside and outside, with waterproof building paper between the layers, the spaces between the sheeting from stud to stud being filled with insulating material.

It was formerly thought that air spaces afforded excellent insulation, but the manufacturers of insulating materials have done their best to explode that theory. It is claimed that air in a space 8 ins. or 10 ins. wide, for example (although theoretically thinner spaces can have the same result), will be set in circulation by the air against the outer wall becoming warm and rising while that near the inner wall is cooler and falls. The difference in temperature of the air at the two sides of the space can certainly be great, as air is a poor conductor of heat and it is evident that the circulation, which will rapidly transfer the heat which penetrates the outer sheeting to the inner sheeting, must be prevented. This can be done by cutting such spaces into smaller spaces by proper framing or by filling them with suitable porous insulating materials, which will fill the spaces yet leave the greater part of the space, consisting of pores in the insulating materials composed of air, so separated as to permit no circulation.

INSULATING MATERIALS.

In the past it was the universal custom to fill the spaces with sawdust or ground cork, but on account of the continual dampness the sawdust decayed rapidly and brought about the decay of the timber enclosing it, which was also true of crushed cork. At the present time various materials, which are specially treated and manufactured for this purpose, principally from flax, cork and limestone, are used.

Flax is made up into blankets enclosed by sheets of waterproof paper. Cork is made up into cakes pressed and cemented together with asphaltum or other waterproof cement. Limestone is heated to 3500° and when liquefied is blown into shreds so as to make a sort of rock wool enclosing a very large percentage of air bubbles forming excellent insulation. The latter material forms an excellent filling for ice house walls.

The treatment of partitions is somewhat similar, but as the difference in temperature between the two sides of the partitions is not so great as between the interior of the house and the exterior the same care and insulation is not justified.

WOOD FRAME ICE HOUSES.

The two principal objections which can be raised to wood frame ice houses are the danger of fire and the decay of the timber. As timber lends itself so well to ice house construction, safety against fire can be obtained by covering the exterior walls with stucco and expanded metal and covering the roof with a fireproof material. Any openings into which birds might otherwise enter and build their nests should be covered by a galvanized iron mesh. Decay can be postponed for many years by treating the timber and there seems no good reason why treated timber should not be used in ice house construction.

Several of the large meat packers are still building ice houses of frame construction with excellent results, the Cudahy company reporting that one of their wood frame ice houses at Seymour Lake, South Omaha, Neb., has gone through two or three seasons with a shrinkage of only about 4 per cent.

In brick and concrete houses the walls must be insulated fully as effectively as for wooden houses. In some cases timber frames have been built inside the brick and concrete houses and filled with rock wool or other material similar to the method used in the wooden houses, but in by far the great majority of cases the inside surfaces of the walls are completely covered by slabs made up of waterproof materials such as cork, board, etc., plastered to the walls and in some cases plastered on the interior.

ROOF CONSTRUCTION.

There is little difference in the type of roof used for the various kinds of houses. Steel trusses are seldom or never used on account of the rapid corrosion which would result from the dampness. These trusses are usually so built that the roof covering can be applied to the top chord while the ceiling for the ice chambers can be applied to the bottom chord. This affords an air space between the roof and the ceiling of the ice house through which air can circulate and a good insulating air cushion results. The ceiling of the ice house must be insulated just as effectively as the walls, and practically the same type of construction is made use of for that purpose.

The greatest care must be taken with the connections of the walls to the foundations and also with the connections of the ceiling to the walls and partitions in order that there may be no opening, however small, through which air can enter directly into the ice chambers.

The height of the attic space is usually not less than 3 ft. and varies in height to 8 or 10 ft., as in some cases extensive machinery is located in this space. The space between the ceiling and the roof affords a good air insulator and the roof prevents the direct rays and heat of the sun from playing on the ceiling. The walls around this space should be built up tight enough to prevent birds from entering and building their nests, while there must be sufficient openings in the form of louvres to permit the free circulation of air. These louvres and any other openings should be covered with galvanized wire screening to keep out birds' nests and rubbish which would otherwise create a fire hazard.

The doors are usually of ordinary refrigerator construction with several sections separated by air spaces which are now sometimes filled with insulating material. The greatest care must be taken in the connections of the doors to the houses that no air can circulate between the door and the frame. In some houses doors are separated and in others they extend vertically from the top to the bottom of the house in each chamber. In the first type the openings can be so controlled as to admit of the entrance of very little warm air to the chambers, although when the doors extend the full height of the house there is more likelihood of the entrance of warm air and greater difficulty in keeping the joints between the doors tight.

STORING OF ICE.

Cakes of ice are stored in all sizes and on edge as much as flat. In many cases the cakes are placed as close together and as close to the walls and partitions as possible, while in others spaces several inches wide are left between the cakes of ice and between the ice and the walls. It is difficult to understand the reason for leaving these spaces, as they give the best possible freedom to the circulation of the air, resulting in increased melting of the ice, which could be avoided by filling up the entire space between the walls with ice laid in contact. The cakes, however, need not be in close contact as would result in their freezing together. This also applies to the individual rooms, which should be filled up as close to the ceiling as possible.

In the past it was the quite universal practice to cover ice with sawdust and to fill all crevices between the cakes and against the walls with that material in addition to covering the ice with several feet of it. That practice has practically ceased, as it was found that the cleaning of the ice resulted in a very great loss due to the ice melting rapidly under the water used in washing it. The circulation of air between the cakes of ice is also decreased somewhat by placing alternate layers on edge and flat when the size of cakes is sufficiently uniform to permit this.

It is necessary, however, to use some means to keep the cakes of ice apart in order that they will not melt together if shrinkage is rather rapid. In many cases strips of board have been placed between the layers, but that does not appear to have been very effective, as the strips usually melt into the ice and permit the cakes to come together. More recent experiments have been tried in the use of waterproof paper which, of course, will effectually prevent the cakes from freezing together and will, in any case, prevent the circulation of air from layer to layer. In rooms holding ice which must be stored for many months for rapid use when the demand arises it is well to cover the ice with a thick layer of some insulating material, such as hay or straw, which will prevent the circulation of air. The use of materials which necessitate washing the ice, however, should be avoided.

PLATFORMS AND HANDLING EQUIPMENT.

The length and width of the platform for handling ice between the cars and ice house depend upon the requirements. As a rule when conveyors are not used and ice is handled in buggies, a platform 16 ft. wide will be found to be very convenient, although narrower platforms have been successfully operated. Where a comparatively small number of cars is iced short platforms are sometimes built in front of the house of sufficient length to accommodate 5 or 10 cars, a switch engine being required to handle the cars when there are more than can be placed at one time. Where there are many trains to be iced it is of considerable advantage to have an icing platform extend the full length of a train so that it can be iced at one spotting, especially at engine terminals where such trains do not need to be broken up.

The method used for distributing the ice along the platform also depends upon the amount of icing to be done. Where many trains are to be iced and where platforms are long it is of advantage to use continuous platform conveyors to carry the ice from in front of the ice house to the points where it is put in the cars.

In some cases adjustable platforms are built adjoining the house and continuous conveyors carry the ice to and from the house by adjustable inclines. Many houses, however, are built with no mechanical means for handling ice, and temporary hoists operated by horse power or by temporary hoisting engine are used. Such means, however, are entirely inadequate for large houses. In the great majority of such cases vertical elevators or skips are used. The skips handle from 600 to 2,000 lbs. of ice each trip and some make as many as three trips a minute, the floor and wiring of the skip being so adjusted that the skip stops and the ice automatically slides off when the proper level is reached. Such elevators will handle from 500 to 1,000 tons per day in or out of the house.

In some cases ice is gotten out along the platform in advance of the arrival of trains, and the cars are iced with very little delay; while at other points, particularly where cars must wait for some time in any event, ice is gotten out while the cars are standing. The best method, of course, is to have the ice on the platform immediately in advance of the arrival of the train, where it can be covered and protected from the direct rays of the sun.

REMARKS.

The information also indicated that wherever at all convenient and practicable the railroads rely on commercial ice companies for ice. This is especially true in the East where the greater density of population necessitates the manufacture of great quantities of ice, and the manufacturers can supply the additional amount required by the railroads at much less cost than the roads can furnish it themselves. Especially in the West, however, it is necessary for the railroads to provide their own ice supply. About one-third of the railroads from whom information was asked reported no ice houses, as they relied entirely upon commercial firms for their ice supply. The remaining two-thirds, comprising practically all of the large systems, gave detailed reports of their standards, customs and experiences.

SEWERAGE

Design of Two Residential Sewage Treatment Plants, Including Settling Tanks of Imhoff Type.

Contributed by Samuel A. Greeley, Hydraulic and Sanitary Engineer, 200 Chestnut St., Winnetka, Ill.

Higher standards of sanitation and a more general knowledge of the nuisances and dangers resulting from improper sewage disposal are requiring architects to seek the co-operation of sanitary engineers in the design of sewage treatment plants for residences and institutions. Experience is showing them that special knowledge is required properly to design such plants and that local conditions must be considered. Owners of estates where there are no public sewers are looking for the best and most practical method of treating their own sewage, so that they may require of their neighbors equally satisfactory methods. The discharge of raw sewage into ravines or small streams will make nuisances and sources of disease which may be distributed by flies or contracted by children. Conditions like this have recently called for the design of two residential sewage disposal plants in a small town near Chicago. These plants are here illustrated and described.

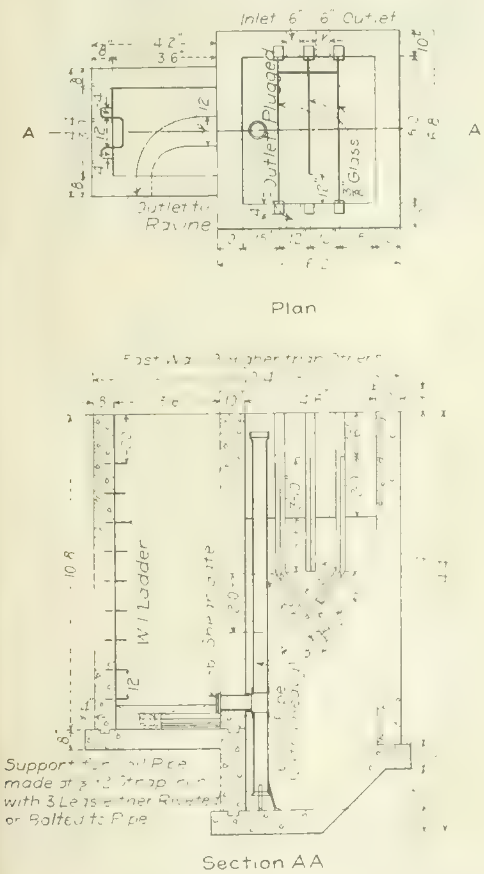

Fig. 1. Plan and Section of Imhoff Settling Tank at Residence A.

The town is cut by seven main ravines extending easterly from the center of the town to Lake Michigan. The extension of sewers across these ravines is difficult. Consequently the high land along one large ravine at the southern end of the town is not sewered and the residences adjacent need individual sewage treatment plants. The flow of water in the bottom of the ravine is negligible during the summer months, so that a thorougtly stable effluent from the disposal plant is necessary for discharge into the ravine.

At both residences the plumbing and outlet sewers were built before the designs for sewage disposal were undertaken, so that the location and elevation of the end of the outlet sewer were fixed. This and certain other local conditions placed limitations upon the designing engineer. The hearty co-operation of the architects in both instances helped to offset these limitations.

Residence A was designed by Mr. Howard Shaw. Sewage disposal is required for the residence, a group of service buildings and a small garage. The number of people was estimated to range from 22 to 28. The outlet sewer for all of the establishment terminated in a small ravine tributary to the main ravine. The outlet was set too low for subsurface treatment on the flat ground above the ravine. The fall to the bottom of the main ravine, however, was about 50 ft.

There was not sufficient space in the small ravine or the main ravine for sand filters. Moreover, the desirability of covering these would have added to the cost and would have made cleaning and attention difficult. There was plenty of fall for sprinkling filters, and these were considered preferable to contact beds. The layout finally adopted included a settling tank of the Emscher or Imhoff type, a covered sprinkling filter and a small secondary settling basin. The details of the design are shown in Figs. 1, 2, 3 and 4.

The average daily flow of sewage was taken at 50 gals. per capita and the rate during the daytime was taken at twice the average rate, the plant being designed on a basis of 25 people. The Emscher tank was designed to give a settling period of three hours on the average daily rate of flow. The sludge digestion space is estimated to provide 20 months' storage. The floating sludge space was made large, amounting to 125 per cent by area of the surface of the tank. This was done because of the character of the sewage. A sludge draw-off manhole was built, but no sludge bed was deemed necessary.

Novel features of the Emscher tank are the wire-glass baffles separating the settling compartment from the sludge chamber. These were selected for structural reasons as being the easiest to install and offering a desirable and lasting material.

The settled sewage flows to a covered sprinkling filter designed to operate at a loading rate of 10,000 people per acre per day. There is a manhole in the line from the settling tank to the filter. The sewage is applied to the filter by a tipping bucket which discharges 1.5 cu. ft. alternately onto one of two brass splash plates. The tipping bucket, illustrated in Fig. 3, is a very simple dosing apparatus, not subject to the danger of breaking if not in use during freezing weather. Each bucket discharges into a galvanized sheet iron hopper designed to give a varying head and varying quantity of sewage, corresponding to the decreasing annular area of filter reached by the spray.

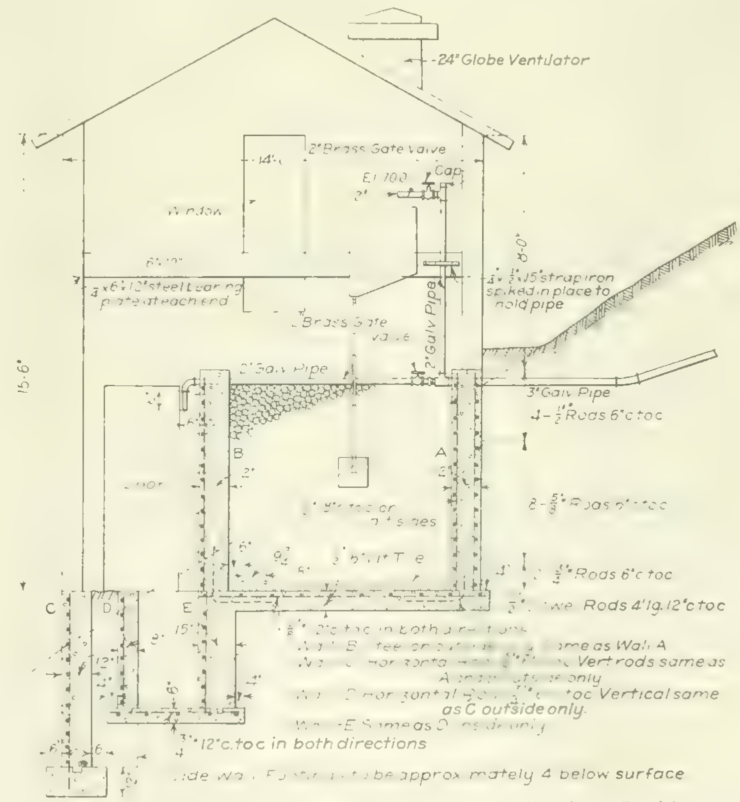

Fig. 2. Section Through Sprinkling Filter and Housing, Residence A.

The splash plates, illustrated in Figs. 4 and 5, are of brass, screwed onto copper rods working in galvanized iron pipes anchored into the filter material by a concrete block. They were selected to avoid trouble with nozzles clogging.

The filtering material is 6.5 ft. deep, composed of 1¼-in. to 2¼-in. Wisconsin limestone carefully graded and washed. The underdrains are half-round tile, sloping to a main drain in the concrete extending along the center of the floor. Riser pipes from the underdrains for ventilation and flushing are set in each corner outside the circular spray line.

A shallow final settling basin is provided having a sloping bottom and large enough to give a settling period of 1.5 hours on the average daily rate of flow.

It was considered advisable to cover the filter and final settling basin with a small house, 17 ft. by 14 ft. in plan. This was done to prevent any possibility of odors and

TABLE I.—ENGINEER'S FINAL ESTIMATE FOR SEWAGE TREATMENT PLANT AT RESIDENCE A.

Item	Cost.
Earth excavation, 284 cu. yds. at $1.00	$ 284.00
6-in. vitrified pipe, 346 ft. at 25 cts.	86.50
Steel reinforcement, 2,879 lbs. at 4 cts.	115.16
Settling tank and manhole, lump sum	400.00
Dosing apparatus, lump sum	95.00
Sprinkling filter, lump sum	520.00
Filter house, lump sum	620.00
Total of estimate	$2,120.66

E. Schmidt, Martin and Garden. The disposal of sewage was required for the residence only, the estimated number of people ranging from 15 to 25. Meter records of the water supply were available, showing an approximate average daily consumption of 50 gals. per capita. The new sewage works were designed on this basis.

ter. This gives a long travel and ample opportunity for large particles in the sewage to become water-logged and sink. The flowing through time, based on the average daily flow, is six hours. This was considered safe on account of the final treatment below the surface of the ground. The sludge digestion chamber has sufficient capacity for 20 months' storage. A sludge outlet manhole is provided. The lower deflecting baffle in this tank is built of creosoted cypress. It was built up outside the tank and lowered into place and nailed to the horizontal cypress strips previously anchored into the concrete side walls of the tank. The upper baffles of the tank are built of concrete. These were cast outside the tank in four slabs, each reinforced with 2-in. wire mesh. The mesh was left extending from the upper end of each slab about 6 ins. These slabs were lowered into the tank with an A-frame, and the wire reinforcement fastened to steel rods set in concrete beams spanning the tank. The corners were afterwards reinforced and plastered with cement mortar.

The settled sewage flows into a siphon chamber, in which is a 4-in. Miller siphon. This siphon discharges through a line of 6-in. tile pipe to a diverting manhole, where the discharge or flush may be turned to either half of the sand filter, as desired.

The sewage is distributed over the filter area through a system of 4-in. tile laid with

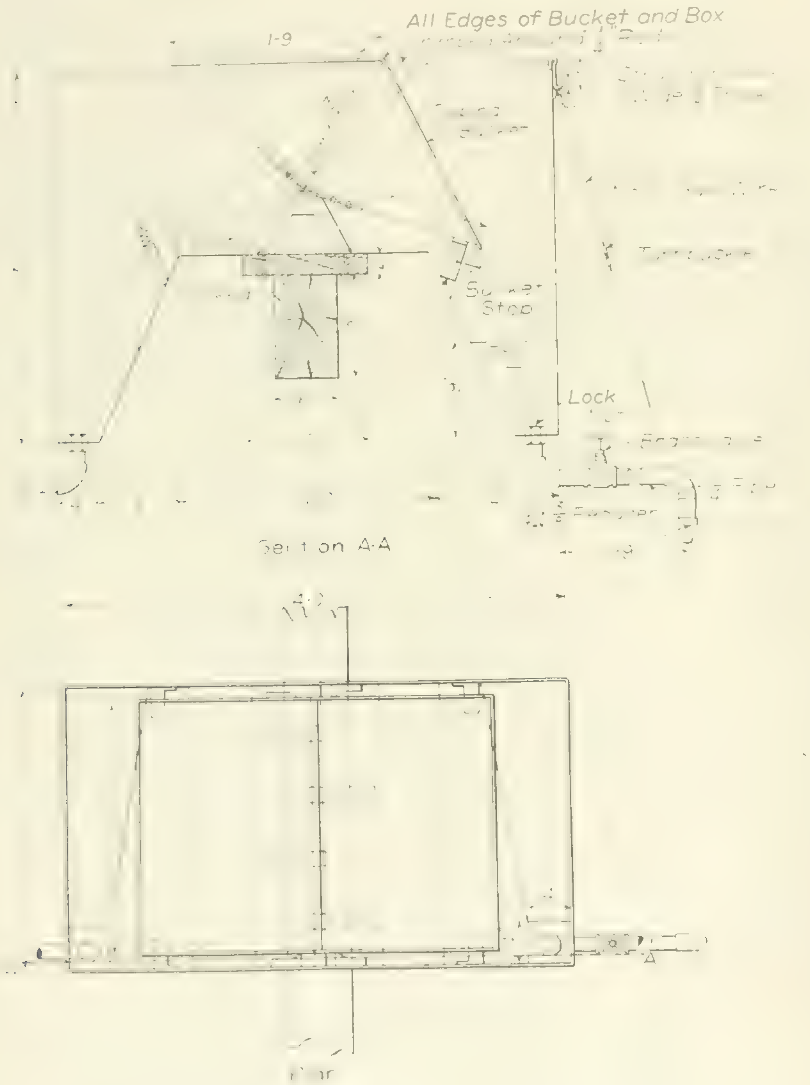

Fig. 3. Plan and Section of Dosing Tank, Residence A.

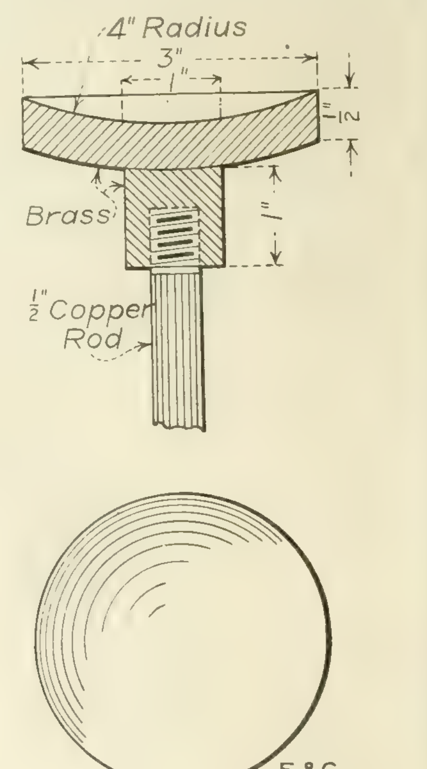

Fig. 4. Detail of Splash Plate, Plant at Residence A.

to give the plant an attractive appearance. The house was built of hollow tile, rough plastered with cement mortar and covered with a blue slate tile roof. The design conforms to the other service buildings on the estate. Four windows, a door and ventilator are provided. The house sets low on the side ravine and is not visible from the residence.

All parts of the plant are accessible, and bypasses are provided for the filter and final settling basin, but not for the Emscher tank.

Bids were received for the construction of the plant in accordance with Table I, which gives the engineer's final estimate. The total amount of the bid was very close to the engineer's estimate and is a fair figure for the cost of the work.

The work at Residence B was done in conjunction with extensive additions to an old house. The architects were Messrs. Richard

The old house was provided with a large sand filter and there was available, therefore, a large quantity of sand. The main sewer from the new house was set at a sufficient elevation so that an area for sub-surface disposal could be made available by grading down the present surface about 3 ft. The soil was heavy clay, so that the excavated space was refilled with the available sand to make a soil suitable for sewage treatment. There was sufficient sand available to make a disposal field covering approximately 6,000 sq. ft. Owing to these local conditions it was found expedient to select treatment by preliminary sedimentation, followed by sub-surface irrigation in a sandy soil.

The settling tank is of the Emscher type, illustrated in Fig. 6. The sewage flows around the tank in the flow space at the outside, with a floating sludge space at the cen-

open joints and terminating in short riser pipes. These distribution tile are set into a sand cover of from 9 to 12 ins. Below these, 3 to 4 ft. from the surface and set in trenches, are the collecting tile, draining to a main drain at the center. These collecting tile are completely surrounded with coarse gravel. The effluent discharges through a sand bank into the main ravine.

The cost of the settling tank complete, including the Imhoff royalty and the siphon chamber, was $385. The filter work was done by day labor under the direction of the writer's resident engineer.

The design of Emscher tanks for small works is new, but is in line with present developments. The tanks are no smaller than a number that have been successfully operated at testing stations. Certain modifications have been made to meet this special service. Experimental work along this line is now being performed by the U. S. Public Health Service, two test plants having been built, one for the sewage of 15 people and another for the sewage of 100 people. A more noteworthy feature is the indication that architects and owners are realizing the need for a rational design of small sewage treatment plants with due regard for differences in local conditions and the degree of purification required.

Fig. 5. View of Splash Plate in Use.

The plants were designed by the writer and built under his supervision. Mr. Robert A. Allton, recently of the New York State Board of Health, was resident engineer. The L. K. Sherman Co., of Chicago, was contractor for both installations. Both plants went into operation in June, 1914, and have been working satisfactorily since that time.

Recommended General Procedure in Sewage Works Operation.

Sewage treatment plants are built to accomplish one of the two purposes here mentioned: (a) To promote the comfort and convenience of the people by maintaining water courses in a clear condition, free from nuisance to sight or smell, or (b) to aid in protecting the public health, when necessary, by maintaining water courses as fit sources of water supply after adequate purification or fit for the cultivation of shell fish.

In line with the two purposes mentioned, the operation of a sewage treatment plant may be judged by the following standards: (a) Prevention of nuisance either at the works or in the body of water receiving the effluent; (b) Protection to the public health, for example, in reducing the pollution in a water course to relieve the load on a water purification plant. In the present discussion, which is from the report of the Committee on Sewage Works Operation and Analytical Methods of the Sanitary Engineering Section of the American Public Health Association, it is assumed that under the guidance of competent sanitary engineers the specific problem has been studied, resulting in the design and construction of efficient sewage treatment works and, upon completion, the owner is confronted with the problem of its operation and maintenance. The second portion of the report, which is a progress report on analytical methods, will be published in a subsequent issue of this journal. The committee members are: C. B. Hoover, H. C. McRae, Langdon Pearse, George C. Whipple and W. L. Stevenson, chairman. The committee acknowledges the helpful co-operation of Mr. F. E. Daniels of the New Jersey State Board of Health.

One of the first and most important matters to be considered in the operation of a sewage treatment works is the heterogeneous nature of sewage. Its composition varies widely in different towns, influenced by the ways of living and amount and kind of industrial wastes. Some works receive a sewage so fresh that original faecal matter, paper and fats are practically unchanged, while in others the solids have disintegrated and partly dissolved.

The hourly character and composition of the sewage may also vary from day to day, being uniform in one place and different in another. During storms the sewage collected by a combined system of sewers is increased in volume and its composition changed to a very marked degree.

Many of the forces of nature are utilized in sewage treatment in an involved and complex manner. Gravitation is utilized in sedimentation and in the hydraulics of the flowing liquids; chemical reactions occur in the use of precipitants, in disinfection and in the changing combinations of sewage materials; and biological forces are active in the digestion of sludge and in the oxidation of the dissolved organic matter.

In successful operation all of these factors must be controlled. It is, therefore, self-evident that sewage treatment works, particularly of any size, cannot be made automatic, but require constant observation and supervision by trained, skilled operators in order to accomplish the purpose for which the plant was installed and the funds invested.

SUPERVISION.

The degree of technical skill required depends upon the following factors, but not necessarily in the order given: (a) The magnitude and design of the works, (b) the processes used, (c) the composition of the sewage treated, (d) the receiving body of water.

It does not always follow that because a sewage treatment works receives a large volume of sewage that the responsibility is in proportion to the magnitude, because the processes used may be simple and the dilution of the effluent large. Some small works, for particular purposes, e. g., fat recovery, may require very expert supervision.

The use of biological works for oxidation or the use of disinfection always requires more attention than simple sedimentation. Emscher tanks, which include sedimentation and sludge digestion in a single device, require a higher degree of skill to operate than plain sedimentation tanks. The presence of considerable volumes of trade wastes in sewage usually introduces difficulties not encountered with ordinary domestic sewage.

Constant attendance upon a sewage plant for a private estate or small institution is unnecessary. Such plants should be simply designed, so that an employee with other duties is required to give but a small part of his time to the care of the plant. But in such cases successful results are not always obtained unless the designing engineer or some competent person is called in at regular times to inspect and advise concerning the technical details of operation. Such inspections are particularly important during the early life of the plant to train the attendant to observe and interpret the conditions which require attention.

For the larger institutional plants and works for small towns more skilled attendants are required and the need for competent supervision more urgent.

In municipal sewage treatment works of any magnitude the constant services of a scientifically trained man in charge is practically essential to success. The larger volumes of sewage treated afford greater potentialities for the creation of nuisance from bad odors if the works are not constantly kept in the best condition. The effect upon the receiving body of water is also greater and a deterioration in the quality of the final effluent may allow the development of conditions such as nuisance in the water course or danger to water supplies which it is the very function of the works to prevent. And finally it is economical to pay a liberal salary to the right kind of man to keep the annual charges for operation and maintenance at a minimum figure by business-like management.

RECORDS.

The keeping of careful, thorough and neat records of operation is essential to efficient operation. State authorities charged with supervision over the condition of the water courses should require a uniform record. Such records from the operating point of view show the effect of changing methods of operation and enable the selection of the most efficient and economical procedure. Complaints of citizens can frequently be answered and explained away by reference to the records, and often well kept records have prevented suits or lessened damages.

As an example of the value of records, two different hypothetical cases may be cited: A sewage treatment works required to prevent the creation of nuisance in a stream not used nearby as a source of water supply. Here records of the stage and conditions of the stream and condition of the final effluent may show that less than complete treatment would be sufficient during a portion of the year and the saving in labor of operation materially reduce annual operating charges and still the plant would efficiently perform the function for which it was installed.

On the other hand, a plant whose purpose is primarily intended to protect the nearby source of water supply requires most careful records, so that in case of the failure of the water purification plant to produce a wholesome water the responsibility cannot be placed upon the sewage treatment works.

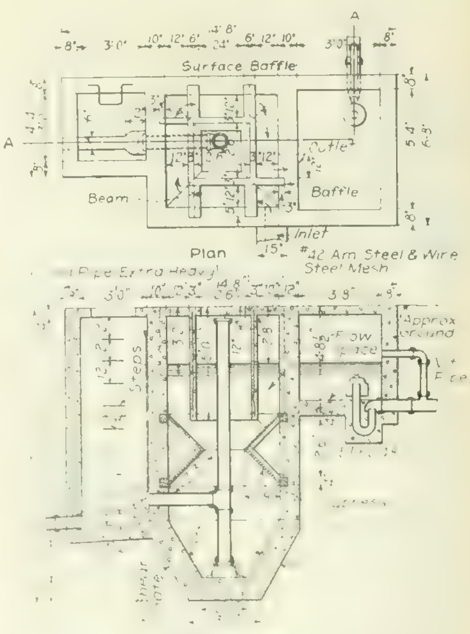

Fig. 6.—Plan and Section of Imhoff Settling Tank, Residence B.

Records to be really valuable should not be cumbered with useless data. The essentials should be carefully determined and made graphic wherever possible.

NEATNESS.

Too much stress cannot be laid upon the importance of neatness and cleanliness in and around the works and the surrounding grounds. To accomplish its purpose this cleanliness must begin in the sewer system, for accumulation of decomposing matters therein will surely hasten decomposition to such a degree that at times no amount of care at the works can prevent the dissemination of bad odors. The cleanliness begun in the sewers should be carried out through the plant and no filth should be allowed to accumulate, but it should be promptly disposed of.

But small expense is involved in laying out flower beds or shrubbery, especially at the entrance to and to serve as a screen around unsightly parts of the works. A moderate amount of attention to these details removes the inherent prejudice in the mind of the visitor or passerby.

Construction Plant and Methods Employed on the North Shore Intercepting Sewer, Sanitary District of Chicago.

Contributed by M. A. Berns, Chicago, Ill.

The Sanitary District of Chicago has done and is doing much to decrease the pollution of the waters of Lake Michigan, from which the city's water supply is drawn, by diverting into the Chicago River and the Drainage Canal the sewage formerly discharged into the lake by the North Shore towns. The sewers built by the Sanitary District are for municipal sewage only, not for storm water or individual house connections. Temporarily, some storm water will pass through these sewers, but gradually the villages will intercept this.

One of the large intercepters now under construction is the North Shore intercepting sewer, which extends along Sheridan Road from the North Shore channel at Wilmette through Winnetka, Kenilworth, Glencoe and part of the Township of New Trier and intercepts all the municipal sewers of these suburbs. It varies in size from 9 ins. in diameter to a large 6x9-ft. egg-shape section running on an average grade of .05 per cent. Concrete is being used as the material of construction because of its durability and economy, and the ease with which it is handled and spouted to place. The usual mixture is 1:2½:5 of Universal Portland Cement, washed and screened sand and gravel, or stone. On the smaller egg-shaped sewers a 1:2:4 mixture is used.

Fig. 1. View Showing Mixer Mounted on Rollers, and Flexible Concreting Chute, North Shore Intercepting Sewer Construction, Chicago.

WORK ON LARGE EGG-SHAPED SEWER.

Work along the North Shore is divided into three separate contracts. Contract No. 1 runs along Sheridan Road from the North Shore channel through Wilmette to Cherry Street, Winnetka. This is made up of 10,060 ft. of 6x9-ft. sewer and 4,280 ft. of 6x8-ft. sewer, giving a total of approximately 2¾ miles. The crown is given a thickness of 10 ins. and the invert a thickness of 12 ins., requiring a total amount of concrete of over 15,000 cu. yds. The cut for this part of the work varies from 20 to 24 ft. and the work is done at a cost per lineal foot, including excavation, back filling and concreting of $15 for the 6x9-ft. section and $14.50 for the 6x8-ft. section, making a total price for the contract complete of $233,999.75.

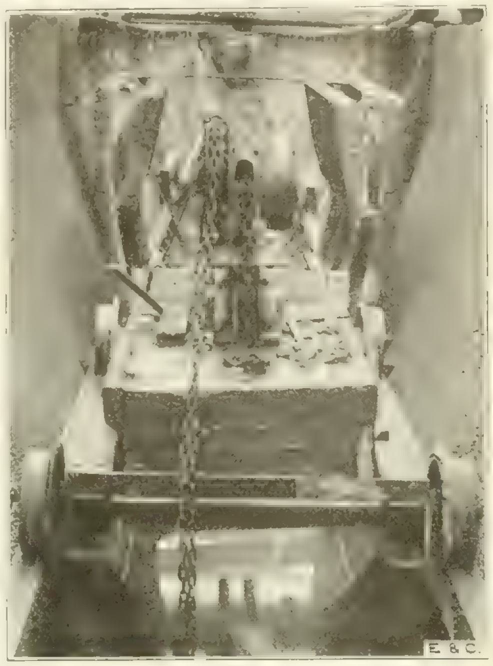

Fig. 2. View Showing Steel Invert Form Collapsed for Removal to New Position, North Shore Intercepting Sewer.

Method of Handling Concreting Materials.—The aggregates are brought to the site in ordinary dump wagons and delivered at convenient places along the sides of the streets. From there they are wheeled in barrows and dumped into the hopper of the Smith concrete mixer on a level with the mixer floor. For convenience in moving, the contractor has dismounted the mixer from its trucks and placed it on a platform spanning the trench, the platform being carried on rollers, as shown in Fig. 1. Concrete is chuted through a flexible steel spout as shown. No concrete for the arch portions is poured until the concrete previously placed up to the springing line has set, so that it is necessary to move the mixer back from 60 to 125 ft. for pouring on the arch form. This moving operation requires only six minutes.

Use of Collapsible Steel Forms.—Specially designed collapsible Blaw steel forms are used. They are in 5-ft. lengths with the invert form a separate unit from the arch form. The invert form is hinged at the bottom center line and at a point 18 ins. below the springing line so that it will telescope within itself in order to permit of moving through the set up forms ahead. Turn-buckles across the horizontal center line serve to keep the forms in line. About 100 ft. of invert forms were supplied, and these are placed on cast concrete blocks, 15x9 ins. long, spaced on 5-ft. centers. When the concrete is cast these blocks become a part of the monolithic sewer.

In operation the men remove the rear forms, telescope them and push them ahead on a specially designed car. Within the forms and for some distance back on the already finished invert is provided a 2¼-ft. gage track about 1½ ft. above the bottom of the invert and supported on cradles, as shown in Fig. 2. On this track runs a 10-ft. traveler, counterweighted on one end with concrete and so designed as to clear the turnbuckles. Unbolting a form and allowing it to collapse permits of raising and supporting it by means of a cable running over a sheave operated by a hoist on the counterweight of the traveler. The men then move the traveler back and place a cradle and temporary track to span the space where the form was removed, after which the traveler is ready to be pushed forward through the forms to a new position ahead. Crown forms are also used telescopically and handled on the traveler. Trained men can move ahead and set up a maximum of 125 ft. of bottom forms per 8-hour day, while the average movement was 80 ft. per day. Except in soft or wet sandy soil, the bottom forms are removed after the concrete has set for 24 hours, and the arch forms after 48 hours.

Excavating and Back Filling.—A 65-ton, 1½ cu. yd. self-propelling Bucyrus shovel mounted across the trench on rollers in a fashion similar to that described for the mixer is used for excavating, as shown in Fig. 3. Hand labor is used to bring the cut to its proper shape and depth. There are about 7½ cu. yds. of excavation per lineal foot of trench, and the shovel is able to make an average progress of 125 lineal feet per 9-hour day. Five-yard Western dump cars and a Davenport dinkey locomotive are used to transport the excavated material for back-filling, the surplus being dumped in the spoil area along the lake shore east of Sheridan Road. At the discretion of the engineer some cold weather work is done. In such cases the sand and gravel are heated by exhaust steam conveyed through pipes running under the sand and gravel.

TUNNEL WORK REQUIRED FOR DEEP SECTIONS.

Contract No. 2 is for the 40x62-in. egg-shaped sewer from Sheridan Road along Winnetka Avenue in Winnetka to Hill Road. It consists of 2,891 ft. of work in open cut, for which a Parsons trench machine was used, and 3,680 ft. of tunnel. The total contract price for this was $84,404.25. Air through 2-in. and 2½-in. pipe lines for the tunnel work was supplied by an engine driven blower. Light was

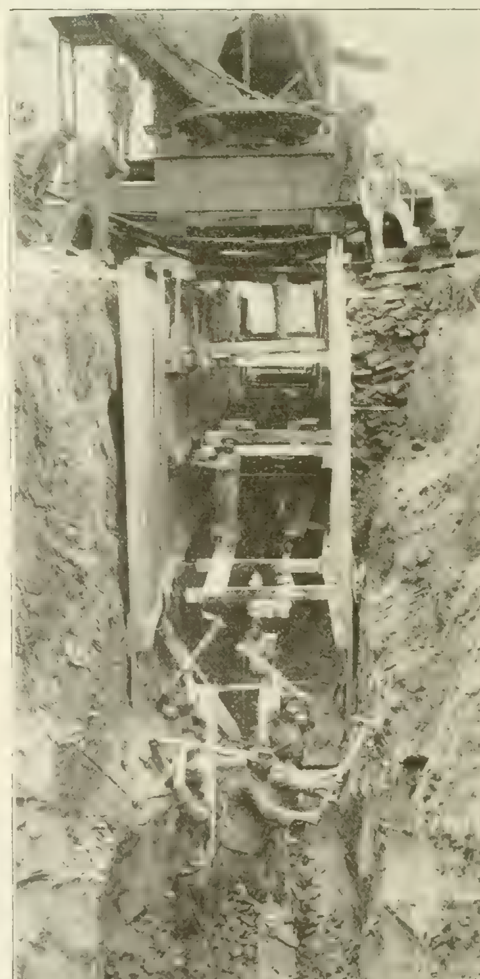

Fig. 3. View Showing Excavating and Timbering Methods, North Shore Intercepting Sewer.

furnished by electricity taken from the Winnetka pole line along the work. Excavation for the tunnel was carried on from three shafts, 800 to 1,000 ft. apart, working one heading each way and using three shifts per 24 hours. Owing to the ease with which the concrete could be placed it was found that the limit of a day's work was always determined by the rapidity of the excavation. For this reason two shifts in each heading excavated and one shift set up the forms and placed the concrete. Small tunnel cars on a 15-in. gage track brought the clay to the shaft, where it was hoisted to the surface and to an overhead trestle from which cars and wagons were filled. The concrete for the sewer was brought to proper shape and grade by placing steel forms on cast concrete blocks similar to those used in the work on Contract No. 1. The contractor finished the work practically 14 months ahead of scheduled time, notwithstanding the fact that right-of-way trouble necessitated making a reverse curve in the tunnel work after it was well under way.

Contract No. 3 was for the work from Hill Road along the Skokie Marsh to Glencoe. Part was in open cut, and part in tunnel with the sewer of varying sizes down to 9 ins. in diameter. This was carried on in much the same manner as that described for Contract No. 2. The contract price was $248,376.30.

These projects are paid for in cash by the Sanitary District of Chicago out of its tax levy or bond issue. Work is under the direction of George Wisner, chief engineer, and Langdon Pearse, division engineer, with H. R. Abbott, assistant engineer in charge of the three sections along the North Shore. On Contract No. 1 the H. J. McNichols Co., Chicago, is contractor, with J. H. Flagg as consulting engineer and Richard McNichols in charge of the work. Work on Contracts Nos. 2 and 3 is being done respectively by the Marquette Construction Co. and Nash Bros., both of Chicago.

ROADS AND STREETS

Paving Procedure in American Cities.

To secure statistics upon which to base recommendations for future paving work in the city of Syracuse, N. Y., a committee of the Chamber of Commerce of that city sent letters to the officials of cities exceeding 50,000 in population, requesting specific information concerning paving practice. From the replies received from 68 cities the following statistics were obtained:

Method of Ordering Pavements.—In 60 per cent of the cities heard from, any city street may be ordered paved by the common council, the majority varying from one-half to five-sixths, it usually being two-thirds. In 30 per cent of the cities, no new paving can be ordered except by a petition of from one-half to two-thirds of the abutting property owners. In 10 per cent of the cities, the common council is permitted by ordinance to order a stipulated amount of new pavement each year.

How New Paving Is Paid for.—In only 22 per cent of the cities do property owners pay entire cost of new paving. In 17 per cent, the city pays for the street intersections only. In 9 per cent, the city pays 2 per cent of the entire cost of paving. In 10 per cent, the city pays 33⅓ per cent of the entire cost of paving. In 10 per cent, the city pays 50 per cent of the entire cost of paving. In 20 per cent, the city pays entire cost of paving. In the remaining cities, the percentage of public benefit is adjudicated by a board of commissioners. Three per cent of the cities make an allowance for corner lot paving.

How Repaving Is Paid for.—In 16 per cent of the cities, property owner pays entire cost. In 9 per cent of the cities, city pays for street intersections only. In 6 per cent of the cities, city pays one-third of repaving cost. In 19 per cent of the cities, city pays one-half of repaving cost. In 43 per cent of the cities, city pays entire cost of repaving. In balance of cases, the public benefit is adjudicated by a board of commissioners.

How Type of Pavement is Selected.—In only 15 per cent of the cities do the abutting property owners have the final decision in selection of type of pavement. In a few cases, the common council or even a single alderman has the deciding power. In about 80 per cent of the cities, however, the type of pavement is selected by a paving commission, highway or engineering department, after due consideration of the wishes of the abutting property owners.

Paving Material.—About 85 per cent of the cities use creosoted wood block to some extent on heavy traffic streets, the average being about 1.9 per cent of the total paved streets. One city has 50 per cent of its pavements of this material. The usual guarantee is five years.

Nearly all cities heard from use asphalt, brick, granite block and waterbound macadam to a greater or less extent, depending on traffic conditions, cost of material, etc. Brick seems to be used largely where it is manufactured locally. The average percentage of the various types of pavement to the total is about as follows: Wood block, 1.9 per cent; sheet asphalt, 29 per cent; asphalt blocks, 1.7 per cent; brick, 14 per cent; waterbound and bituminous macadam, 35 per cent; stone block, 13.5 per cent. Balance is made up of concrete, bitulithic, asphalt and other types in small quantities.

A five-year guarantee is customary with asphalt, brick and wood block; two to three-year guarantee on waterbound and bituminous macadam, and one year on granite block. The total mileage of paved streets in different cities varies from 12 to 90 per cent of the total street mileage, a fair average being about 48.5 per cent.

Maintenance.—Practically all the cities pay for repair and maintenance of pavements after expiration of guarantee. A large number own and operate their own asphalt plants, some even building their new pavements by day labor.

Financing.—Where city pays entire or major portion of paving, in most cases it is taken care of in the annual budget and tax levy. In a few cases by serial bonds.

Where the property owner's share of paving is proportionately small it is usually paid in cash within thirty days from time work is begun. In some cases two to three years are allowed in which to pay. Where the property owners pay the major share it is usually paid in ten annual payments. In a few cases it is five or six years. Bonds are usually issued.

Railroad Strip Paving.—This is more often done by the city at railroad expense, although in many cases it is done by the railroad itself. In 50 per cent of the cities the strip outside the outer rail which the railroad is obliged to pay for and maintain is 2 ft. wide. In the remaining cases it varies from 1 ft. to 18 ins. Usually depends on franchise agreement.

Recommendations.—In order to make the work of this committee constructive and of the greatest value to our city we must determine the fundamental cause of the poor road conditions and change that first.

The paving situation in Syracuse will never be satisfactorily settled until the statutes governing us are radically changed and a more equitable treatment of the property owner is made.

Removing from the property owner the burden of paying the proportionate cost of new paving, which is a public benefit and should be paid for by the city at large, will be an added inducement for new paving, and the present necessity of forcing pavements on the property owners will be greatly lessened.

Placing in the hands of experts the designation of the type of pavement most suited to the traffic conditions, after a public hearing of property owners, will result in the most efficient type being designated, and less objection will come from the property owner, as he pays only part of the cost.

The present poor condition of our paved streets is due to the objections of property owners on account of the cost of resurfacing. If the city would assume the entire cost of resurfacing, property owners through their representatives would not hold up the resurfacing of the many miles of pavement which the city engineer's record shows to be worn out.

From a careful study of the experiences of sixty-eight cities and also that of our own city, we would make the following recommendations:

(1) Property owners should not be permitted to choose type of pavement without the approval of some more expert authority. (2) City at large should pay one-third of the cost of all new pavements as well as street intersections. (3) City at large should pay entire cost of repaving.

Following is a list of cities from which information was received:

Akron, O.	Minneapolis, Minn.
Albany, N. Y.	Manchester, N. H.
Atlanta, Ga.	Newark, N. J.
Atlantic City, N. J.	New Haven, Conn.
Baltimore, Md.	Nashville, Tenn.
Birmingham, Ala.	New York—Bronx.
Boston, Mass.	New York—Manhattan.
Bayonne, N. J.	New York—Brooklyn.
Bridgeport, Conn.	New York—Richmond.
Charleston, S. C.	Norfolk, Va.
Camden, N. J.	Oklahoma City, Okla.
Columbus, O.	Oakland, Cal.
Chicago, Ill.	Providence, R. I.
Cambridge, Mass.	Pittsburgh, Pa.
Duluth, Minn.	Paterson, N. J.
Dayton, O.	Philadelphia, Pa.
Des Moines, Ia.	Portland, Me.
Elizabeth, N. J.	Richmond, Va.
East St. Louis, Ill.	Somerville, Mass.
Erie, Pa.	Schenectady, N. Y.
Evansville, Ind.	Scranton, Pa.
Fort Wayne, Ind.	St. Paul, Minn.
Fall River, Mass.	St. Louis, Mo.
Grand Rapids, Mich.	Salt Lake City, Utah.
Hoboken, N. J.	Spokane, Wash.
Hartford, Conn.	South Bend, Ind.
Jersey City, N. J.	Troy, N. Y.
Johnstown, Pa.	Toledo, O.
Jacksonville, Fla.	Tacoma, Wash.
Kansas City, Mo.	Utica, N. Y.
Lawrence, Mass.	Washington, D. C.
Lynn, Mass.	Worcester, Mass.
Los Angeles, Cal.	Wilmington, Del.
Milwaukee, Wis.	Youngstown, O.

Methods and Cost of Constructing a Bituminous Carpet on Concrete Roads, with Notes on General Maintenance.

Contributed by F. W. Whitlow, Superintendent of Construction, Milwaukee County, Wisconsin.

BITUMINOUS CARPET CONSTRUCTION.

In Milwaukee County, Wisconsin, 58,000 sq. yds. of concrete road surface were covered with a bituminous carpet in 1913. The method employed in this work was as follows:

Method of Construction.—The surface of the concrete was first swept with steel brooms to loosen the heaviest of the accumulated dirt, after which it was again swept with fibre street brooms, these sweepings removing all the dirt except a thin film of dust, which is impossible to remove with street brooms, but which we found could readily be removed with ordinary house brooms. (It is astonishing the amount of dust that can be swept from a concrete road surface with house brooms after it is, apparently, perfectly clean.) The depressions in the concrete were then painted over with Tarvia A, after which they were brought to grade with crushed stone, using stone, as near as possible, equal in size to the depth of the depression. This stone was carefully tamped in place with hand tampers and the voids filled with a heavy bituminous road binder. For each square yard of surface about 1 gal. of binder was used to each inch in depth of stone.

When all depressions had been brought to grade in this manner, the entire surface of the pavement was then covered with Tarvia "A," heated to about 230° F. The tarvia was delivered to the work in barrels and was heated in portable kettles on the job. It was spread on the surface by hand with spreading cans, using about ⅓ gal. to each square yard of road surface. The tar was then covered with clean, sharp sand, using about 1 cu. yd. of sand to 100 sq. yds. of road surface, no attempt being made to dry or heat the sand. The surface was then left to the traffic to iron out.

This gave a surface that was resilient, noiseless and certainly added greatly to the life of the pavement, as it cushioned the impact from steel tires and horses' shoes and prevented chipping and wear from abrasion. Such a treatment also waterproofs the surface and prevents frost from acting on the cracks in the concrete. By keeping the moisture in the concrete uniform it tends to reduce the expansion and contraction to a minimum.

The roads treated were all without curbs and all intersecting roads were unpaved, resulting in considerable dirt being carried onto the surface of the pavement, there to be ground into the bituminous covering, this, of course, not adding anything to the life of the coating. The roads also carry a heavy mixed traffic which, at the end of one year, had worn through the bituminous carpet, leaving the concrete bear in spots. In the spring of 1914 some of the roads were again treated with Tarvia X, and from present appearances are going to give much better service than the Tarvia A treatment. Tarvia X is heavier and seems to adhere to the surface equally as well as the lighter material.

Cost.—The cost of placing bituminous carpet coats on concrete road surfaces in Milwaukee County has been about 10 cts. per square yard for labor and materials, this including the cost of cleaning the old concrete surface. This cost could be reduced considerably if the tar was bought in tank cars and applied with a pressure spray system.

CONCLUSIONS.

As a result of the experience obtained in this work the following suggestions are made in connection with the bituminous carpet treatment of concrete roads:

After thoroughly cleaning the surface of the concrete of all dust and dirt that has accumulated, and being assured that the concrete is thoroughly dry, spread evenly over the surface from ¼ to ⅓ gal. of bitumen per square yard. If Tarvia is to be used, I would by all means recommend the "X" grade. The Tarvia should be heated before spreading to about 230° F. Immediately after the bitumen is spread it should be covered with clean ¼ in. gravel or stone chips and rolled with a light road roller. After this course is spread and rolled, a second treatment should be given in the same manner as the first, using the same amount of material. This should build up the coating to a sufficient thickness to withstand the traffic for a considerable time.

It has often been stated that tar should be applied on very hot days. This, I believe, is not necessary, as we have had better success when the temperature of the air has been in the neighborhood of 55 or 60° F. A good, clean, dry surface is more essential than hot weather.

The treatment with a bituminous carpet of concrete roads that have proven defective through poor workmanship, bad materials, freezing or other defects, seems to be the most satisfactory method of repair in use at the present time. It will not build up the surface more than ⅓ in. at the most. It acts as a cushion, preventing wear on the concrete and prolonging the life of the pavement. It can be applied cheaply and quickly and does not tie up traffic on the streets for any great length of time, in fact, in most cases, traffic can be allowed on the street all the time that the repairs are going on.

GENERAL MAINTENANCE

Patching.—Patching with concrete has proven expensive and unsatisfactory, as it is necessary to keep traffic off the patch for at least three weeks, while the difficulty of properly caring for and curing several widely scattered patches is no small matter. It would be necessary to cut entirely through the old pavement and square up the edges of the hole before placing the patch. It is very difficult to secure a bond between the old concrete and the new. This would result in a new source of trouble, as the joint around the patch would soon break down from chipping, resulting in new holes. Experience has proven that an unprotected joint in a concrete road surface will chip and wear away very quickly under traffic.

Joints.—When concrete pavements are properly laid with proper materials, maintenance becomes easier. We find on our later work that it is only necessary to maintain joints and cracks which occasionally appear and now and then a small defective spot on the surface, which is usually caused by a small lump of clay or a chip of wood that floated to the surface when the concrete was poured.

In Milwaukee County we are filling the joints in the concrete as soon as the road is completed, using Tarvia X applied hot and covered with coarse sand or fine gravel. Just enough tarvia is used to fill the depression in the surface and extend about ½ in. onto the good surface. Great care is taken to thoroughly clean the concrete before pouring the tarvia, as dust or moisture on the surface will destroy the bond between the concrete and the bitumen. We are using for this work a crew consisting of 1 foreman, 1 team and 4 laborers. A crew of this size will, under average conditions, as found in Milwaukee County, cover all the joints, cracks and small pits in from one to two miles of 18-ft. road, in one day at a cost of from $15 to $30 per mile for labor and material.

Concrete Pavement Design.

To the Editors:—Engineers have written numerous articles during the past few years concerning concrete pavements, laying stress on the importance of workmanship and materials, and in their endeavor to secure the best results by better *workmanship* and *materials* they have almost ignored the more important item of *design*.

Concrete is subject to certain laws and its behavior has often been demonstrated, both in bridge and building construction. What are these stresses which cause defects in concrete pavements? Where are they applied? How great are they? How may they be overcome? These questions may be briefly answered as follows:

Expansion and contraction will cause transverse cracks in a pavement slab 6 ins. thick, provided the transverse joints are left 50 ft. apart, for experience has demonstrated this fact, and also the further fact that 25 or 30 ft. is a safe distance to space them. Joints so spaced and protected by metal plates will free such a slab from any danger of transverse cracks due to contraction and expansion.

The other important stresses tending to produce cracks have been found to act vertically. This vertical motion is produced by the movement of the foundation upon absorbing moisture and again upon drying out. It is produced by the settlement of the sides of the roadbed, where they are less compact than the center and, furthermore, this vertical movement is sufficient to cause the slab to act as a beam.

In every city may be found miles of concrete sidewalk blocks cracked longitudinally, and in every city where concrete pavements have been built there may be found a large percentage of the slabs cracked longitudinally. They all show a failure along the center, or, in cases of extreme width, probably ten feet out from the gutter line. Many hundreds of miles of such failures, all of them similar, teach but one thing, i. e., they were caused by forces acting vertically.

If a concrete sidewalk block 4 ft. long, 4 ft. wide, and 4 ins. thick does not crack longitudinally, and yet a block 4 ft. long, 8 ft. wide and 4 ins. thick does crack longitudinally, what is the reason? If a thousand of such blocks crack longitudinally, what is the reason? It isn't temperature stresses, contraction, expansion, unknown-factor, dished or crowned sub-base, lack of reinforcement, but simply lack of strength in the slab to withstand the vertical movements of the foundation. In other words, for every *width* of slab, a definite *thickness* is required. This law is absolutely universal and every longitudinal crack in Wayne County, Mich., Iowa, Illinois, or any other place, could have been prevented had the slabs been correctly designed.

It is a waste of time to listen to the argument that the cracks in these pavements were not caused by vertical movements in the foundation. These facts are known by too many engineers and yet, notwithstanding, we often hear the plea that we haven't any scientific knowledge that would lead us to expect such powerful vertical movements in the foundation.

However, we know the force of the vertical movements, for if they have caused uniform failures, and if they have been prevented by a uniform reduction in width of slab to meet the thickness, we know all the necessary facts required to correctly design such a slab to withstand such vertical movements.

One reason why engineers have devoted so little time to concrete pavement design is that pavements have been a gradual growth from sidewalks and private driveways and few people have realized that a concrete slab for such purposes requires a design. Also another reason is that the cement and steel people have designed either directly or indirectly many of the pavements and year after year they have proceeded with utter disregard to engineering principles.

The writer attended the National Conference on Concrete Roads, held last February in Chicago, and was in touch with many of the engineers there. It was observed that few came out with a frank admission of failure in design, and yet were more than anxious to learn a means of building concrete pavements without cracks, when it was intimated that such a thing was possible. In an article published in Engineering and Contracting in July, 1913, by The Morse-Warren Engineering Company of Carlinville, Illinois, the mathematics of design was discussed at some length, and a table presented showing the thickness required for certain widths. During the past year, the correctness of design outlined therein has been demonstrated to numerous engineers. While it is true that in rare instances a pavement may be built on a porous foundation, which will not be subject to vertical movements, it is poor economy to build any concrete pavement which is not designed to meet *general* conditions.

There are a few principles of design which, if followed, will produce at an economical cost, a concrete pavement free from cracks. They may be briefly outlined as follows:

1. Transverse and longitudinal cracks are caused by stresses which are greater than the strength of the concrete.

In other words, a slab of concrete, say 4 ft. square and 6 ins. thick, could withstand the forces of expansion and contraction and the vertical movements of the foundation, even though they should raise it several inches. It could stand the shock of travel and the action of the elements, in fact, such a slab would be free from defects.

If it is possible to design a slab free from defects, the next proposition is to secure an economical design. That the economical slab is much greater in size than one 4 ft. square has been often demonstrated, and that there is a definite economical size of slab will be more fully explained in this article.

2. Vertical movements in the foundation cannot be ecomonically prevented by additional rolling of the subgrade nor by drainage.

This principle will be more fully understood when it is known that some of the strongest efforts to reduce cracking have entirely failed where it was undertaken to put the subgrade in a permanent, immovable, and absolutely stable condition, free from moisture, and free from any tendency toward future settlement.

Take for instance the sub-grade at the edge of a pavement with a 6-in. tile laid 4 ft. deep under that edge. In our section of the country, today, the entire sub-grade is dry. Now suppose we have a season of real wet rainy weather, with water covering the ground in many places. The 6-in. tile will act promptly and proceed to carry off water, but at the same time the earth at the sides of the pavement will have absorbed moisture, causing it to swell and the earth under the center of the pavement being more dry will swell less, thereby causing unequal movement in the foundation and cracks in the pavement slab. This force again acts when the earth becomes drained at the sides more quickly than at the center.

The writer has observed slabs of concrete absolutely supported by a longitudinal ridge of earth, with the edges of the slab extended out into space, so to speak, and acting as a cantilever.

As said before, in this article, the proof of these vertical movements and the uselessness of extreme pains in rolling and draining have been too universally demonstrated not to be known by those seeking better design.

3. Reinforcement is a deception, as generally recommended.

Several scientific articles have been published in Engineering and Contracting during the last year, calling attention to the fact that any sufficient amount of reinforcement to prevent cracks in concrete pavements is prohibitive in cost; also it has been shown that the per cent of reinforcement now generally recommended and used is only about one-tenth that ordinarily required.

Many engineers have built reinforced pavements after having had failures in plain concrete pavement construction, and have not taken into consideration the fact that they are getting better results in their later work, not with reinforcement, but with better workmanship and material.

A representative of one of the large steel industries, after having inspected many miles of pavements, recently informed the writer that he had at last given up the theory that concrete pavements were to be saved from cracking by the use of reinforcement.

The vertical forces tending to destroy a concrete pavement act both upward and downward, and yet some material men have the boldness to advertise that a reinforcement placed 2 ins. below the top of a pavement will absolutely prevent cracks. If necessary 2 ins. below the top it is equally as necessary 2 ins. above the bottom, but the point to be made here is that all such so called reinforcement is not reinforcement at all; never was figured to be; and never can be anything but an extravagance and a deception.

4. For a given thickness of slab, with good workmanship and material, a definite width is required. The limit of width for a 6-in. slab, 1:1½:3 mix, is 10 to 12 ft.,

In every city you will find concrete pavements with longitudinal cracks, where the width is any greater for the thickness than that mentioned.

5. It is economical to make a longitudinal joint in a pavement slab rather than increase the thickness where the width is increased.

If 6 ins. is ample to give strength and durability sufficient for travel, it is poor economy to make the pavement any thicker simply to have a continuous beam or slab across the roadway. Such a longitudinal joint should be protected by a metal plate now manufactured for that purpose.

6. A joint in a concrete pavement, properly protected and covered by a metal plate, is not a weakness.

Engineers have in recent years thrown up their hands in horror at the idea of a joint down the center of a concrete pavement. The facts are that today thousands of taxpayers are throwing up their hands in "holy horror" at the joints in their pavements—joints not planned, either, and not protected by metal plates—but joints which are unsightly, damaging, and irregular, and each year a growing expense to maintain. As suggested a year ago in one of our leading engineering magazines, longitudinal joints are not to be avoided; they are an *absolute structural necessity.*

7. A factor of safety should be allowed in design.

Every pavement should be designed to meet average conditions, not ideal conditions. The methods of design herein outlined will produce a concrete pavement free from cracks—a pavement costing much less per mile than now recommended by engineers, and a pavement which contractors will with safety guarantee against cracking for a five-year period.

The attempt is frequently made to belittle the importance of cracks and to refer to them as of slight expense to maintain, but to those who will read the proceedings of the National Conference on Concrete Roads, held in Chicago, the attention given them will be appreciated. And yet with all the time devoted at the conference to this topic, there was no effective remedy agreed upon.

One of the leading engineering journals in commenting upon the results of the conference advises its readers as follows:

"Tradition is adhered to in design and in construction methods and material, in subgrade construction, expansion joints, and in mixing, placing and curing the concrete slab. This is wholly natural and proper and it lends emphasis to the few new thoughts of attraction for the more expert."

It is the belief of the writer that the design of concrete pavements and the proper protection of a longitudinal joint down the center of the pavement should now receive the attention of the "more expert," and that present knowledge of construction should be classified, with the thought in mind that concrete pavement design be redueed to a science.

W. D. P. Warren,
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Decatur, Illinois.

Results of French Experiments on the Transmission of Pressure Through Macadam to the Subgrade.

The following notes on the transmission of pressure through macadam to the subgrade were made by W. de H. Washington in the last annual report of the New York Highway Commission:

French tests on the amount of pressure exerted through the road on the subgrade by a wheel load of 4 tons, with a 5½-in. tire give the following results:

On macadam alone:					
Thickness of crust, inches	1.97	3.94	5.91	7.87	11.81
Pressure on subsoil, lbs. per sq. in.	102.5	47.7	27.4	17.4	9.1
On foundation alone:					
Foundation thickness, inches	....	....	5.91	7.81	11.81
Pressure on subsoil, per sq. in.	....	....	56.0	37.4	20.7
On combined foundation and macadam:					
With foundation thickness, of, ins.	....	5.91	7.87	9.84	11.81
3.94 in. of stone, pressure on subsoil, in lbs., sq. in.	....	19.3	14.7	12.6	10.2
5.91 in. of stone, pressure on subsoil, in lbs., sq. in.	....	13.2	10.9	9.1	7.7
7.87 in. of stone, pressure on subsoil, in lbs., sq. in.	....	9.7	8.2	6.8	6.1

The pressure on the subsoil through a 12-in. bed of simple macadam is apparently the same as the pressure through a 6-in. bed of macadam laid over a 10-in. stone foundation.

It is considered that poor soil requires at least a 12-in. macadam layer or its equivalent.

Cost of Tree Planting in Queen Victoria Park, Canada.—The report of the Commissioners of Queen Victoria Niagara Falls Park states that in connection with the planting operations an experiment was started on the sections of stiff clay where the ordinary methods were likely to prove unsuccessful, the soil being so impervious as to prevent drainage, proper aeration, and the ramification of roots. Holes dug with the spade were useless and dynamiting was, therefore, resorted to, the 40 per cent grade being used. Fissures for drainage and aeration were thus opened, and the soil shattered, although not actually ejected. The data relative to the foregoing experiment will, when published, be interesting crease on maintenance, for as the trees, shrubs and lawns approach maturity they must be properly cared for, otherwise dilapidation will work of ornamentation in sight, the expenditure will decrease on capital account, and in- and instructive. For tree planting purposes considerable quantities of soil were filled into the dynamited holes to afford a rooting medium until the surrounding area will through cultivation and aeration be brought into a fertile condition. The approximate cost of constructional operations and items are as follows:

Item.	Cost.
Filling, grading, harrowing and seeding during the year 1913	$14,000.00
Four miles were completed, the cost per mile being	3,500.00
Planting, staking, mulching and pruning trees	3,000.00
Trees (2,000 planted)	1,000.00
Average cost of established tree	2.00
Initial cost of each tree	.50
Four miles of boulevard were planted, the average cost of ornamentation per mile being $4,500, or a total of 4 miles of	18,000.00

For the year 1913 the cost of maintenance of the sections ornamented in previous years amounted to $1,800. This, however, must not be used as a criterion of the cost of subsequent years as with the completion of the ensue.

Civil Service Examination for County Road Supervisors in Wisconsin.—Civil service examinations have been conducted by the state highway department of Wisconsin in various counties during the past month. The county boards are required to hire one of the two highest eligibles for highway commissioner.

The points on which the examinations are graded are as follows:

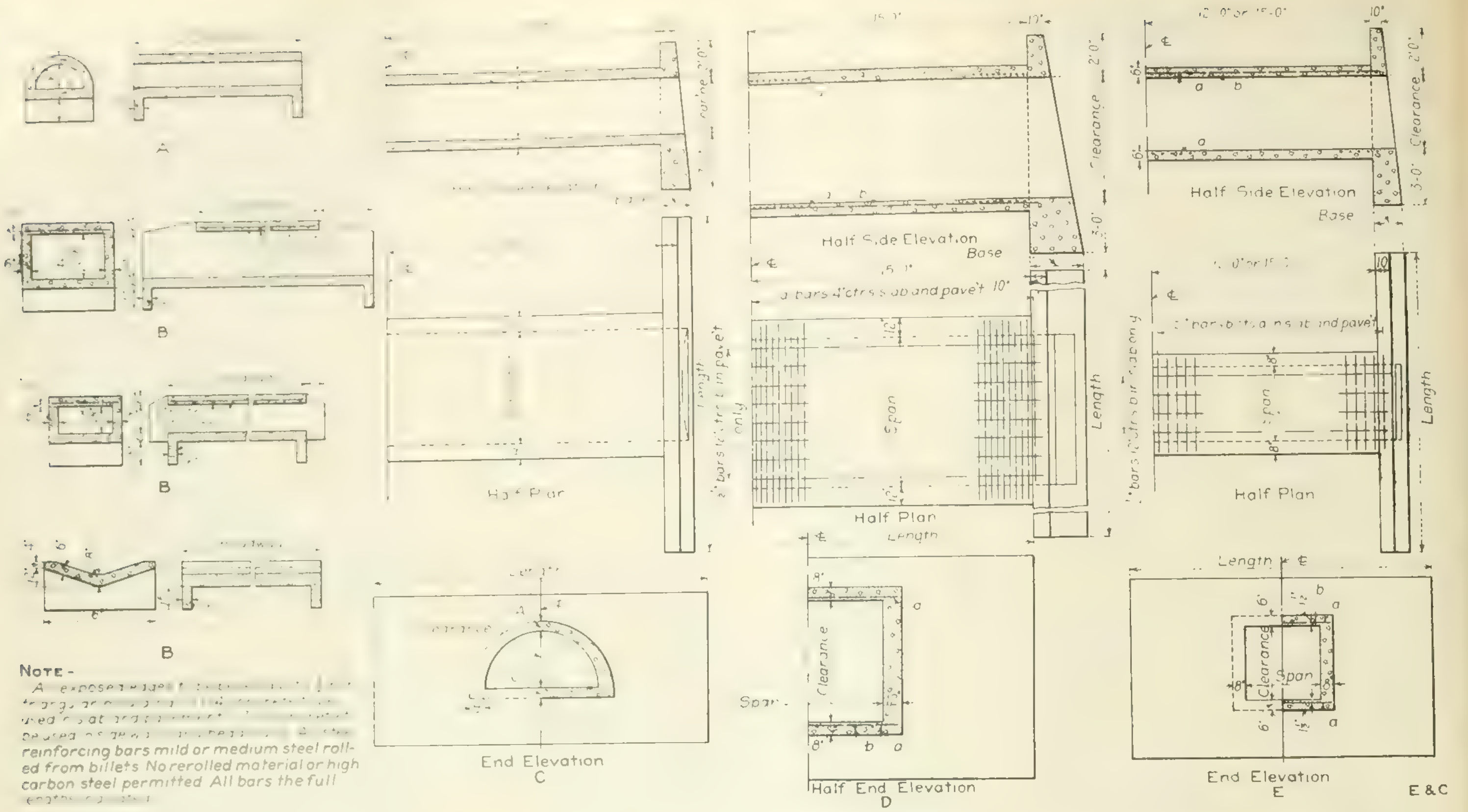
Physical ability, industry, sobriety, 10; general education, 6; knowledge of mathematics and computations, 10; knowledge of state highway law, 4; business experience and ability, 10; personality and indications of leadership, 12; familiarity with county, 4; engineering and surveying, 8; experience in road-building and work of allied nature, 20; experience in selecting and handling stone and gravel, 6; experience with heavy machinery, 4; culvert and bridge construction, 6.

Candidates for the examinations need not necessarily be residents of the county, or even of the state.

The standings of candidates who pass the tests are submitted to the county clerk. No candidate is eligible who has a standing of less than 70 per cent.

Wood Block Paving in Honolulu.—Wood paving block is specified by the Departmeent of Public Works of Honolulu, as one of five kinds of paving that may be laid in Honolulu streets. Approximately 100,000 Douglas fir blocks have been laid in the city streets. The size of block that is thought best adapted is 3x4x4 ins. A bid submitted by a contractor during the middle of October, 1914, named $1.962 per square yard for Douglas fir blocks, furnished and placed, including foundation.

Ohia also has been used for paving block and, being a native wood, has probably been favored. No large amount of wood-block paving remains to be done in Honolulu, but much will be done in the future in the next largest city, Hilo.



Standard Small Culverts Recommended by the Illinois Highway Commission.

The drawings shown on this page are prepared from standard plans used by the Illinois Highway Commission, Clifford Older, bridge engineer. The tables give the quantities of materials required. The drawings and tables are self-explanatory.

A—HALF ROUND PRIVATE ENTRANCE DITCH CROSSINGS.

Radius, Ft. Ins.	Roadway, 12 ft. Cu. yds. concrete.	Roadway, 16 ft. Cu. yds. concrete.
1′ 0″	1.67	2.18
1′ 1″	1.82	2.38
2′ 0″	2.91	3.80
2′ 6″	3.53	4.61

C—SPAN OF 2 TO 10 FT., ROADWAY 30 FT.

Diameter [illegible]	Clearance [illegible]	Thickness of arch [illegible]	Thickness of arch [illegible]	[illegible]	Head wall Length, feet	Head wall Base, inches	Plain concrete, cu. yds.
24	12	[illegible]	[illegible]	6	8	10	6.7
[illegible]	[illegible]	6	[illegible]	6	10	12	8.9
[illegible]	[illegible]	[illegible]	[illegible]	6	13	14	13.7
[illegible]	[illegible]	[illegible]	[illegible]	6	16	16	18.4
[illegible]	[illegible]	[illegible]	10	[illegible]	18	18	[illegible]
[illegible]	[illegible]	[illegible]	12	6	[illegible]	21	[illegible]
120	60	8	12	6	28	27	49.0

B—DITCH CROSSINGS FOR PRIVATE ENTRANCES.

Size, ft.	Roadway, 12 ft. Steel. No. of bars.	Length.	Lbs.	Concrete. Cu. yds.	Roadway, 16 ft. Steel. No. of bars.	Length.	Lbs.	Concrete. Cu. yds.
				1.79				2.26
1′ 6″ x 2′ 0″.	24 2	2′ 6″ 11′ 6″	71	2.30	32 2	2′ 6″ 15′ 6″	94	2.97
1′ 6″ x 3′ 0″.	24 3	3′ 6″ 11′ 6″	100	2.80	32 3	3′ 6″ 15′ 6″	134	3.61
2′ 0″ x 3′ 0″.	24 3	3′ 6″ 11′ 6″	100	3.05	32 3	3′ 6″ 15′ 6″	134	3.95
2′ 6″ x 3′ 0″.	24 3	3′ 6″ 11′ 6″	100	4.13	32 3	3′ 6″ 15′ 6″	134	5.09
2′ 6″ x 4′ 0″.	24 4	4′ 6″ 11′ 6″	131	4.73	32 4	4′ 6″ 15′ 6″	175	5.84
3′ 6″ x 4′ 0″.	24 4	4′ 6″ 11′ 6″	131	5.40	32 4	4′ 6″ 15′ 6″	175	6.66
4′ 0″ x 5′ 0″.	24 5	5′ 6″ 11′ 6″	160	6.28	32 5	5′ 6″ 15′ 6″	215	7.76

D—BOX CULVERTS 6 TO 10-FT. SPAN, ROADWAY 30 FT.

Span, ft.	Clearance, ft.	Head walls. Length, ft.	Base, ins.	Reinforcing steel. Letter.	No.	Size.	Length.	Weight, lbs.	Plain conc., cu. yds.	Reinf. conc., cu. yds.
6	3½	20	22	a b	184 12	½″* ½″	7′ 9″ 17′ 0″	1,385	22.5	11.9
6	5	24	28	a b	184 12	½″ ½″	7′ 9″ 17′ 0″	1,385	35.9	11.9
7	4	22	24	a b	184 14	½″ ½″	8′ 9″ 17′ 0″	1,570	26.9	13.4
7	5½	27	30	a b	184 14	½″ ½″	8′ 9″ 17′ 0″	1,570	42.5	13.4
8	5	26	28	a b	184 16	⅝″ ½″	9′ 9″ 17′ 0″	2,610	37.1	14.8
8	6½	31	34	a b	184 16	⅝″ ½″	9′ 9″ 17′ 0″	2,610	56.1	14.8
9	5½	29	30	a b	184 18	⅝″ ½″	10′ 9″ 17′ 0″	2,885	43.9	16.3
9	7	33	36	a b	184 18	⅝″ ½″	10′ 9″ 17′ 0″	2,885	63.3	16.3
10	6½	33	34	a b	184 20	⅝″ ½″	11′ 9″ 17′ 0″	3,160	57.6	17.3
10	8	37	40	a b	184 20	⅝″ ½″	11′ 9″ 17′ 0″	3,160	80.2	17.8

*Square.

E—BOX CULVERT 1 TO 5-FT. SPAN.

Span, ft.	Clearance, ft.	Head walls. Length, ft.	Base, ins.	Roadway 30 ft. Reinforcing steel. Letter.	No.	Size.	Length.	Weight, lbs.	Conc., cu. yds.	Roadway 24 ft. Reinforcing steel. Letter.	No.	Size.	Length.	Weight, lbs.	Plain conc., cu. yds.	Reinf. conc., cu. yds.
1	1	7	10	[illegible]	100 4	½″* ½″	2′ 3″ 13′ 6″	240	[illegible]	a b	124 4	½″* ½″	2′ 3″ 16′ 6″	293	4.01	2.59
[illegible]	1½	10	12	[illegible]	100 4	½″ ½″	3′ 3″ 13′ 6″	[illegible]	8.9	a b	124 4	½″ ½″	3′ 3″ 16′ 6″	399	6.43	3.70
[illegible]	[illegible]	11	14	[illegible]	100 4	½″ ½″	3′ 3″ 13′ 6″	320	10.7	a b	124 4	½″ ½″	3′ 3″ 16′ 6″	399	8.37	3.70
[illegible]	[illegible]	12	14	[illegible]	100 [illegible]	½″ ½″	4′ 3″ 13′ 6″	430	12.0	a b	124 6	½″ ½″	4′ 3″ 16′ 6″	532	8.75	4.82
[illegible]	2½	13	16	[illegible]	[illegible]	½″ ½″	4′ 3″ 13′ 6″	430	14.0	a b	124 6	½″ ½″	4′ 3″ 16′ 6″	532	10.95	4.82
4	[illegible]	13	14	[illegible]	100 [illegible]	½″ ½″	5′ 3″ 13′ 6″	540	13.3	a b	124 8	½″ ½″	5′ 3″ 16′ 6″	665	9.12	5.93
4	[illegible]	16	18	[illegible] b	[illegible]	½″ ½″	5′ 3″ 13′ 6″	540	18.3	a b	124 8	½″ ½″	5′ 3″ 16′ 6″	665	14.52	5.93
[illegible]	[illegible]	17	18	a b	[illegible] 10	½″ ½″	6′ 3″ 13′ 6″	650	19.7	a b	124 10	½″ ½″	6′ 3″ 16′ 6″	799	14.94	7.04
[illegible]	4	20	22	a b	[illegible] 10	½″ ½″	6′ 3″ 13′ 6″	650	26.3	a b	124 10	½″ ½″	6′ 3″ 16′ 6″	799	21.82	7.04

*Square.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design and to Methods and Cost of Construction

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The Contractor and Management Engineering.

Close attention to technical literature in recent years has given the editor the impression that there is a growing sentiment on the part of engineers in favor of the day labor as opposed to the contract system of handling public works construction. This trend is due, largely, to a desire to save the contractor's profit. Much of the reasoning in this connection is fallacious, as we have many times pointed out. Whether the trend mentioned is for the ultimate public good or not its immediate effect should be to put the contractor on his mettle. It indicates the extreme desirability of reducing contract prices, in some localities at least, and this, of course, will necessitate the reduction of costs to the contractor. This, in turn, will call for the more general adoption by contractors of the principles of scientific management or, as we prefer to call it in this field, of management engineering.

In this issue we publish an article which describes the results of applying the principles of management engineering to the cleaning of filter sands at the several slow sand water filtration plants in the city of Philadelphia. This kind of work is, of course, usually done by day labor, but the contractor reader will find much of interest in the article to which we refer. However, it is the introductory paragraph of the full paper, from which our article is taken, that we desire here to call to the attention of contractors. The paragraph follows:

> Efficiency in municipal government will come about only as the work in the various departments is put on a basis which gives each man, from the common laborer up to the skilled artisan and clerk, a well-defined task to do in a given time, with a definite reward for its accomplishment. Under the ordinary methods of handling city work it is cheaper where there is fair competition to let work by contract than to handle it by day labor. With an effective system that eliminates not merely favoritism, but also presents a definite incentive for each man to do a fair day's work, a city may save the contractor's profit, employ its own force of city men, and avoid one of the largest sources for mulcting a city treasury through collusion between the city officials and contractors.

The paragraph quoted relates to maintenance and operation as well as to construction. Very likely the introduction of set tasks, piece work and the bonus system of payment will do much to promote economy and efficiency in the maintenance and operation of city works and in doing the work of city departments; we are not at present concerned with these matters. Moreover, we agree with Mr. Thompson that "under the ordinary methods of handling city work it is cheaper where there is fair competition to let work by contract than to handle it by day labor." The point in the quotation of particular interest, however, is the expressed belief that with an effective day labor system, of handling construction work, which presents a definite incentive for each man to do a fair day's work, the city may save the contractor's profit. There is truth in this also, for obviously day labor plus management engineering is more likely to produce good work at low cost than is day labor alone. On the other hand, it is equally true, and this is the point we wish to emphasize, that the contract system plus management engineering is better than the contract system alone.

In general, we believe in the greater economy of the contract system as opposed to the day labor system. This belief is not based so much on comparative figures, although these when truly comparable show to the advantage of the contract system in a large majority of cases, as on the principles involved. The day when the city officials and employes will have the same incentive that the contractor now has may come, but as yet it is an uncertain but long distance in the future. The contractor now has, in full measure, the money incentive and the money mentioned is *his* money, not the city's. This incentive is so much greater than is likely ever to be held out to a city official that we believe that the contract system will always be more economical than the day labor system, provided that the contractor is as good a manager as the engineer representing the city on a day labor project.

As we view the matter the contractor is now confronted with the necessity of lowering his prices and this means he must lower his costs. He must lower his prices, in the first place, on account of the ever-present and always-strong competition, actual or potential, of the day labor system. Advocates of the latter system sometimes succeed in making it appear to better advantage than the contract system by means of more or less frenzied bookkeeping. This is a condition with which the contractor must contend.

Even though the present run of contract prices should be maintained, it is incumbent upon the contractor to reduce his costs. This point was well brought out in the article on water works valuation published in our issue of December 16. Speaking of over-head charges allowed for by the experienced manufacturer, the author, Mr. J. W. Ledoux, said:

> The same principle applies to a general contractor of waterworks; pipe, trenching, lead, yarn and labor are only part of the actual cost to the contractor. He has his plant to maintain, his managing force, his office and practically must consider the same items as are included in the estimate of a skillful manufacturer, and that he does not include such items is proved by the fact that it is only a question of time before nearly all these waterworks contractors become bankrupt.

The tendency to report only the physical costs, labor and material, plus the expense of direct supervision, by advocates of the day labor system, probably prompts some contractors to approximate the same faulty method of estimating preparatory to making bids. The temptation to figure in this slip-shod manner is no doubt augmented by the necessity of keeping down the bidding price to where there is a fair chance of obtaining the work. The hard fact remains, however, that the contractor must allow for extra and overhead expenses of all kinds or eventually fail, as Mr. Ledoux points out.

In the foregoing the disadvantages under which the contractor operates are considered and but for a single saving advantage one might almost conclude that the business of contracting will always be one in which financial disaster eventually overtakes the majority of those who engage in it. The saving advantage is that the contractor has only himself to convince before adopting the principles of management engineering on his construction work. In view of the fact that in this way only can he make enough profit to enable him to continue in the business indefinitely, he ought to be easily convinced.

In closing, just a word as to Mr. Thompson's statement to the effect that the contract system facilitates the debauching of the public treasury. It is really necessary to go farther back than that in the eradication of graft. Bad morals are very elastic and readily adjust themselves to changes in forms of government or to changes in the methods of administering the public affairs. The disposition to graft is the thing to eradicate and not the chance vehicle of the grafter. Where city officials or employes are dishonest they can graft quite as readily under one plan of doing public work as under another. For instance, while writing the foregoing, a contributed article was handed to the writer. It tells how the municipally owned water works in one of our large cities checks its payrolls. The plan of checking was devised following "the betrayal of his trust by a foreman of a construction gang by a method of juggling his payroll returns."

The Engineer and the Public.

In this issue we publish two articles of exceptional merit pertaining to the relations which now exist and those which should exist between the engineer and the public. Engineers have long complained of a lack of appreciation of their services on the part of the public. They have also complained of poor salaries and other things which spring directly from this lack of mutual understanding between themselves and the public. The belief is rapidly growing that the engineer has only himself to blame that conditions are as they are, and plans for the promotion of a better understanding are beginning to take definite form. The article on the publicity work of the Cleveland Engineering Society describes the various publicity measures employed by that progressive body during the past two years and outlines the results effected.

The fact that the Society mentioned readily secured the co-operation of the local newspapers in its publicity work is a fact of the highest significance. Engineers have often wondered why the large dailies pay so little attention to matters of an engineering nature. If we may judge from the statement of the managing editor of one of the Cleveland papers the explanation of this condition of affairs is exceedingly simple. He said engineering news does not ordinarily get into the newspapers for the reason that the average reporter lacks the technical knowledge to handle it satisfactorily. The Cleveland papers gladly accepted for publication publicity matter prepared for that purpose by a committee of the Society.

While probably the most important educational agency in this connection is the newspaper there are other agencies for the popularizing of engineering. These, also are described in the article mentioned. Engineering society members and workers will no doubt be interested to note that the publicity measures at Cleveland had a direct effect upon engineers as well as laymen. This is conclusively shown by the fact that the membership increased over 50 per cent in two years. This is a truly remarkable growth in a society more than 30 years old. It indicates that there are many engineers who hold to the view that it is high time that engineering societies should broaden their field of activities.

The other article relates to the education of the public in municipal engineering matters. In view of the fact that engineers have a pronounced tendency to commiserate with each other when discussing their relations with the public the two following sentences from the second article are refreshing: "I am not one of those who feel that all our short-comings are 'the fault of the people.' I would rather assume my share of the responsibility for conditions as they are and then join with my professional associates and the community at large in bettering them."

GENERAL

The Engineer and Publicity, with Special Reference to the Publicity Work of the Cleveland Engineering Society.

People, generally, are interested in the work of engineers. The public wants to know of advance in engineering science and also what men make it possible. There is also a general desire to give credit where it belongs. There is also a duty, on the part of engineers, to give the public dependable technical information. The ordinary newspaper uses the same reporter to write about crime, politics, sport, invention and technical achievement. An editor of a Cleveland daily paper once expressed the opinion that one reason why news of an engineering nature does not get into the daily papers is because the ordinary reporter has not the technical knowledge to handle it.

The public and the engineering profession have something to exchange and both sides will receive substantial benefit in the exchange. The profession will find itself in a better position by having the public appreciate the important service it is rendering; the public will find itself deeply interested in the information we are able to give, because it advances public welfare. Here, then, are the two elements of a bargain. This subject was discussed by Mr. C. E. Drayer, chairman of the Publicity Committee of the Cleveland Engineering Society in a paper before the annual meeting of the American Society of Mechanical Engineers. The following paragraphs are from that paper:

PUBLICITY WORK IN CLEVELAND

Let us review the publicity work of the Cleveland Engineering Society, extending now over a period of two years. Our experiences there are the same as might be expected in any similar situation. Our first step was to get acquainted with the managing editors of the two leading papers in the city. They were told that the society had about 500 members, many of whom are at the head of large undertakings on which the growth and prosperity of the city depend. They were told that the society stood ready to co-operate with them in obtaining such engineering news as would be of interest to the community. We were assured that the newspapers welcomed our assistance, and were advised to have the society try being its own reporter.

It so happened that the outcoming issue of the society's journal contained the report of a special committee on technical education in Cleveland. Naturally, a large number of people in the city were interested in what engineers had to say about their technical schools. Abstracts to make about three-quarters of a column were written, ready to set in type, and were handed to the editors of the two morning papers. They were printed without alteration. A third paper printed its own abstract and an editorial. If the committee had something to say which the public would be benefited in knowing, surely 200,000 papers with authentic information was a better medium than the 1,500 copies of The Journal read largely by technical men.

Another subject of great interest to the public, although it might not appear so from the title, was discussed in R. H. Fernald's paper on The Relation of the Gas Producer to Low-Grade Fuels and Concentration of Power at the Mines. When an abstract appeared in the paper on the Sunday following the lecture, it was headlined as shown in the accompanying bold-face type.

Reference to the paper printed in the Journal of the Cleveland Engineering Society will bear out the assertion that the headlines do not exaggerate the statements made by Dr. Fernald. Our task was merely to make news of plain facts. To the citizens of any large industrial center like Cleveland, smoke elimination and fuel conservation are mighty live questions. News is the turning of facts and ideas into interesting reading.

Probably the largest service to the community performed by the publicity committee of the Cleveland Society was the publishing of 14 inspirational articles by prominent local engineers on Engineering as Life Work. The articles appeared on successive Sundays in the magazine section of a local paper. Beneath the title of each was an editor's note stating the purpose of the series and that they were published under the auspices of the society. Among the contributors were two past-presidents of The American Society of Mechanical Engineers and a past-president of the American Railway Engineering Association. The theme of the series was to tell the young man about to choose life work what is before him in the various branches of the engineering profession. Besides appearing in the local paper, most of them were published in the Scientific American and the Case Tech, the student publication of Case School of Applied Science. Some appeared in other periodicals over the country. A young immigrant wrote to one of the contributors and asked for permission to translate his paper into French and Russian to send to those countries.

PRODUCER-GAS TO ELIMINATE SMOKE AND SAVE FUEL

United States Engineer of Mines Tells Local Society of Broad Conservation Plan.

MILLIONS OF TONS OF COAL WASTED YEARLY

Production of Gas at Mines from Coal Now Unmined Would Solve Problem.

While we might multiply instances like the above where our work was distinctly a service rendered the public, we shall pass to an enumeration of the tangible benefits to the society and to the profession growing out of the publicity work.

A higher standing in the estimation of the people and of those in authority in the affairs of the community is one gain. In Cleveland, the co-operation of the society is usually sought in the solution of questions of public welfare were engineers are qualified to speak. To cite an instance, the Civil Service Commission early in the present year asked the society to assist it by taking charge of the preparation and marking of papers for engineering positions. The first request was for ten examinations, and the results to both the Commission and to the society were very gratifying. The Commission secured the service of experts at no cost to the city, but which were worth more than it had available funds to employ. The secretary of the Commission told us that the candidates were satisfied with the fairness of the examination. Concerning previous examinations complaints have been made that proper relative weights had not been given to experience and theoretical training. Our publicity committee saw to it that the public learned through the newspapers of the arrangement between the Civil Service Commission and the Engineering Society. Credit was given where it was due.

It is hardly possible that any sudden gust of public disapproval would arise where engineers are concerned if the public felt that it was well acquainted with them. When the local society engages in publicity work, a revival in the interest of its members in the activities of the society will be apparent. The indifferent members find they have some pride in their society when its activities, of which they approve, are described in the daily paper. It is said on good authority that the publicity campaigns undertaken by Memphis and Des Moines, to present to business men their advantages for a location, resulted in a renewed city spirit equal in value to the new business acquired.

Due largely to the publicity work, there has been a substantial increase in percentage of attendance at the meetings. One estimate was 15 per cent. To stimulate attendance, the committee furnishes the papers advance notices of the meetings, consisting of a picture of the speaker and some 150 words of text telling about him and his subject.

During the last two years some 250 new members have been added to the roll, an increase of over 50 per cent in a society past 30 years old. Of course, it is difficult to say just what per cent of increase in an organization recently very active in all its functions may be credited to publicity work.

ANALYSIS FOR EXTENDING THE WORK.

We have enumerated some of the benefits accruing to the profession and to the public from publicity work conducted along a definite plan in one locality. Let us now turn our attention to a statement and suggestion of a solution of the problem in its application to a broader field.

It is possible at this time to know only the general nature and approximate limits of publicity with any degree of accuracy. They may be determined in accordance with the laws of psychology. For instance, it is the province of the educator and psychologist to formulate the laws involved in the mental processes between appeal and response. They are also able to measure memory in a practical manner. It is sufficient to say that considerable study should be given to the technical aspect of our problem before active work in a large way is undertaken.

We can, however, enumerate the various channels by which information of an engineering nature may be placed before the public. It is also possible to give approximate relative values to them. In the matter of choosing mediums we are inclined to lay down this broad general principle: When one man has something to tell another, the telling of which will do them both good, he may employ the most direct honorable means. It may be either the written or spoken word.

Under the written word we would include newspapers, periodicals, such as national magazines, and pamphlets. The spoken word would be confined to a rather narrow field and would consist for the most part of talks by engineers before high school classes, classes in Y. M. C. A.'s, and lectures before clubs, at special gatherings in churches and the like.

Newspapers.—Everybody reads. If they read nothing else, they read newspapers, so it is the most direct way to reach the greatest number. An important point in favor of the newspaper is that it is local and offers an opportunity for the local engineering society to place in its columns engineering news that has an especial appeal to the community. Most people have a friendly attitude toward one of their local papers and are ready to take to heart what they read in its columns, particularly when they know the source is authentic. So the community will learn about its citizens who are engineers and what they are doing to promote its welfare. For these and other reasons, such as frequency of appeal, we are inclined to give newspapers the place of first importance in publicity work for engineers.

National Magazines.—The national periodical or magazine has a quality of permanence which the newspaper lacks. Each issue is likely to be before the public for from a week to a month, whereas, at least in our large cities, each issue of the newspaper is being read but part of one day. On the other hand, care must be taken in the selection of magazines for publicity purposes unless the plan is to reach a particular class. If a magazine has an article on a special subject, it is unlikely that any considerable space will be allotted to the same subject during the same year. It has been demonstrated that the first appeal must be reinforced by subsequent ones for the memory to retain a definite impression.

Pamphlets.—Pamphlets or monographs possess an important advantage over newspaper and magazine publicity in that their contents do not have to pass a possibly unsympathetic censorship. They can be placed directly and quickly into the hands of those for whom they are intended. While they should have attention-holding power able to compete with the newspapers for the readers' interest, they are free from the competition due to a multiplicity of appeals existing in every magazine and paper. It is probable that effective work could be done by using pamphlets for engineering publicity.

Lectures by Engineers.—It is doubtful if memory is more retentive and the mind more plastic at any time in life than during the high-school age; also at that time information is eagerly sought for making a choice of lifework. Here lies an opportunity to tell the future leaders in all walks of life what it means to be an engineer as well as to warn away those who have been captivated merely by its romantic side.

In every city there are a great many opportunities to talk before clubs of many kinds. We are aware that a good deal is being done in the way of popular lectures on scientific subjects, but more might be done, as a part of our service to the community. Where possible the local engineering society might let it be known that it could by arrangement furnish speakers on certain subjects.

Miscellaneous.—There remain a variety of ways by which information may be placed before the public. For the most part they have not been analyzed, some not even suggested. In this miscellaneous class may be mentioned the exhibit and moving pictures. We are inclined to believe that both have possibilities which would be discovered when once the work was undertaken.

CONCLUSIONS.

We have shown that the public and the engineering profession are in a position to make an exchange at a profit to both parties. A record of results in one locality where it has been tried points to what may be expected through co-operation in a larger field. Mediums of exchange have been discussed. There remains yet to be suggested a preliminary plan by which systematic and effective work may be done.

Inasmuch as all the profession will share in the benefits of a closer relation with the public, we assume that the efforts of all should be united on a common ground. More definite plans may be worked out by representatives of leading national engineering organizations at such a time and place as is deemed best. In general we believe that the local society working in co-operation with a central national organization will produce the most satisfactory results.

Educating the Public in Municipal Engineering Matters.

The role which the engineer is to play in the development of our municipalities will depend primarily upon the attitude taken by the profession as a whole toward what appears to be a wonderful present opportunity and also upon the ability with which the work of the engineer is brought to the attention of the public. There is no real reason why municipal engineering should not be made to comprise most municipal undertakings.

There was recently held by the American Society of Mechanical Engineers a Public Service meeting. At that meeting the possibilities of engineers in the public service were considered from several viewpoints. Mr. Morris L. Cooke, Director of the Philadelphia Department of Public Works, read a paper entitled: Some Factors in Municipal Engineering. The following paragraphs are quoted from that paper:

In inviting the attention of our profession to the municipal field, we are apparently opening the door of opportunity to tens of millions of dollars worth of work which is not now either considered engineering nor carried on by engineers. The municipal field is almost virgin soil so far as engineering is concerned. As recently as ten years ago the problem of snow removal which is being discussed as a part of this day's program, was so absolutely in the hands of thumb rule, and in many instances even of inexperienced men, that it is probably true that in no city in this country was it being attacked either by engineering methods or by engineers. Yet it will not be denied that on work of this kind, in which one city spent nearly $3,000,000 in six weeks last year, there is in reality an engineering problem of considerable magnitude.

If this municipal field is to be one in which engineers of ability, sincerity of purpose and high ideals are to find a permanent and satisfactory outlet for their energies, our profession acting as a profession will be one of the main agencies bringing about certain fundamental changes in the attitude of the public. In the minds of too many engineers, participating collectively in matters pertaining to municipal engineering means "getting into politics." Architectural work being a part of the business of the Department of Public Works in Philadelphia, we have had the co-operation of the American Institute of Architects and of its Philadelphia chapter from the beginning. We have made many demands on them and in every case have met with generous and enthusiastic response. This assistance has been entirely apart from the help rendered us by individual architects. We have had the constant, indefatigable and valuable support of the secretary of the American Society of Mechanical Engineers in our efforts to maintain the highest professional standards in the work of the department. But engineering bodies as such have given us no assistance and so far as I know have taken no part in the discussion of federal, state and municipal engineering except in the matter of conservation which for some reason is considered as innocuous from an engineering standpoint as a prayer-meeting.

Many municipal engineers in this country are beginning to adopt the European system of employing non-residents for certain highly specialized positions. Whenever this is practiced it excites criticism and abuse. As yet no technical organization, so far as I know, has recognized the opening thus made for technical merit and given moral support to the movement. Again, I have tried to get support from organized engineers in the obviously necessary procedure of employing experts outside our regular staff, but without results.

From an engineering standpoint any scheme of highway construction which does not include an ample repair program, is futile. In our community this is not at all understood because the plan has never been followed. It would seem a proper function of an engineering body to educate the community to this point of view. But as a matter of fact engineers as such have taken so small a part in public discussions that the layman with us settles more engineering questions than the engineers themselves and the taxpayer foots the bills.

The public today is undoubtedly impatient for "results." It has an unmistakable liking for men who "do things." It has little patience for experimental work. There was introduced in our Philadelphia City Councils an appropriation for $50,000 for an experimental test roadway and the bill was defeated on the ground that I had stated that the man who was to spend the money was one of the best experts on bituminous road work in the country. It was held that this statement and the appropriation were inconsistent—that either the engineer was less expert than I had claimed or that we did not need an experimental test roadway. The title of the road in the appropriation bill was changed to read "service test roadway" and the bill was passed by a good majority.

The public must be taught that public service is not different from private service in that forward steps come frequently, even usually, as the result of a large amount of preliminary investigation. Again, the public, of which please remember we are a part, must be educated to place more responsibility on individuals, thus making it possible to do away with the great inefficiencies which inevitably accompany board and committee management. As long as we have boards and committees they will vote—and they will insist on voting—on matters that are not questions of personal opinion, but questions of facts which ought to be determined by the facts. It is one of our duties as technical men to carry on a propaganda which will show to the public the difference between those problems of policy and public interest, that are properly settled by public opinion and those scientific problems which are improperly settled unless they are settled according to the facts. Notwithstanding all the boards and commissions that are created in the generally approved laws of today, there should be no uncertainty as to what questions they may vote upon. It is therefore one of the duties of the educated to carry this message to the people and in doing so I do not think there will be any more powerful method than to give the great mass of the people a larger and larger knowledge of expert work.

Lord Kelvin is the authority for the statement that the physicist who has discovered some great principle or invented some new process in physics should be able to explain it to the first man he meets on the street—both what the discovery is and why it is useful; or failing in this he should go back to his laboratory and put in some more time. If Lord Kelvin could say this about discoveries in physics, surely our city officials and the public service engineers with whom they co-operate should take the position that their work is capable of being fully explained to those who are footing the bills.

I am not one of those who feel that all our short-comings are "the fault of the people." I would rather assume my share of the responsibility for conditions as they are and then join with my professional associates and the community at large in bettering them. If we engineers are to have any prominent part in this there are fundamental changes which we shall have to make in our own equipment for the work. In the first place, we have to get rid of the now old-fashioned idea that advertising is a crime. I admit that as a part of my work as a public official I put in a great deal of thought on what may be quite properly called advertising. By that I mean that I pay less attention in my reports to dignity of form and diction than to making them sufficiently interesting to be read. It is only as we engineers who are public officials learn to make the public, sometimes against its will, understand our work, that we are to get that degree of popular support for it which will make it possible for it to be done in an efficient manner.

In my opinion it is going to become more and more a necessity, not only in public but in private work, for engineers to be able to popularize what they are doing. It is true today that a man who wants to do really good and efficient work can do so only after an aroused public opinion. You cannot drive people in a democracy. So I admit that in offering employment to an engineer, other things being equal, I want what might be called a good advertiser. You can secure appropriations for work more easily when it is well advertised. The Panama Canal is a good example of this principle. Again, advertising is the best possible check against ill-advised expenditures. In building our By-berry and Bensalem Service Test Roadway we erected sign-boards on each of the 26 sections, giving the layman the exact method of its construction in non-technical language. If the public knows how a street is supposed to be constructed or cleaned, you do not require as many paid inspectors on the job.

The development of some varieties of municipal engineering is absolutely dependent upon the development of public opinion and must proceed with it. The matter of street cleaning is largely a question of an improved public taste in the matter of street paving. Unless streets are well paved they cannot be well cleaned except at a prohibitive cost. To jump from one degree of cleanliness in this respect, to another, without a supporting public opinion, may be enough to wreck an administration and to set the tide of civic improvement running in the opposite direction.

The newspaper is the great educator in these matters today. But we are already using in Philadelphia moving pictures, parades and exhibitions. The possibilities of these and other means of publicity are not yet fully understood.

Again, more effort must be put into humanizing public administration. The engineer shares with those who have had the opportunity for education, the mistaken idea that the man at the top is in a position to tell the man at the bottom what is good for him. The fact that our country was founded and has been perpetuated on the contrary idea has not seemed to affect the situation very much.

Take, for instance, the movement which has led to the formation of large numbers of business men's associations and improvement associations. This affords one of the very best examples of the present vitality of American public life. Our leading men should accept them as something that has come to stay and co-operate with them in such a way as to direct their activities into profitable channels. It seems to me they afford the most promising agency through which in the first place, the thought of the public on civic questions can be crystallized, and secondly through which that thought can be given expression in definite public procedure. I have found these associations ready and anxious to hear from men who had definite knowledge on matters of public interest. It should be the attitude of any engineer who wants to play his part in the community, to affiliate with one of these organizations and to help make it an influence. You can rest assured that the man who is in public life for his own personal advancement is bending every energy to defile and degrade these institutions and to divert them from the high mission which they have it in their power to carry out, so they need our help.

Experiences in Refuse Collection and Disposal with Reference to Odors.

Public opinion greatly needs development along rational lines with regard to the question of odors which result from the collection and disposal of municipal refuse. For example, the location of a garbage disposal plant recently received much public discussion in a certain large city. A city official in urging the removal of the plant from one location to another argued that the residents near the old location had suffered their share of odors from city garbage. Later in urging the purchase of a new site, he described in glowing terms the sanitary and odorless plant to be erected there. Unfortunately, the first argument was not forgotten and the plant location was not changed. The matter should have been determined on other grounds as well. The present discussion of the odor question in refuse collection and disposal is from a paper by Mr. Samuel A. Greeley, of Winnetka, Ill., before the American Public Health Association.

The odor question should be met squarely, with due consideration of the cost of removing the plant to another location and of suppressing the odors altogether. Odors from manured lawns, tanneries, slaughter houses and other industries are tolerated in many places, because they are concerned in the livelihood of many, and cause an inconvenience from odors not commensurate with losses that would follow the loss of the industry. Projects for the location of refuse disposal plants should not be based solely on the odor question, but on the greatest benefit to the largest number of people at a fair cost.

These comments are not advanced as an argument that odors from the collection and disposal of garbage are necessary or that every reasonable means should not be taken to avoid them. The point at issue is that the presence of relatively small odors should not prevent the carrying out of a needed public improvement on an economical basis. In other words, a fair distinction should be made between odors and a "public nuisance."

In refuse disposal work, odors come chiefly from garbage and quite as often from the collection of garbage as from its disposal. It is entirely feasible to control and suppress odors from the decomposition of garbage to a safe and reasonable minimum.

It should be remembered that unclean garbage wagons will carry their odor along their route. Such odors are noticeable two to three hundred feet from the wagon. They can be eliminated by the selection of a type of wagon easily cleaned, with a good cover, and by washing the wagon at regular intervals. The European practice is to wash the collection wagons daily. The practice is growing in this country, particularly during the summer months. Washing wagons on alternate days will materially reduce odors, but will not eliminate them. With only two washings a week, odors will surely develop in garbage collection wagons in warm weather. Odors in the wagons will be reduced if the garbage is collected at least every other day in the summer months.

Odors from garbage disposal are also subject to control through proper design and operation. The cost for suppressing odors must be judged after due consideration of the character of the neighborhood. The garbage of a city of some 300,000 people is disposed of by burning with coal. The quantity of coal used amounts to about 150 lbs. and costs 25 cts. per ton of garbage. With this quantity of coal, it is inevitable that odors should at times escape from the chimney top. I have seen a thick brown smoke coming from the chimney. To completely eliminate this condition would require the use of more coal at an increased cost of disposal per ton, which might amount to 15 cts. Yet this plant is considered a success by the people in this city and is widely advertised in that light. There would seem to be no justification for the additional amount of coal and extra cost required, in this instance, to eliminate the odor.

Reduction plants have in the past come in for a large share of public criticism because of odors. The balance between the elimination of odors and the cost thereof had not been properly adjusted. This was because many reduction plants were operated by contractors who had no incentive to spend money unproductively to suppress odors. A great deal of improvement has been made during the last few years. A new reduction plant has recently been put into operation at Schenectady, N. Y. This plant is attractively housed and its location is near other buildings and indicates a growing confidence in the possibility of reducing odors from a reduction plant.

The odor question as relates to the workmen at the disposal plant, is not so important as the general cleanliness of the plant and its freedom from dust. Attention to these details will promote odorless operation. Odors, in themselves, are not injurious to health.

In small towns, where the method of disposal may be by burial or feeding to pigs, the odor question will not be so acute. Isolated locations are generally available, where the odors should not be more of a nuisance than those from a farm pig pen. Odors and nuisance will more often arise from the uncontrolled dumping of garbage in vacant lots near residential districts. I have known odors from such an uncovered dump to carry three-quarters of a mile. The remedy lies in stricter supervision by public officials rather than in large expenditures for improved methods.

The structural details for suppressing odors at refuse disposal works are not within the scope of this discussion. Good construction, however, is essential. Poor construction is an important cause of odors. Incomplete combustion may result from broken grates and cracked furnace walls. These defects are inevitable with bad foundations and poor furnace construction. In reduction works, floors and conveyors should be substantially built with ample provision for washing. Many details of construction, if properly carried out, will materially assist in the elimination of odors. All these matters should be taken up in the design. Both faulty design and construction are difficult to correct later and should be given competent attention in the first place.

Summarizing, the effect of odors from the collection and disposal of refuse is largely relative, depending a good deal upon the environment and in particular upon the industrial odors present. Nuisances should, of course, be avoided. But sentimental objection to garbage works are too often raised through a fear of odors of less intensity than a nuisance, and frequently not noticeable at all.

A rational viewpoint has due regard for the cost of suppressing odors and also emphasizes the advantage of good design and construction in the first place, followed by clean and careful operation.

Lake Superior Iron Ore Shipments 32,021,900 Tons in 1914.—Reports to The Iron Age from the 11 shipping docks on Lake Superior and Lake Michigan show that the total movement of Lake Superior ore by water in the season of 1914 was 32,021,900 tons. In this total is included an estimate for one cargo which was being loaded at Escanaba and had not left the dock on Dec. 1. The total shipments in November were 1,070,095 tons. The water shipments in 1913 were 49,070,478 tons, so that the falling off is roundly 17,050,000 tons. Shipments by ports for the season are given below, together with the comparison with the two preceding years:

IRON ORE SHIPMENTS FROM UPPER LAKE PORTS—GROSS TONS.

	1914.	1913.	1912.
Escanaba	3,664,454	5,399,444	5,234,655
Marquette	1,755,726	3,137,617	3,296,761
Ashland	3,363,419	4,338,230	4,797,101
Two Harbors	5,610,262	10,075,718	9,370,969
Superior	11,309,748	13,788,343	14,240,714
Duluth	6,318,291	12,331,126	10,495,577
Total by lake	32,021,900	49,070,478	47,435,777
Total all rail	....	876,638	785,769
Total shipments	....	49,947,116	48,221,546

It will be noticed that Superior has been far in the lead in shipments this season, the total for that port being nearly equal to the movement from both Duluth and Two Harbors. As is well known, the Steel Corporation's lease of Great Northern ore properties expires at the end of this year and the heavy Superior shipments represent the effort to get out all the ore called for by the minimum provisions of the lease. Minnesota's percentage of the total Lake Superior shipment this year, due to the special activity on the Great Northern properties, is greater than heretofore, being close to 73 as against 72 in 1913, the previous maximum.

American Concrete Pipe Association.—The next annual convention of the American Concrete Pipe Association will be held in Chicago, Feb. 15 and 16, 1915. This will be during the cement show, thus enabling all those who come to the association convention to attend the show as well.

Several addresses of unusual interest are being prepared for the program, detailed announcement of which will be made in due time.

WATER WORKS

Design and Construction of the North Side Reservoir of the Pittsburgh Water Works.

The new North Side (Cabbage Hill) reservoir of the Pittsburgh water works is located about six miles from the point section of the city. The flow line elevation is 275 ft., Pittsburgh datum, and is the same as that of the lower Highland Park reservoir which supplies that portion of the central city along both rivers, and the entire South Side.

The reservoir is of 160,000,000 gals. capacity with a water area of 17 acres and a water depth of 38 ft. It has been built by the cut and fill method, three sides being built of rolled embankment, the remaining side being entirely in cuts, surrounded by a reinforced concrete retaining wall along a public roadway. The reservoir is divided into east and west basins by a reinforced concrete dividing wall 18 ft. high, containing the inlet conduit which connects the main, or inlet and outlet gate house in the northerly embankment, with the distribution or secondary gate house in the southerly slope.

The objects of the reservoir are: (1) To replace the old, small and unsafe Troy Hill reservoir in the supply system of the North Side (formerly Allegheny); and (2) to act as an equalizer in conjunction with Highland No. 2 Reservoir. The main distribution feeder mains from both reservoirs are to be cross-connected by a 48-in. steel main under the Allegheny River. The design and construction of the reservoir are here described from information taken from a paper by E. E. Lanpher and J. S. Cole before the Engineers' Society of Western Pennsylvania. The full paper and discussion are in the October, 1914, Proceedings.

In 1896 the former city of Allegheny laid a 60-in. steel water main from its pumping station on the Allegheny River, a distance of about 8.6 miles, to the Troy Hill reservoir. At Etna this main deviated north from the river over a divide and thence back to the river bank near Millvale.

The new North Side reservoir is constructed on this divide, so that the toe of the northerly embankment is bounded by the water main. Allegheny had intended to construct such a reservoir about one-third mile north, and about 100 ft. higher than the present reservoir, and installed two Y's and valves on this main about one-half mile apart and looking toward the higher elevation. These Y's and valves were of great value in the connecting of the new reservoir to the supply main, without interfering with the water supply, inasmuch as the 60-in. main was the sole source of water supply of the North Side.

The reservoir is constructed from designs and specifications prepared in the Division of Engineering and Construction of the Bureau of Water, and contract was awarded upon the unit price basis, July 6, 1912, to the John F. Casey Co. The contract price was $681,976.

EXCAVATION AND EMBANKMENTS.

Work of clearing the site was started July 20, 1912. Preparatory to embankment work, the entire site of 24 acres was cleared of all vegetable growth and the top soil removed to storage piles. Good foundation for embankment was found in most places at a depth of about 18 ins. All embankment material was obtained from excavation in the basin and consisted mainly of first class clay of several shades, and of easily broken stone of shale formation, although in some cases low grade dynamite was used to facilitate digging. Most of this shale disintegrated rapidly when exposed to the atmosphere. A mixture of the clay and shale spread in layers of 6 ins. or less, and rolled with grooved rollers of two tons per foot produced a very hard, impervious embankment, but it was necessary to pass the roller over each layer at least six times to thoroughly compact the soil, and to further break the small stones. (There are no stones in the embankment larger than 4 ins. in diameter.)

Three steam shovels with 3, 1½, and ½ cu. yd. dippers respectively and a McMyler crane were used in excavating, the larger shovels working efficiently in the shale. Four contractor's engines and 50 dump cars, of 4 cu. yds. capacity, were used in conveying the material to the embankments. An engine operated spreader, two road graders and four 16-ton rollers were used in spreading and rolling the embankments. The maximum yardage excavated and rolled in embankment in one day of 11½ hours, was 3,500 cu. yds.

The embankments have a slope on the outside of 2 to 1 for a height of about 20 ft., where there is a 5-ft. berm. Above this berm the slope is 1¾ to 1 for a height of 20 ft. On the inside, the slope is uniform at 2 to 1 from the bottom to the top, the height being about 42 ft. above the floor of the basins. The outside of the embankment was scored and covered with 6 ins. (measured vertically) of top soil, which is seeded and sodded. This top soil was dumped from the top of the embankment, graded and rolled, a horse operated roller being used.

The top of the embankment is 15 ft. wide, while the maximum width of the foot of the embankment is 185 ft.

STEEL PIPE AND CONNECTIONS.

In order to maintain the supply through the single 60-in. main and make the difficult connections to the main gate house, about one-half mile of 60-in. riveted steel main of ⅜-in. plate was laid parallel to, and connected to the Y's left in the old main. From these two mains two 60-in. steel inlet pipes and two 60-in. steel outlet pipes were installed to the main gate house in the northerly embankment. Both 60-in. mains are equipped with Venturi meters (30-in. diameter of throat) consisting of cast iron up-stream ends, bronze throat pieces and reinforced concrete tail pieces. These tail pieces were built in place and at ten days were subjected to a 90-lb. pressure test, showing no signs of leakage.

The 70-ton traveling crane was used in connection with all of the steel pipe work, no tripods being used. Even the excavation was done by this crane with an orange peel bucket. A 300-ft. length of 60-in. pipe was laid in one day, the major portion of it being riveted. At one time it became necessary to shut off the water in the old main, cut out the rivets of three joints, remove 125 ft. of the pipe, replace a portion of this pipe with one section containing a bulkhead and rivet this section to the pipe in place; all of which was done in four hours.

Riveted steel Y's and Tees of ½-in. plate were used in place of the usual cast iron specials.

MAIN AND SECONDARY GATE HOUSES.

The main gate house foundation is a reinforced concrete structure, 42 x 38 ft. in plan, extending from bed rock to the top of the northerly embankment. Water enters through the two inlet pipes at elevation 242.5, and rises in the inlet wells and flows over weirs at elevation 271 into two inlet chambers controlled by stop planks, thence through the main inlet chamber again controlled by stop planks to a 7-ft. reinforced concrete conduit through the embankment and directly across the reservoir to the secondary gate house in the southerly slope. The secondary gate house is a well 24 ft. 6 ins. x 11 ft. 3 ins. in plan, into which the water enters at reservoir floor elevation, rises to elevation 263.90, where it is passed through 5-ft. circular sluice gates to distributing conduits in both basins. The water again re-enters the main house through two 5-ft. reinforced concrete conduits, passing directly through the gate house to the two outlet pipes. Both outlet channels through the gate house are controlled by 6-ft. x 3½-ft. sluice gates in addition to stop planks on both sides of each sluice gate.

The main gate house is so arranged that water may be passed from the inlet pipes to the inlet chambers, to the outlet chambers and outlet pipes, without entering the reservoir. Water may be drained from either basin to the main drain through 3-ft. x 2½-ft. sluice gates in the outlet chambers.

The main gate house was poured in two horizontal sections and five vertical sections, and contains approximately 800 cu. yds. of concrete mixed in a 1-cu. yd. batch mixer, electrically operated, all material being handled by the traveling crane.

The superstructures of these gate houses are not included in the present contract.

DRAINAGE SYSTEM.

In the main gate house a waste weir at elevation 275 is provided against overflowing, the water being carried to the main drain which is a 36-in. circular conduit approximately 2,300 ft. long, running parallel to the main pipe lines to a small stream discharging into the Allegheny River. Into this drain also discharges surface water from a 3 x 2-ft. egg shaped concrete drain, extending along the roadways on the southerly and easterly sides and a 12-in. and 18-in. tile drain extending along the westerly and northerly sides of the reservoir. Along the southerly, or hillside, portion of the reservoir, between the Friday avenue wall and the roadway, a ground water cut-off drain was installed consisting of a 6-in. open joint tile pipe in a 30-in. trench, the trench being filled with gravel to the surface. This tile pipe is at elevation approximately 255, or about 18 ft. above the floor of the basins, and failed to cut off all ground water as was shown when excavation proceeded to grade. This condition caused the construction of shallow diagonal drains under the floor system along the easterly and southerly slopes as nearly perpendicular as possible to the direction of ground water flow as determined by observations from a system of test wells. These diagonal or herring-bone drains carry the water to main collector drains along the toe of the southerly slope, and thence to two 8-in. main drains under the dividing wall and through the embankment to the waste drain in the gate house. All of the collecting drains consist of 4-in. and 6-in. open joint tile surrounded by gravel, all covered by a 12-in. concrete cap which forms a foundation for the floors. The 8-in. drains are surrounded by concrete to prevent upward pressure on the base of the dividing wall.

Steel forms were used for the 36-in. main drain and part of the egg drain, and both drains were poured in two sections, except where the main drain was reinforced near the gate house. In the deep trench of the cut-off drain an A-frame movable crane with hoisting drums was used to remove excavated materials. Concrete in drains was mainly hand mixed. These drains were constructed mainly in cold weather, making it necessary to heat the sand and gravel, and to maintain fires in the trenches.

FRIDAY AVENUE RETAINING WALL.

The reinforced concrete retaining wall, extending for 875 ft. along the southerly side of the reservoir and holding in place a public roadway, is built at the top of the 2 to 1 slope and varies in height from 2 to 21¾ ft. The heel of the wall is tied into a 2x3-ft. egg shaped drain for stability. The vertical section of this wall is 18 ins. thick. The foundation extends 2 ft. in front and in the higher sections 9 ft. back of the face of the wall.

The vertical reinforcing of the higher sections of the wall consists of ⅝-in. rods on 16-in. centers from the top to the base, ⅝-in. rods on 8-in. centers from a point 10 ft. below the top of the base, ⅝-in. rods on 4-in. centers from a point 12 ft. below the top to the base, and in addition, ¾-in. rods on 10-in. centers through the base and the top of the egg drain, and ⅝-in. rods on 12-in. centers extending from the base both sides of the egg drain, with additional short rods at construction joints.

The base of this wall was constructed in 8-ft. sections because of interference of the bracing that held the roadway in place, while the wall proper was built in 48-ft. sections, or from one galvanized plate expansion joint to another.

After this wall had been practically completed and the slope trimmed, a slippery clay was exposed, showing that a general movement of the wall would probably follow the back-filling. A large toe pier or buttress was therefore constructed below and in front of the base of the retaining wall and extending to the rock, increasing the stability of the wall against sliding from 0.98 to 1.50 as computed from Baker's formula with a coefficient of 0.25 for the sliding of clay on clay. By working the concrete dry a close joint was made between the toe pier and the base of the wall, no sulphur, lead or wedges being used.

Concrete for the wall and toe pier was mixed along Friday avenue in a ½-cu. yd. batch mixer; the A-frame traveling crane being used in the handling of the forms and in the backfilling.

DIVIDING WALL AND CONDUIT.

The original design of the reservoir provided for an earth dividing embankment surmounted by a 7-ft. reinforced concrete inlet conduit extending between the two gate houses. Inability to obtain sufficient clay to mix with the shale for outside embankment purposes caused the substitution of a combined dividing wall and conduit for the dividing embankment and circular conduit. A view of the dividing wall and conduit is shown in Fig. 1.

The conduit wall is a triangular conduit of an area equivalent to a 7-ft. circle, supported on two legs or cut-off walls extending from 5 to 9 ft. below the floor to bed rock, all surmounted by an 18-in. wall about 5 ft. high, making the height of the conduit wall 16 ft. above one basin floor and 18 ft. above the other basin floor. The sides of the triangular conduit are 20 ins. thick, reinforced with ⅞-in. diamond bars on 9-in. centers, extending from the legs to the surmounting walls and ½-in. diamond bars on 12-in. centers at both surfaces. The base of the triangle is designed as a beam with the same reinforcement as the sides. The surmounting wall is designed as a cantilever. Copper expansion joints are used in the legs and floors.

This conduit wall was built in 58½-ft. sections; the legs, the conduit floor and the sides with the surmounting wall being built at separate pourings. It being practically impossible to work the concrete in the conduit forms, two air hammers were kept constantly but gently hammering the forms during the pouring, resulting in a thoroughly compacted concrete of smooth finish. All forms and material for the dividing wall and conduit were handled by the cableway.

That portion of the inlet conduit from the main gate house to the conduit wall is a 7-ft. circular, reinforced, concrete conduit for a distance of 40 ft. to a vertical well, which forms the junction of the circular section and the triangular section of the inlet conduit.

The reinforcement of the circular conduit consisted of ½-in. rods on 6-in. centers running longitudinally, and ½-in. rods on 4-in. centers running transversely, except in the outside layer of the reinforcement in the bottom where ½-in. rods on 24-in. centers running transversely, and ½-in. rods on 4-in. centers running longitudinally, were used.

The concrete for the circular conduit was mixed in a ½-cu. yd. batch mixer, while the concrete for the vertical well was mixed in the central mixing plant, and placed by means of the cableway.

CENTRAL CONCRETE MIXING PLANT AND ITS SUPPLY AND DISTRIBUTION PLANT.

All the sand and gravel used in the construction of the reservoir was dug from the Allegheny River in the vicinity of Hoboken, from which point it was transported in barges to Millvade, where the contractor installed a hoisting plant. The material was taken from the barge in a clam shell bucket and dumped into an elevated storage bin containing compartments for coarse gravel, fine gravel and sand. From this bin it was taken to the reservoir, a distance of 1¾ miles by a group of six 5-ton auto trucks, each truck making from 18 to 20 trips per day of 11 hours. The cement was received at a siding adjacent to the Millvale storage bin, and transported to the reservoir in a 5-ton flat-bed truck. During the summer of 1914 these trucks were operated double shift, or 22 hours per day, in order to maintain a supply of materials at the reservoir site. From the fleet of seven trucks it was found necessary to hold one truck each day in the shop for overhauling.

At the reservoir the supply bins and the cement house were built on the southerly slope between the foundation of the secondary gate house and the Friday Ave. wall. These bins held approximately 400 cu. yds. of concrete gravel, 200 cu. yds. of sand and 200 cu. yds. of coarse gravel. Storage was provided for two cars of cement in the cement house and one car on the platform. Sand and gravel were dumped from the trucks directly into the bins. The cement was dumped on the Friday Ave. platform and hand handled to the chute leading to the cement house.

An electrically operated 1½ cu. yd. batch mixer was installed on the secondary gate house foundation directly in front of the storage bins. The sand and gravel from the bins were controlled by shear gates directly above the mixer hopper, while the cement was placed in a vertical chute directly over the hopper, and controlled by a slide gate. The hopper was standardized for 1 cu. yd. of concrete. During the busy season 1 cu. yd. of concrete could be turned out of the mixer every minute, although the average time of mixing was about 1½ minutes; this, of course, was for wet concrete used in walls and all flat work. The dry concrete used on slopes required about 2¼ minutes for mixing each batch.

The mixer discharged directly into shear gate buckets on flat cars, or to cradle cars. Three contractors' locomotives were required, during the time of the laying of the floor and slope blocks, to transport the concrete from the mixer to the crane. A view of the central mixing plant is shown in Fig. 2.

Fig. 1. View of Combined Dividing Wall and Inlet Conduit of North Side Reservoir, of Pittsburgh, Under Construction.

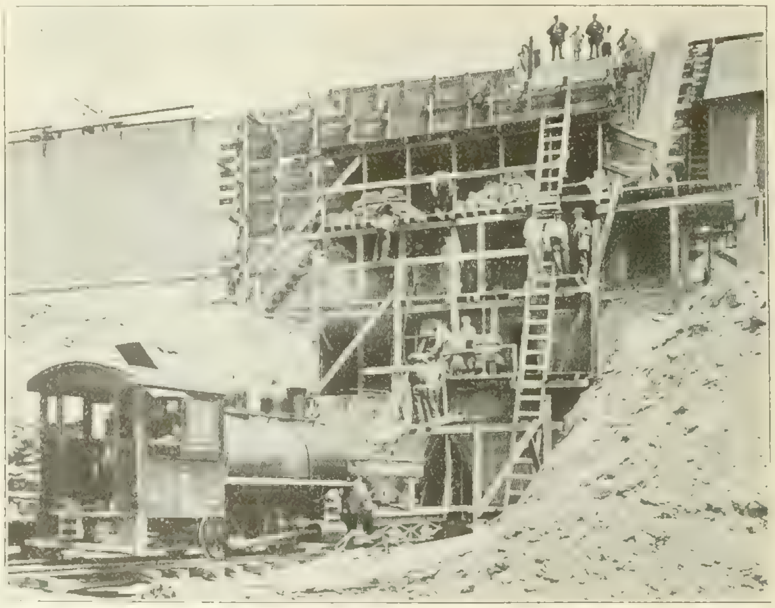


Fig. 2. View of Central Concrete Mixing Plant, North Side Reservoir Construction, Pittsburgh, Pa.

A stationary cableway was used in the construction of the dividing wall and conduit, and the well in the northerly embankment, also for general transportation purposes from Friday Ave., including the transporting of the steel reinforcement and concrete forms. The head tower is 50 ft. high, located on the southerly side of Friday Ave., and the tail tower is 45 ft. high, located on the gate house in the northerly embankment. The distance between towers is approximately 850 ft.

RESERVOIR LINING.

The lining of the reservoir floor and of the slopes up to the revetment consists of two 4-in. layers of concrete, the joints overlapping 8 ins. instead of the usual method of overlapping from center to center of blocks. The lining of the slopes above the revetment drain consists of a 4-in. layer of concrete covered to a depth of 12 ins. with large gravel, upon which rests a 6-in. layer of reinforced concrete, the waterproofing and a 4-in. surface layer of concrete. The floor blocks are 10 ft. square, while the blocks on the slopes vary from 8 × 10 ft. to 16 × 38 ft., the larger blocks being in the 6-in. reinforced layer.

Waterproofing of the slopes consists of

four 1/16-in. layers of asphalt and three layers of 8-oz. saturated burlap laid alternately, between the layers of concrete, while in the floors three layers of asphalt and two layers of burlap in 18-in. strips were used at the floor joints only in such a manner that the waterproofing overlaps the joints of both layers by 5 ins. All joints were treated with asphalt either by mopping or by pouring the spaces left by the ¼-in. steel plates used as forms. The three-ply waterproofing is about ⅜-in. thick, while the two-ply waterproofing is about ¼-in. thick.

All concrete used below the revetment was mixed at the central mixing plant, transported in 1-cu. yd. batches in shear gate buckets on flat cars, lifted from the flat cars by the 70-ton crane and dumped into the forms. A few of the blocks above the revetment were placed in this same manner; a few such blocks were also built from concrete mixed in a ½ cu. yd. mixed on top of the embankment, dumping directly into the forms; but the major portion of these blocks were built from concrete mixed at the central plant, conveyed in yard cradle cars along the top of the embankment and dumped into the forms. The concrete for the floor blocks was of a wet mix, while the concrete for the slope blocks was much dryer.

On the slopes the usual system of pouring alternate blocks was followed. On the floors the concrete was laid in rows extending from one side of the basin to the opposite side, the longitudinal forms consisting of 4-in. × 6-in. stringers, while the transverse forms were ¼-in. × 4-in. steel plates set in the stringers and the points of the preceding row of blocks. The concrete was finished with straight edge and wooden trowels. In a day of 11 hours 488 cu. yds. of concrete were placed in floor blocks and finished.

The distributing conduits, which extend along the southerly slopes of both basins, were built from concrete mixed in the ½ cu. yd. mixer working along Friday Ave. A section of this conduit is in the form of a trapezoid, the top of which is horizontal, the bottom of which is the upper layer of the floor system on the 2 to 1 slope, and the sides of which are vertical. Circular openings are provided in the top of the conduit, to provide for circulation of water near the secondary gate house.

The asphalt was heated to 400° F. and applied with ordinary cotton mops. In three-ply waterproofing the burlap was lapped 13 ins., the width of burlap being 39 ins.

All concrete used in the construction of the reservoir was of 1:2:4 mix, except in negligible cases, such as manholes, catch basins and gate vaults.

The responsibility of the reservoir lies with Mr. Joseph G. Armstrong and Mr. Robert Swan, former and present directors of the Department of Public Works; Mr. Charles A. Finley, superintendent of the Bureau of Water; Mr. C. O. Daughaday and Mr. John M. Rice, Division Superintendent and Assistant Engineer, respectively, of the division of engineering in charge during the design and first four months of construction work; Mr. E. E. Lanpher and Mr. John S. Cole, Division Superintendent and Assistant Engineer, respectively, of the distribution division, in charge since November, 1912.

DISCUSSION.

The following interesting points were brought out in the discussion. Mr. J. M. Rice explained the advantage of lapping the two layers of concrete only 8 ins. instead of lapping them half their length, as is usually done, as follows:

We adopted this design from experience gained in the construction of the Pittsburgh Filtration Works. During the first season of construction the floor which was of two layers, each 4 ins. thick, was put in with the upper blocks breaking joints at the center of the lower layer blocks, and after the winter had passed we found that the upper blocks had broken just over the joints in the lower layer. In the endeavor to avoid this next season we brought out the construction used here, i. e., offsetting the vertical joints in the upper and lower layers from 6 to 8 ins., and this proved effective.

The explanation for the cause of the cracks offered is that the contraction in the lower blocks during the cold weather had been sufficient to break the upper blocks and by lapping the joints a small amount we did not get that effect, as the adhesion of the surface in contact was less than the strength of the upper layer of concrete, and thus sliding instead of cracking occurred.

In spite of the precaution taken to secure watertightness and the adoption of a waterproofing membrane, this reservoir compares very favorably in cost per million gallons with reservoirs of similar size, as shown in Table I, which gives some costs of large basins constructed in the United States.

One of the most interesting points discovered in the course of the construction work was in connection with the waterproofing. This consisted of three plies of saturated burlap mopped on with hot asphalt. Mr. S. L. Fuller discussed that feature as follows:

The specifications said "all surfaces receiving waterproofing shall be clean, dry and smooth." The word "dry" in relation to setting concrete was subject to the decision of the inspectors and naturally was an open point for argument, as in order to keep the work moving we desired to lay the waterproofing as soon as possible, while the inspectors held out for strictly dry surfaces. We had occasion to remove one of the top blocks and much to our surprise and gratification we found that while the layer of waterproofing which had been mopped on to the 24-hour old concrete had no adhesion to it at all, yet the top block which had been poured wet on to the asphalt could not be separated from it. We then made several experiments and we found that the best bond between the asphalt and concrete was obtained when the asphalt was mopped on as soon as the concrete had set, so that it could be walked on without injury. The theory we advance is that as soon as concrete starts to set it starts to "dust," and by applying the asphalt practically before this starts we avoid the thin layer of dust, which it is impossible to remove and which prevents the close adhesion of asphalt to concrete.

Probably the most remarkable thing about the job was the speed obtained in laying the 4-in. floor slabs. Through a carefully designed concrete plant with one Ransome 40 cu. ft. mixer, motor driven, we were able to average 375 cu. yds. per day for a period of four weeks, the maximum day's output being 489 cu. yds. All of the material for this was hauled by five, 5-ton Pierce Arrow motor trucks, which we found most economical in operation. These we worked 24 hours a day and they hauled over a distance of 1¾ miles, the last ¾ mile being all up a 4 to 6 per cent grade. The economy of the motor truck was marked. We cut down the haul by probably half over what we could have accomplished with teams, and it would have been impossible to complete the reservoir in one season had it not been for the truck proposition, because we could not have worked 150 teams in and out. As it was, with the team making six trips a day from the hoist to the job, we had to snap them all up the hill. We put on three teams and had four teams to bring up the three wagons. That was before we procured the trucks. Of course the upkeep of trucks is heavy on that hill, but there is no question that the truck proposition was the life saver of the job.

TABLE I.—COST OF SIX LARGE AMERICAN WATER WORKS RESERVOIRS.

Reservoir.	Location.	Capacity in million gals.	Cost.	Cost per million gals.
Queen Lane	Philadelphia, Pa.	383	$1,188,000	$3,100
New Roxborough	Philadelphia, Pa.	147	524,000	3,600
Settling Basins	Cincinnati, Ohio	330	1,276,000	3,900
Service Reservoir	Minneapolis, Minn.	93	442,000	4,750
Prospect	Rochester, N. Y.	110	554,000	5,000
Northside	Pittsburgh, Pa.	150	676,000	4,100

Time Studies in Connection With the Cleaning of Filter Sand at Philadelphia.

Efficiency in municipal government will only come about as the work in the various departments is put on a basis which gives each man, from the common laborer up to the skilled artisan and clerk, a well-defined task to do in a given time, with a definite reward for its accomplishment. Under the ordinary methods of handling city work it is cheaper where there is fair competition to let work by contract than to handle it by day labor. With an effective system that eliminates not merely favoritism but also presents a definite incentive for each man to do a fair day's work, the day labor plan becomes more attractive from an economic standpoint. The construction and maintenance work in the department of public works is a field that offers the largest opportunities from an engineering standpoint. It includes such operations as trenching, pipe-laying, sewer construction, aqueduct construction, filter cleaning, street cleaning, road building, grading, concrete work and building construction. Under ordinary municipal management it is out of the question, as above indicated, to handle work of this kind economically. However, practically every one of the classes of work mentioned has been handled at large reduction of cost by means of systematic planning of the work in advance and layout of tasks or establishment of piece rates.

The present article describes one accomplishment of Philadelphia along the line of improved methods. The particular operation studied is the cleaning of filter sand. Our information is from a paper before the American Society of Mechanical Engineers by Mr. Sanford E. Thompson.

Philadelphia has five large filtration plants consisting of covered reservoirs operated by slow sand filtration. The water pumped into the reservoir from the Schuylkill and the Delaware Rivers, after passing through the pre-filters, percolates through about 4 ft. of sand and gravel and is thus purified. The impurities are caught largely in the upper few inches of sand, so that if this upper portion is washed the filtration area is practically renewed. Several methods of cleaning filter sands are in use, all of them involving considerable manual labor. Further details of the methods followed in the case under observation are referred to below.

RESULTS.

The object of the plan has been to lay out the work of each gang of men so as to increase the effectiveness of the plant and provide a definite task to be accomplished in a day. The results of the plan which is being put into operation are as follows:

Rotation of cleaning the filters is planned in advance by well-defined rule.

A definite area of sand to clean is assigned to each gang, this area depending upon the depth of cleaning necessary.

This setting of tasks has increased output of each gang 15 per cent and this should be further increased to at least 25 per cent.

Accurate records are kept, showing the time consumed by each gang.

Cost accounts, as well as pay-roll, are made up from the time tickets furnished to the men.

Gang leaders are required to pay closer attention to their duties.

Improved apparatus and machinery are under consideration.

Methods of determining depth of sand to clean are being standardized.

OBSTACLES.

The greatest obstacle encountered has been the city ordinance fixing the rate of pay of unskilled laborers on a level wage per day regardless of the quality of the workman or the amount of work he is able to accomplish. While in city government strict regulation is necessary, a plan such as is followed in Chicago, where the employes in each department are definitely graded, with different wages for each grade, provides a means for rewarding a man according to his ability and giving a city good value for money expended. The Philadelphia ordinances prevent the payment of a bonus and thus make it difficult to encourage the men to accomplish the tasks assigned them.

Another hindrance to efficient management that is found ordinarily in city laws is the fact that civil service regulation, while preventing the discharge of a man for political reasons, at the same time limits the power of discharge for inefficiency. The municipality thus pays inferior men not only the same rate as first-class men, but prevents the good man who is out of a job which he really deserves from obtaining a situation which the loafer holds. Although the fear of discharge or reduction in wage is a crude and unsatisfactory way of obtaining a fair day's work, its elimination with no means of providing a corresponding substitute increases the difficulties in the way of giving a city fair return for its money.

In the case under consideration, for example, the handling of the work would have been much simplified and made substantially automatic if it had been possible to provide, for a man well fitted to his work, a reward for accomplishing a good day's task. As this was impossible, and as it is unfair in any case to expect a man to do an exceptional day's work without an exceptional day's pay, the tasks laid out were lower than would be set under a proper system of task and bonus, and their maintenance was more difficult to enforce.

At the beginning, the general attitude of the men and the foremen was antagonistic, as is almost always the case where new methods are being introduced. This is gradually overcome as the results become evident.

METHOD OF CLEANING.

In the filtration plant first handled by the new method there are 65 filters, employing about 128 men for cleaning. Each filter is about 140 ft. wide by 250 ft. long, and is built with groined arch bottom and roof, having columns about 16 ft. on centers.

The Nichols method of washing is used in this plant. In this method the dirty sand from the surface of the bed to a depth specified is shoveled to an ejector, furnishing water under about 85 to 100 lbs. pressure, which forces it through a large hose into the separator, which is a cylindrical iron tank provided with a concentric baffle about 6 or 8 ins. from the outside shell. The water and sand swirl around this, the clean sand settling in the conical bottom and passing out through a 2-in. hose below. The dirty water passes under the baffle and out of the top of the tank, whence it passes out of the bed through a hose and pipe to sewer.

From the separator the sand is returned by the hose to the bed, where it is properly distributed and leveled. Sometimes, according to conditions, the dirty sand is shoveled direct to the hopper of the ejector, and in other cases is scraped and piled from the first and one-half the third bay into the second line of bays; from the other half of the third and one-half the fifth line of bays into the fourth line of bays; and so on, to include the ninth bay. This scraping and piling is done usually as an independent operation by old men unfit for harder work.

Four washing gangs are required for each filter bed, the outside gangs having 2½ bays each and the inside gangs having 2 bays to clean. In each gang there are 3 shovelers to a hopper, 2 men shoveling at a time while one rests. Each man shovels 40 minutes and then rests 20 minutes. The fourth man takes care of the hose from the separator, distributing the clean sand to the bed. A fifth man, recently introduced, working with 2 gangs, spades up the hard sand that has been uncovered before replacing the washed sand.

METHOD OF ATTACK.

The method of attacking any problem in order to place it on a scientific basis varies with the character of the work. In certain cases, such as intricate factory operations, it is necessary, before any tasks are set or even before time studies are made, to establish a complete system of routing the materials and the employes. In other cases the first necessity is to establish standards, making minute investigation of the processes. In still other cases time studies can be made at the start.

In the sand cleaning proposition all of these methods were carried on in a measure simultaneously. Studies were made of the men and the methods employed to see where the manner of handling the work could be improved. Time studies were made to determine the unit times for each individual operation, so that the tasks could be figured accurately in advance. From records already on file, giving the approximate time for cleaning, it was possible to begin the organization of the routing system.

OPERATION ORDER

Returned | Charge to
Issued | DFT

Operation or Symbol | Filter Station
DFT

W... B... Finished | H... D...

Rate	Am't Earned	Total Hours	Regular Hours	O. Time Hours

(Do not fill out here for gang work)

Man's Name

If Job is not finished scratch out this ☞ F.
If Job is finished scratch out this ☞ N. F.

Foreman

Route Sheet | Pay Roll | Cost Div.

Superintendent

Fig. 1. Time Ticket.

Besides the studies in this particular plant, time studies were also made on various operations in the other plants and methods in various places compared. The general laws of the variation of the filters as regards loss of head, rate, and turbidity, were sought by means of graphical tabulations and curves. Unit costs of labor in the past were investigated.

ROUTING.

As soon as the preliminary studies were far enough advanced a bulletin board of the type used in the Taylor system was set up in the office at the plant, provided with suitable hooks for the tickets which designated the work of each man. One of the lines of hooks held tickets indicating "Work to be done NOT READY"; a second line above it, "Work to be done READY"; and the third line, above this "WORK IN PROGRESS." On these tickets, which are 4¼ by 4¼ ins., there is space for all the information required. The ticket is shown in Fig. 1.

The order in which the filters were to be cleaned was designated by the superintendent. The time each task is begun is recorded on the ticket and also the time of completion. From these tickets records are kept at the plant in terms of labor hours on every class of operation for each individual bed, and the payroll is made up from them. From the payroll division the tickets go to the cost division, where they are filed under the various charges, so that a monthly cost of operation sheet can be made.

A definite system of cross-checking is provided. Previously the time was charged against the various cost items by the gang leader on a daily report sheet and the pay roll and the cost sheet made out from these were sent to the cost division. The results were inaccurate, as the charging of the time was kept by the gang leader.

The initial stages of the introduction of the new methods were under the personal direction of Mr. Thompson's associate, Mr. Lichtner. The system of routing was handled by Mr. Albert Tolson, and the studies and tabulations made by Mr. Lyle L. Jenne. As soon as the routine was established, it was taken over by Mr. Siddons, the superintendent.

Eventually, the central planning office will be established at City Hall, with auxiliary bulletin boards at the various plants operated by telephone from the central office. This plan was started at the beginning, but it was found that further standardization of the method of determining the order in which the filters are to be cleaned was necessary before the centralized planning office would work satisfactorily. New record forms have been developed and the system is working smoothly.

UNIT TIMES.

Time studies were made by the aid of the stop watch on the labor operations in the beds, such as shoveling dirty sand to hopper, cleaning up around hopper, moving hopper, moving separator, and moving track. These times for individual operations were then converted for direct use into the time per cubic yard for 1 in. of depth.

Studies were also made on the rate of delivery of sand from separator and the effect of opening and closing the separator on the rate of shoveling. Different methods of handling the ejectors were also included in the investigation.

The object of the time studies was to find the time of each individual operation, so that unnecessary operations could be eliminated and the unit times of the necessary operations could be combined to apply to all conditions. Over-all time records are of no use whatever, because, for example, with each change in depth of shoveling, the number of moves of the hopper and of the separator vary.

The unit times for the individual operations were determined by the taking of a large number of time studies in such a way as to eliminate all unnecessary delays, but with a sufficient allowance for resting and delays which were unavoidable. The unit times obtained are given in Table 1.

TABLE I—UNIT TIMES FOR VARIOUS OPERATIONS IN CLEANING FILTER SAND.

Operation.	Unit time per operation, min.	Time per cu. yd. per 1-in. depth, min.
Moving hopper	0.20	0.34
Moving separator	0.50	0.45
Moving hopper hose	0.25	0.11
Moving track	0.83	0.44
Waiting for hopper to empty	0.42	0.38
Moving pressure hose	1.80	0.36
Additional necessary rest	..	0.12
Shoveling to hopper	....	6.32

The time given in each case is that for the gang, since it was necessary on this work to set a task for the entire gang instead of starting the individual men, as it is always best to do when possible. The time of shoveling into the hopper is in each case based on the rate of output that the ejectors will take care of. It was found that one man, instead of two, could very nearly produce the required output, but this would have lengthened the time of cleaning so as to be inadvisable. For example, with one man shoveling, the shoveling time per cubic yard is 8.8 minutes with a 1-in. depth, and 6.75 minutes per cubic yard when the depth is 18 ins. These studies indicate therefore that further change is necessary in the method of operation so as to increase the output of the ejector and separator in order to obtain the full value of the labor of the gang.

In addition to the time studies on the work of the laborers in the filters, time studies were also made on the clerical work, such as making out tickets, operating bulletin board, extending time on tickets, entering time on various records, and checking up the payroll in order to distribute the work equally among the force employed to carry it on.

SETTING TASKS.

Having determined the unit times and established the system of routing and giving out of tickets, the area of surface that should be shoveled by each gang was figured and the point to which they were supposed to go in a day's work was marked with a flag. In order to fix this, it is necessary to determine in advance by test holes the depth which should be cleaned, figuring the area from the volume at the required depth. Curves have been plotted, giving areas or rather distances to clean for the outside and inside gangs for various depths. These distances are converted into pier locations, so many feet in front or back of pier number so and so. The actual point reached each day is reported at the office and the mark for the following day calculated therefrom.

On the first two days, after everything was ready, no instructions were given the gang leader or the men as to how much they were expected to do. The total area shoveled by each gang, however, was noted, and compared with the area they should have accomplished. Every gang shoveled less than the figured area, the amount running from 10½ per cent less to 31½ per cent less. After this second day's work we concentrated on E-1 gang, since it is always necessary in order to avoid friction to work with a single man or a single gang, and laid out in advance the amount this gang should accomplish in a day by setting a flag at the point which marked the end of the day's work. As a result, they readily accomplished the task and reached the mark. The task setting was then extended to other gangs.

One rather interesting point came up in connection with the handling of the work at first. The men in the outside bays had to shovel about 7 per cent more sand than those in the inside bays because the areas were wider; nevertheless, all gangs had been accustomed to keep abreast, the men who had the narrower width to handle slowing up to accommodate their speed to the outside men. When the men began working by the task, the operation was somewhat similar, except in the other direction, until the men realized the difference. The inside men, because of the narrower width, were given the longer area to cover and gaged their speed to accommplish their task. The outside men, although shoveling a greater width kept abreast with them without special trouble, thus exceeding their task.

ACCOMPLISHMENTS.

The rates were set on the basis of a fair day's work which should be accomplished with a first-class foreman and with no incentive to the laborers. Because of this absence of incentive the work actually done averages considerably less than the actual tasks.

To compare the amount of work accomplished before and after setting tasks the records were averaged of 27 cleanings taken at random from a period of 1½ years previous to the introduction of the new methods. These showed an average rate of 6.3 cu. yds. shoveled per day per gang. An average of 55 cleanings after task work was started gave 7.2 cu. yds. per day, an increase of nearly 15 per cent. This increase, however, was less than half of what it should have been, the figured rate being 8.4 cu. yds. per day. Although the 15 per cent increase was well worth accomplishing, our tests showed positively that the larger increase of over 30 per cent should readily be accomplished with first-class supervision. One plan considered as a partial incentive is a record card for each man showing his output and thus indicating his relative rank as a workman. The rank of a man would influence the laying off if work is slack, or, on the other hand, if a man is required for a higher position, this ranking would be taken into account. If it had been possible to pay an actual money bonus, the task would have been set still higher and the output would have been increased about 50 per cent.

As the work on the filter management was getting under way, circumstances called the men in charge to other locations in the city temporarily. Going back to the job and making further studies, it was found that time had been lost: (*a*) by not throttling down the separator so as to make it run continuously and thus deliver its full output; (*b*) by unnecessary throttling of the hopper and cleaning up ahead before moving hopper to next portion of pile; (*c*) by not keeping spray open to fullest capacity. It was noticed whenever the gang was watched closely that they accomplished their task without any difficulty.

APPARATUS.

The studies, as is always the case where thorough investigations are made, indicated a number of changes advisable in the apparatus and methods of handling it. It was found that the line of piping for the water used under pressure were poorly arranged, so as to require in certain cases long lengths of hose and a consequent deduction in pressure which largely increased labor costs. In other cases certain pipe lines had to be moved from bed to bed during the operation of cleaning. The studies have shown that a mechanical washing device probably can be devised which will greatly reduce the cost of cleaning.

Even with the present apparatus the method of handling the separators and ejectors can be considerably improved and the cost of this quickly made up by labor saved.

The design of the hoppers and separators, as already stated, could be improved so that they would handle just the right amount of material that a gang can readily shovel. The present output is limited by the design of the hopper and ejector.

These various matters are under consideration and improvements are being made from time to time.

CONCLUSIONS.

It is evident from the results obtained here and also from similar experience in other work, that output on city work can be appreciably increased simply by careful planning of the work in advance, systematizing its handling, and following it up in routine fashion. To obtain the full value of organization, however, it is necessary to provide some definite incentive to the men to accomplish their tasks. This makes the handling of the work automatic and eliminates friction, and, especially, gives the men the extra money which they actually deserve whenever the tasks are set high enough to require a good day's work.

BRIDGES

From the Soil Up—A New Method of Designing.

Contributed by Arvid Reuterdahl, Consulting Engineer, Kansas City, Mo.

Modern engineering design involves a knowledge of two factors—constants pertaining to the materials and to the principles of mechanics. These "weapons" of design are employed by the designing engineer in investigating the engineering efficacy of an assumed quantity of material displayed in a tentative shape or form. At the end of the investigation the engineer becomes aware of the correctness or incorrectness of his assumptions in regard to quantity and of his guess as to the form of the proposed structure. Experience and knowledge concerning quantity and form of other similar structures aids the engineer in these assumptions and guesses.

This procedure is universal in engineering design. First, the designer guesses at mass and form; then he tests the engineering value of the guess by an application of the constants of the material and the principles of mechanics to the tentative or trial design. The writer is unaware of any other procedure in the entire literature of engineering.

It is with a thorough knowledge of the difficulties involved in any radical departure from the above universally adopted procedure in engineering design that the writer ventures to suggest the splendid possibilities open to the designer who has the courage to leave the trodden path of universal practice and sally forth into regions unexplored but replete with gratifying possibilities.

The writer has selected the design of a retaining wall in order to exhibit the steps and principles involved in his proposed method of design. This single illustration should prove sufficient to demonstrate the possibilities of the method.

DESIGN OF A REINFORCED CONCRETE RETAINING WALL.

Order of Procedure.—According to the writer's method, the following procedure should be followed:

(1) Adopt a safe maximum bearing power for the soil at the site.

(a) This value is determined from actual soil tests of the ultimate bearing power by dividing the ultimate value by some arbitrarily adopted safety constant.

(2) Review the given field data pertaining to the required structure.

(a) Ground profiles, required grade lines, earth slope, etc.

(b) Determine height of wall from top to bottom of foundation from the given field data.

(3) Review the data from engineering experience applicable to the proposed structure.

(a) Adopt a ratio between height of wall and length of base (a section being considered) consistent with good engineering practice.

(b) Adopt a ratio between length of toe and length of base in accord with engineering practice.

(4) Locate the position of the final force resultant in respect to the foundation center.

This is equivalent to a predetermination of the amount of the final force eccentricity which the designer intends that the structure shall have. In other words if the designer desires that the final force resultant shall fall within the middle third of the base he locates it accordingly and then proceeds to force the subsequent design to meet this requirement.

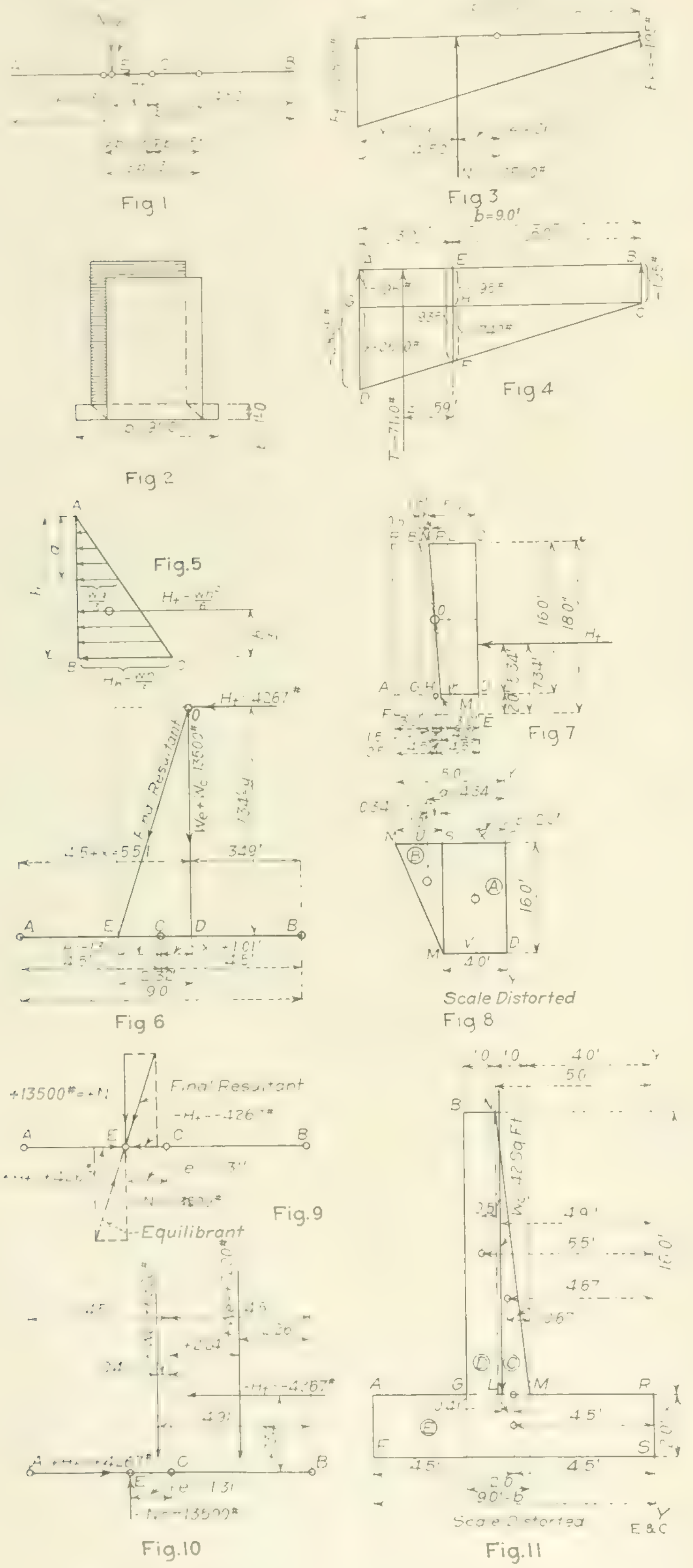
(According to prevalent methods, the final resultant, in magnitude and location, is discovered last and after an investigation of a tentative design. If the designer is experienced or a good guesser he is ready for congratulations if the final resultant appears in such a relative position to the foundation center that excessive soil pressures will not be developed.)

(5) Calculate the value of the normal (vertical) component of the final force resultant.

(6) Calculate the required base thickness

... the toe consistent with the value of the normal component and its eccentricity.

(7) Calculate the magnitude and determine the location of the horizontal component of the earth pressure on the stem of the wall.



Figs. 1 to 11. Diagrams Used to Explain New Method of Designing as Applied to the Design of a Retaining Wall.

(8) Calculate the required thickness of stem at the plane of junction with the base.

This can readily be done since the magnitude and location of the horizontal component of the earth pressure is known.

(9) Find the location of the resultant of all the vertical force components.

In this case the vertical forces are the earth load, W_e, and the wall load, W_c.

The point of intersection of the resultant of the horizontal components with the resultant of the vertical components is one point in the line of direction of the final resultant. Point "E" (see Fig. 1), 1.31 ft. to the left of the base center "C," has been arbitrarily located. These two points determine the location and direction of the final resultant.

(10) Calculate the factor of stability against overturning for the system as now developed.

(11) Adopt an earth load area and calculate its value in pounds.

(12) Find the location and direction of a single force equivalent, in its effect, to the earth load area.

(13) Find the magnitude and location of the resultant of the load due to the weight of the wall itself.

(a) Wall load = normal component — earth load.

(b) The *magnitudes* of all the load factors are known at this stage of the work. The *locations* of all except one of the force factors (the unknown being the location of the wall load) are also known.

(c) Calculate the location of the wall load by taking moments about some convenient point.

(14) Calculate the wall load area.

This is found by dividing the wall load by the weight of one unit of wall volume.

(15) Determine the actual working dimensions of the stem consistent with the eccentricity of the final resultant and the magnitude of its vertical component.

(16) Calculate the actual thickness of the base required to fulfill the conditions of location and magnitude of the wall load.

APPLICATION OF SUGGESTED PROCEDURE TO AN ACTUAL DESIGN.

The above outlined method will now be applied to the actual design of a reinforced concrete retaining wall.

(1) *Safe Maximum Bearing Power of the Soil.*—Soil tests at the site developed the fact that displacement of the adjoining soil occurred when the load per square foot exceeded 11,220 lbs. This value was then taken as the ultimate bearing power of the soil. Using a safety factor of 4, the safe maximum bearing power of the soil is 11,220 ÷ 4 = 2,805 lbs. per square foot.

(2) *Field Data.*—From the field data it was ascertained that the wall height h must be 18 ft. from bottom of foundation or base to the top of the wall. Furthermore the location precluded a surcharge.

(3) *Applicable Engineering Data.*—(a) The ratio between the length of base b to the wall height h used in good construction varies from 0.4 to 0.5.

Using the maximum of these ratios, $b \div h = 0.5$, and since $h = 18$ ft., $b = 9$ ft.

(b) A prevalent and safe ratio between the length of the toe t and the base length b is 1:3. Hence with $b = 9$ ft., $t = 3$ ft.

(4) *Location of Final Force Resultant.*—It is generally a desideratum in the design of walls, piers, abutments and dams that the final force resultant shall lie within the middle third section. Since $b = 9$ ft. the middle third zone is 3 ft. in length. Hence the distance from the mid-point of the base to either extremity of the middle third zone is 1/6 of 9 ft. = 1.5 ft.

If the final resultant falls within one of the extreme thirds then tension is developed in the other extreme third. Consequently the designer who employs the orthodox method of designing is often chagrined when he finds, after a series of laborious calculations, that the final force resultant falls outside the middle third, for this, in most cases, necessitates a complete revision of his design.

According to the writer's method this is entirely avoided by arbitrarily, and at the start, locating the final force resultant in a position satisfactory to the designer, after which the design is constructed to conform with the requirements involved in the initial predetermination of the position of the final resultant.

Let us then arbitrarily select a suitable location of the final resultant. It is reasonable to suppose at the outset that the horizontal component of the earth pressure will tend to translate the force lines in a direction similar to its own. Hence with the toe of the wall located at the left we assume that the final resultant will be found to the left of the base center. Therefore, in Fig. 1, where AB represents the bottom of the foundation, the final resultant R is shown cutting the line AB at a point E to the left of the center C. The distance EC is generally termed the ***eccentricity*** of the force and we shall use the symbol ***"e"*** to denote its value. Furthermore, we shall adopt the following conventional system in regard to distance and forces:

Horizontal distances measured to the right of the center are	+
Horizontal distances measured to the left of the center are	—
Vertical distances measured above AB are	+
Vertical distances measured below AB are	—
Horizontal forces acting to the right are	+
Horizontal forces acting to the left are	—
Vertical forces acting downward are	+
Vertical forces acting upward are	—

In accordance with this conventional notation the distance EC or "e" is here negative.

Since $\frac{1}{6} b = 1.5$ ft., e may be any value less than 1.50 ft. Let $e = 1.31$ ft.

(5) *Calculation of Magnitude of Normal Component of Final Force Resultant.*—The forces considered are those developed by the active load masses and the reactive soil pressures contained between two parallel vertical planes perpendicular to the longitudinal axis of the stem of the wall and distant 1 ft. from each other. We therefore consider a rectangular plane area between the bottom of the foundation and the supporting soil whose dimensions are 9 ft. x 1 ft., and whose area is 9 sq. ft. This basic rectangle (Fig. 2) is contained between the two vertical cutting planes.

Let

F_t = maximum force exerted under the front edge of the toe.

F_a = average force exerted on the soil.

F_h = minimum force exerted under back edge of the heel.

e = eccentricity of the final force resultant.

b = width of the foundation.

N = normal component of the final force resultant.

Then the following relations obtain:

$$F_a = \frac{F_t}{\left\{1 + \frac{6e}{b}\right\}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$F_a = \frac{F_t}{\left\{1 - \frac{6e}{b}\right\}} \quad \ldots\ldots \quad (2)$$

$$F_h = F_a \left\{1 - \frac{6e}{b}\right\} \quad (3)$$

$$N = \left\{\frac{F_t + F_h}{2}\right\} b \quad (4)$$

$$N = F_a b \quad \ldots\ldots\ldots \quad (5)$$

Since the maximum force will be exerted at the front edge of the toe with "e" negative, and because the maximum safe bearing power of the soil is 2,805 lbs. per square foot, it follows that we must equate the maximum toe pressure to the maximum safe soil bearing power. Hence,

$F_t = -2{,}805$ lbs. per square foot.

Substituting the known values ($F_t = -2{,}805$ lbs., $e = 1.31$ ft., and $b = 9.0$ ft.) in equation (1) we have,

$$F_a = \frac{-2{,}805}{1 + \frac{6\,(1.31)}{9}} = -1{,}500 \text{ lbs. per sq. ft.}$$

Substituting the now known values in equation (3), we have

$F_h = -1{,}500\ (1 - 0.87) = -195$ lbs. per square foot.

Now solving equation (4) for N, we have

$$N = \left\{\frac{-2{,}805 - 195}{2}\right\} 9 = -13{,}500 \text{ lbs.}$$

The value of N can also be determined from equation (5) as the latter equation follows from the fact that

$$\frac{F_t + F_h}{2} = F_a.$$

A graphical illustration of the force relations is seen in the force trapezoid of Fig. 3.

N = area of force trapezoid =

$$\left\{\frac{-2{,}805 - 195}{2}\right\} 9 = -13{,}500 \text{ lbs.}$$

The center of gravity of the force trapeozid is located at a distance x from F_t, where,

$$x = \frac{b}{3}\left\{\frac{F_t + 2(F_h)}{F_t + F_h}\right\} = \frac{9}{3}\left\{\frac{2{,}805 + 2\,(195)}{2{,}805 + 195}\right\}$$
$$= 3.19 \text{ ft.}$$

Hence $e = 4.50 - 3.19 = 1.31$ ft. to the left of the base center c, which demonstrates the fact that the force trapezoid truly represents the assumed conditions.

The force trapezoid may be regarded as representing the relative distribution of the earth reaction under the bottom plane of the foundation. Its force area is equal in magnitude to $+ N$ the normal active downward force component but opposite in direction. Therefore the force elements of the trapezoid must point upward and the resultant of the force trapezoid acts upward, and is equal to $- N$, thus causing the system to be in equilibrium as far as vertical components are concerned. The horizontal component will be considered later in the proper place.

(6) *Calculation of Base Thickness at Toe.*—In the main force trapezoid $ABCD$ (Fig. 4) the distance $AE = 3.0$ ft. represents the adopted length of toe.

Draw the line GC parallel to AB. It is evident from this construction that

$DG = (AD - BC) = (-2{,}805 - (-195)) = -2{,}610$ lbs.

From the similar triangles DGC and FHC we have,

$DG : GC = FH : HC$; hence

$$FH = \frac{DG \times HC}{GC} = \frac{-2{,}610 \times 6.0}{9} = -1{,}740.$$

Furthermore, it is evident that $EF = FH + HE = -1{,}740 - 195 = -1{,}935$.

We shall now consider the toe force trapezoid $ADFE$.

$$\text{Force area } ADFE = \left\{\frac{AD + EF}{2}\right\} AE$$

$$= \left\{\frac{-2{,}805 - 1{,}935}{2}\right\} 3.0 = -7{,}110 \text{ lbs.} = T.$$

The resultant T, equal to the force area of the toe trapezoid $ADFE$, acts through the center of gravity of this trapezoid.

The center of gravity of $ADFE$ is distant z from point E, where

$$z = \frac{AE}{3}\left\{\frac{2\,(AD) + EF}{AD + EF}\right\}$$

$$= \frac{3}{3}\left\{\frac{2\,(2{,}805) + 1{,}935}{2{,}805 + 1{,}935}\right\} = 1.59 \text{ ft.}$$

Regarding the toe of the foundation as a cantilever acted upon by the force T at a distance z from point E, we find that the moment due to the force T is

$M = T\,(z) = 7{,}110\ (1.59) = 11{,}305$ ft.-lbs., or 135,660 in.-lbs.

If

$n = E_s \div E_c = 15$ = ratio of the moduli of elasticity of steel and concrete;

f_s = safe unit fiber stress in steel = 15,000 lbs. per square inch;

f_c = safe unit fiber stress in concrete = 600 lbs. per square inch;

b_1 = width of beam = 12 ins. (see Fig. 2);

d = depth of beam from compressive face to center of gravity of steel;

p = steel ratio = 0.0075;

and R = a constant = $p.f_s.j.$ = 98.44;

then

$$d = \sqrt{\frac{M}{R\,.\,b_1}} = \sqrt{\frac{135{,}660}{98.44 \cdot 12}} = 10.72 \text{ ins.}$$

Allowing for sufficient concrete covering for the steel the minimum thickness of the base, bending moment effect alone being considered, is 13 ins.

Let us use a thickness of 2 ft. for the foundation to provide amply against the shearing effect of the stem and for the earth load action on the heel.

(7) *Magnitude and Location of Horizontal Component of Earth Pressure on Stem.*—According to Rankine, the pressure H_h at a depth h, in a bank of earth whose upper surface is horizontal, is given by the expression,

$$H_h = w\,h\left\{\frac{1 - \sin\theta}{1 + \sin\theta}\right\},$$

where w is weight of a cubic foot of earth and θ is the angle of repose.

Since w and $\left\{\frac{1 - \sin\theta}{1 + \sin\theta}\right\}$ are both constants for any given case it follows that the pressure varies directly with h, and hence the variations in pressure can be represented by the parallel elements of a triangle ABC, as in Fig. 5.

If θ, the angle of repose, is 30°, the expression $\left\{\frac{1 - \sin\theta}{1 + \sin\theta}\right\}$ reduces to $\frac{1}{3}$, and

$$BC = H_h = \frac{wh}{3}.$$

The summation of all the parallel forces acting upon AB (Fig. 5) is readily accomplished by means of the integral calculus and is given by the expression,

$$H_t = \frac{w\,h^2}{6}.$$

It is evident that this expression represents the area of a triangle whose base is $\frac{wh}{3}$ and whose altitude is h.

The force H_t is equivalent to the total action of the forces varying in intensity from zero at the top of the wall to $\frac{wh}{3}$ at the depth h. Force H_t acts through the center of gravity of the force triangle ABC (Fig. 5). The vertical ordinate of the center of gravity is equal to $\frac{h}{3}$.

Since $h = 16$ ft., H_t acts $\frac{h}{3} = \frac{16}{3} = 5.34$ ft. above the top surface of the foundation, or since the foundation is 2 ft. thick, H_t acts 7.34 ft. above the bottom surface of the foundation (see Fig. 7).

Since w, the weight of a cubic foot of earth, is 100 lbs.,

$$H_t = \frac{wh^2}{6} = \frac{100\,(16)^2}{6} = 4{,}267 \text{ lbs.}$$

(8) *Stem Thickness.*—The moment of H_t referred to point D of the line AD (Fig. 7) is,

$M = H_t\ (5.34) = 4{,}267 \times 5.34 = 22{,}786$ ft.-lbs., or 273,432 in.-lbs.

$$d = \sqrt{\frac{M}{R\,.\,b_1}},$$

where $R = p.f_s.j = 98.44$, and $b_1 = 12$ ins.

Hence

$$d = \sqrt{\frac{273{,}432}{1181}} = 15.2 \text{ ins.}$$

To cover the steel reinforcement we need about 3 ins. This would make a minimum required thickness of stem of 19 ins. Had we regarded the altitude h of the pressure triangle ABC as 18 ft. (the distance to the bottom of the foundation) then H_t would equal 5,400 lbs., and the arm $\frac{h}{3} = \frac{18}{3} = 6$ ft., which multiplied by H_t would give a moment of

$H_t\ (6.0) = 5{,}400\ (6) = 32{,}400$ ft.-lbs., or 388,800 in.-lbs. To resist this bending moment d must equal 18.1 ins., which means a minimum working thickness of 21 ins.

It is evident that the above calculation assumes that the stem is a cantilever beam loaded with a concentrated load H_t.

Since the 2-ft. thickness of base, which is included in $h = 18$ ft., is not productive in exerting pressure on the stem, the moment calculated with $h = 18$ ft. is, no doubt, in excess of the actual moment.

We may find later in the investigation that more mass will be needed in the stem in order to satisfy the conditions inherent in the arbitrary location of the final resultant. Therefore, we leave the actual determination of the stem thickness until we have considered the requirements imposed upon the vertical loads. The sole purpose of the present investigation concerning the stem thickness is to set a minimum limit to the stem thickness.

(9) *Location of Resultant of All Vertical Force Components.*—If we define the factor of stability against overturning as the ratio of the resisting moment to the overturning moment, we have:

$$\text{Factor of stability} = \frac{\text{Resisting moment of entire structure}}{\text{Overturning moment of structure.}}$$

Referring to Fig. 6, where line AB represents the bottom of the foundation, we find:

Resisting moment $= (W_e + W_c)\,AD$, where W_e = earth load, and W_c = wall load.

If the wall is to overturn, it must do so about the point A. Consequently the resisting moment must be calculated with point A as a center of moments. Therefore, the arm of the combined force $(W_e + W_c)$ is AD. The length AD is unknown at this stage of the computation. In order to find the length of AD we shall proceed as follows:

$AD = AC + CD = 4.5$ ft. $+ x$, where $x = CD$.

Overturning moment $= H_t\,(y)$.

Factor of stability $=$

$$\frac{(W_e + W_c)\,AD}{H_t\,y} = \frac{(W_e + W_c)\,(4.5 + x)}{H_t\,(7.34)}$$

Since the triangle OED (Fig. 6) is a proportional triangle in which the ratio of the sides OD and ED is equal to the ratio of the forces $(W_e + W_c)$ and H_t, it follows that

$$\frac{W_e + W_c}{H_t} = \frac{OD}{ED}.$$ Substituting this value in the expression for the factor of stability, we have:

$$\text{Factor of stability} = \left(\frac{W_e + W_c}{H_t}\right) \times \frac{AD}{y}$$

$= \dfrac{OD}{ED} \times \dfrac{AD}{y}$, but since $OD = y$, the expression becomes:

$$\text{Factor of stability} = \frac{AD}{ED} = \frac{4.5 + x}{1.31 + x},$$ because $ED = EC + CD = e + x = 1.31$ ft. $+ x$.

Equating this latter value of the factor of stability with the former, we obtain,

$$\frac{(W_e + W_c)\,(4.5 + x)}{H_t\,(7.34)} = \frac{4.5 + x}{1.31 + x}$$

Since the vertical or normal component N of the final resultant must be equal to the sum of the contributing vertical components W_e and W_c, it is evident that

$$(W_e + W_c) = N = 13{,}500 \text{ lbs.}$$

Hence all the values in the above equation, except x, are known, since $H_t = 4{,}267$ lbs. Substituting these values the expression becomes

$$\frac{13{,}500\,(4.5 + x)}{4{,}267\,(7.34)} = \frac{4.5 + x}{1.31 + x},$$ which reduces to the form

$$13{,}500\,x^2 + 47{,}115\,x - 61{,}357 = 0;$$

and since this quadratic equation is in the form

$ax^2 + bx + c = 0$, in which

$$x = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a},$$ we have by proper substitution, $x =$

$$\frac{-47{,}115 \pm \sqrt{(47{,}115)^2 - 4\,(13{,}500)\,(-61{,}357)}}{2\,(13{,}500)}$$

$$= \frac{-47{,}115 \pm 74{,}385}{27{,}000}$$

Using the negative sign, $x = -4.5$ ft.

Using the positive sign, $x = +1.01$ ft.

For the value $x = -4.5$, the expression for the factor of stability vanishes. Consequently the true value of x is 1.01 ft. to the right of the base center C.

By the above physico-mathematical maneuver we have found the location of the combined vertical component $(W_e + W_c)$.

(10) *Calculation of Factor of Stability.*—For the above imposed conditions the

$$\text{factor of stability} = \frac{13{,}500\,(4.5 + 1.01)}{4{,}267\,(7.34)} = \frac{4.5 + 1.01}{1.31 + 1.01} = 2.37, \text{ since } x = 1.01 \text{ ft.}$$

(11) *Calculation of Earth Load.*—In Fig. 7 the lengths FE, DE, AG and CD are known at this stage of the work. Suppose we assume that the minimum length KD of the heel shall be equal to the toe length $AG = 3.0$ ft.

Then the cross-sectional area of the stem must be contained within the area $GBLK$. (This assumes the front face BG vertical. A slight batter may be given to this face.)

A line joining points B and K will represent the limiting position of the back face of the stem. In this position the stem thickness at the top is zero. Practical reasons eliminate this position from further consideration. Suppose that we decide to make the top stem thickness not less than 1 ft., then if BK is revolved about its own center point O until $BN = 1$ ft., the new position of line BK will be NM, for which position the batter of the back face of the stem will be ¾ in. for each foot of vertical height. This will give us an earth trapezoid $NCDK$ whose upper base NC is 5 ft. and whose lower base MD is 4 ft., with an altitude CD equal to 16 ft.

Let us now determine the abscissa of the center of gravity of the earth trapezoid in respect to the axis Y-Y (Fig. 8).

Subdivide the trapezoid into the triangle MNS and the rectangle $MSCD$.

It is a well known fact that the center of gravity of the triangle MNS must be located along a line through point U parallel to the base MS, provided that US is equal to ⅓ of NS. Since $NS = 1.0$ ft., $US = 0.33+$, or say 0.34 ft.

Note that MS is parallel to axis Y-Y and that distances are measured perpendicularly to Y-Y.

Similarly the center of gravity of the rectangle is found in the line XV, parallel to Y-Y and at a distance of 2 ft. from the axis.

Let the area of the rectangle $MSCD$ be represented by A, and the area of the triangle MNS be represented by B. Furthermore, let a and b be the respective distances from axis Y-Y to the center of gravity of A and B; then the following equation will enable us to find the distance x of the center of gravity of the combined area $(A + B)$ from the axis Y-Y:

$$A\,(a) + B\,(b) = (A + B)\,x,$$

hence,

$$x = \frac{A\,(a) + B\,(b)}{(A + B)}$$

The calculation for x may be conveniently tabulated as shown in Table I.

TABLE I.—CALCULATIONS FOR DETERMINING VALUE OF X.

Section.	Area.	Arm.	Area Moment.
A	$16.0 \times 4.0 = 64.0$ sq. ft.	$a = 2.0$ ft.	$A\,(a) = 64.0 \times 2.0 = 128.00$
B	$\dfrac{16.0 \times 1.0}{2} = 8.0$ sq. ft.	$b = 4.34$ ft.	$B\,(b) = 8.0 \times 4.34 = 34.72$
A + B	72.0 sq. ft.	$x = \dfrac{A(a)+B(b)}{(A+B)}$; $x = \dfrac{162.72}{72.0} = 2.26$ ft.	$A\,(a) + B\,(b) = 162.72$

(12) *Earth Load Area as a Single Force.*—Since we are considering a slice of earth between two parallel planes 1 ft. apart, the volume corresponding to the earth trapezoid $MNCD$ is $72.0 \times 1.0 = 72.0$ cu. ft. If the weight of the earth be taken at 100 lbs. per cubic foot, the earth load W_e is—

$$W_e = 72.0 \times 100 = 7{,}200 \text{ lbs.}$$

This earth load W_e acts at a distance 2.26 ft. to the left of the axis Y-Y, or 2.24 ft. to the right of the base center C, since $4.5 - 2.26 = 2.24$ ft.

(13) *Magnitude and Location of Wall Load Resultant.*—In the following discussion we shall use the conventional signs regarding forces and distances adopted in Section 4.

The "Final Resultant" (Fig. 9) can be replaced by its vertical component $+N = +13{,}500$ lbs., and its horizontal component $-H_t = -4{,}267$ lbs., both acting at the point E of the line AB, which represents the bottom surface of the foundation. The system is not in equilibrium under the action of these two components. Consequently we must introduce two components equal in magnitude and acting in directions opposite to the above at point E. Therefore, in Fig. 9, we have added $-N = -13{,}500$ lbs. and $+H_t = +4{,}267$ lbs., both acting at the point E, in order to produce equilibrium.

In Fig. 10, there is shown the equilibrated system. Above line AB we show the individual forces $+W_e$, $+W_c$, and $-H_t$ which, when combined, produce the final resultant or its equivalent $+N$ and $-H_t$.

At this stage of the investigation the magnitude and location of $+W_e$ and $-H_t$ are known. It remains to determine the magnitude and location of W_c, the wall load.

Since

$N = W_e + W_c$, it follows that

$W_c = N - W_e = 13{,}500 - 7{,}200 = 6{,}300$ lbs.

Referring once more to Fig. 10, let us take moments about the mid-point "C" of AB; then

$$+W_e\,(+2.24) + W_c\,(x) - H_t\,(+7.34) - N\,(-1.31) = 0.$$

The arm x of W_c is the only unknown quantity in this equation, which represents algebraically a system in equilibrium since the sum of the moments of all the forces equals zero. Substituting the values of the known quantities, we have,

$$+7{,}200\,(+2.24) + 6{,}300\,(x) - 4{,}267\,(+7.34) - 13{,}500\,(-1.31) = 0.$$

from which,

$$x = -\frac{2{,}583}{6{,}300} = -0.41 \text{ ft. to the left of point } C.$$

(14) *Calculation of Wall Load Area.*—If we take the weight w of a cubic foot of concrete at 150 lbs., we have

$$V = \frac{W_c}{w} = \frac{6{,}300}{150} = 42 \text{ cu. ft.}$$

where V is the volume in cubic feet.

Now since all loads have been calculated between two parallel vertical planes distant 1 ft., it follows that

$A \times 1.0 = V$, where A is the area shown in the section.

Hence, $A = V \div 1 = 42.0$ sq. ft.

This area A is made up, in this case, of the stem and the base, or foundation.

(15) *Dimensions of Wall Stem.*—Since A = stem area + base area, we may adopt a definite area for either (consistent, however, with the minima already determined from other requirements), and then force the dimensional distribution of the remaining area to satisfy the condition which we have imposed upon the system when we determined the location of the wall load W_c.

Let us therefore arbitrarily remain with our assumptions concerning the dimensions of the earth trapezoid which was equivalent to a tentative determination of the dimensions of the stem. In Fig. 11, the stem $MNBG$ is shown. The top $BN = 1.0$ ft., and the thickness $GM = 2.0$ ft. This is safe, since it is in excess of the requirements due to the earth pressure H_t.

(16) *Calculation of Base Thickness Necessary to Conform with Imposed Conditions.*—In Fig. 11 we have subdivided the stem into two areas, a triangle MNL, whose area is designated by the letter C, and a rectangle $LNBG$, whose area is designated by the letter D.

For the area of the base we shall use the letter E.

The respective centers of gravity of C, D, and E are shown in Fig. 11.

Since the length, $FS = h = 9.0$ ft., has already been determined, we must calculate RS, the thickness necessary to conform with the following two conditions.

(*a*) Stem area + base area = 42.0 sq. ft.

(*b*) Wall load W_c must act 0.41 ft. to the left of the base center C, or 4.91 ft. to the left of axis Y-Y.

Letting $x = RS$ = the required base thickness, and taking moments about the axis Y-Y, we have,

$C\,(4.67) + D\,(5.5) + 9.0\,(x)\,(4.5) = 42.0\,(4.91)$.

Area $C = (16 \times 1) \div 2 = 8.0$ sq. ft.

Area $D = (16 \times 1) = 16.0$ sq. ft.

Substituting these values of C and D in the above equation we have,

$8.0\,(4.67) + 16.0\,(5.5) + 9.0\,(x)\,(4.5) = 42.0\,(4.91)$,

or $x = 2.0$ ft.

DISCUSSION.

The consideration of the required steel reinforcement is omitted in this article for the reason that expositions of the method involved in the steel determination can be found in any of the works on reinforced concrete.

The reader has undoubtedly discovered numerous modifications in the above outlined method and procedure. The writer is aware of a number of them, for instance, the wall load W_c may be determined before the earth load is determined. It is not the purpose of this article to exhaust the possibilities but to suggest them.

We have advisedly entered into numerous calculation details in order that even the engineering student may appreciate the possibilities of the outlined method.

We have proceeded from the soil up, in the same manner in which the structure is built, instead of beginning at the top and proceeding downward to the soil only to find, perhaps, that we have exceeded the bearing power of the soil upon which the structure is to rest. We could have calculated the factor of stability against overturning for the proposed structure at the very beginning of the investigation.

Furthermore, it is evident that the calculations can be reduced to a few simple steps not in the least more cumbersome than the present orthodox procedure and much more positive and certain in its ultimate effect. The writer wishes to emphasize the fact that the selection of a retaining wall was not a choice due to necessity but due solely to the fact that the elements involved are known almost universally among engineers.

Dams, piers, and abutments can be designed by the method here outlined.

A fertile, though more difficult field of application, is that of concrete arch designing.

In conclusion, the writer expresses the hope that the method here suggested will be productive in turning the trend of engineering thought into a new domain, fertile in possibilities and fruitful in valuable results.

Design, Construction and Detailed Costs of the Richelieu River Bridge, Lacolle Junction, Quebec.

(Staff Article.)

In our issue of Dec. 9, 1914, we published an article describing and illustrating the design and construction features connected with the replacement under traffic of an old railroad bridge at Lacolle Junction, Quebec, by a new structure consisting of a 250-ft. swing span and twelve plate girder spans. In this issue we shall give the detailed costs of this work. The substructure work occasioned by the renewal of this bridge is of particular interest, and the cost data are complete.*

Part II—Cost Data.

The cost of the substructure was $155,955.49, and that of the superstructure $77,877.79, a total of $233,833.28.

The detailed cost data which follow represent actual costs and include all possible charges against the structure. They were taken from the contractor's records and give the correct labor distributions and material costs. All railway departmental charges were obtained from the auditor's statement. The superstructure was erected by the railway company, while the substructure work was done on a percentage basis, the contractor receiving, as profit, 6½ per cent of the cost of all labor and materials.

DISTRIBUTION OF COSTS.

Table I gives the distribution of the costs of the substructure and superstructure of the bridge, taken from the auditor's statement.

TABLE I. — DISTRIBUTION OF SUBSTRUCTURE AND SUPERSTRUCTURE COSTS.

Substructure		
Engineering		$ 2,705.[illegible]
Grillage beams and freight		2,460.59
Special Accounts:		
Soundings	$ 81.00	
Dredging	845.00	
Damage to barge	75.00	1,001.00
Transportation Dept., switching	$ 1,773.41	
Less superstructure acc't	200.00	1,573.41
Freight on material	$11,395.95	
Less superstructure acc't	1,680.44	9,715.51
Material, G. T. Ry. Purchasing Dept.:		
Stationery	$ 67.58	
Ottawa stores	55,077.53	
Montreal stores	1,222.32	56,367.43
Fuel		2,159.40
J. S. Metcalfe Co.:		
Percentage	$ 9,263.09	
Less superstructure acc't	74.29	9,188.80
Expenditure — Supt., $2,635.12; mat., $1,100.66	$ 3,735.78	
Less superstructure acc't Sup't	50.00	3,685.78
Labor	$66,063.95	
Less superstructure acc't	1,143.07	64,920.88
B. & B. Dept.:	$ 8,159.32	
Less superstructure acc't	6,312.12	*1,847.20
Motive Power Dept. for B. & B.:		
Driving piles		329.61
Total, substructure		$155,955.49

*Material, $1,200.00, labor, $647.20.

Total Material:	
Stores Department	$56,367.43
J. S. Metcalfe Co.	1,100.66
B. & B. Department	1,200.00
Total	$58,668.09
Total Labor:	
Labor	$64,920.88
Superintendence	2,585.12
Sounding	81.00
B. & B. Dept	647.20
Total	$68,234.20

Superstructure		
Engineering		$ [illegible]
Dominion Bridge Co.		70,201.92
Motive Power Dept.—Power house		102.77
B. & B. Dept.:		
Removing ties and painting girders	$ [illegible]	
Bridge floor	5,050.00	
Placing runouts, temporary trestle spur and removing old deck during erecting of steel	[illegible]	6,312.12
Transportation charges, switching		200.00
J. S. Metcalfe Co.:		
Operating bridge during const.:		
Labor	$[illegible]	
Superintendence	50.00	
Percentage	74.29	1,267.36
Road Department		[illegible]
Freight on old and new steel		1,680.44
Total		[illegible]
Credit:		
Scrap steel	[illegible]	
Grillage beams	$2,[illegible]	
Freight	[illegible]	
		2,726.17
Total Superstructure		$77,877.79

MATERIALS.

The following data give the materials used, where they were obtained, and the prices paid for them:

Sand, f. o. b. Swanton, Vt., 32 cts. per ton, duty free.
Crushed stone, f. o. b. Chazy, N. Y., 75 cts. per ton, duty 17½ per cent.
Rubble stone, f. o. b. Chazy, N. Y., 60 cts. per ton, duty free.
Rubble stone, f. o. b. bridge site, by barge from Isle La Motte, Vt., 85 cts. per ton, duty free.
Cement, f. o. b. Belleville, Ont. (Canadian brand), net $1.25 per barrel.
Lumber, McAuliffe-Davis Lumber Co., Chicago:
  2 x 4-in. x 10 to 14-ft. hemlock at $15 per M.
  2 x 6-in. x 10 to 14-ft. hemlock at $15 per M.
  2 x 8-in. x 10 to 14-ft. hemlock at $15 per M.
  1 x 6 to 8-in. x 10 to 18-ft. hemlock at $15 per M.
  1 x 6-in. x 12 to 16-ft. pine form lumber at $19.50 per M.
Lumber, E. Hines Lumber Co., Chicago:
  12 x 12-in. x 26-ft. l. l. y. p. at $28.50 per M.
  12 x 12-in. x 18-ft. l. l. y. p. at $26.50 per M.
  12 x 12-in. x 22 to 24-ft. l. l. y. p. at $27.50 per M.
  10 x 10-in. x 22-ft. l. l. y. p. at $25.50 per M.
  10 x 14-in. x 14 to 25-ft. l. l. y. p. at $34.00 per M.
Lumber, John Harrison & Sons, Owen Sound, Canada:
  10 x 10-in. x 10 to 20-ft. hemlock at $20 per M.
  12 x 12-in. x 10 to 20-ft. hemlock at $20 per M.
Lumber, A. J. Martin, Sherbrooke, Canada:
  10 x 10-in. x 14-ft. hemlock at $18 per M.
  10 x 10-in. x 20-ft. hemlock at $20 per M.
  10 x 10-in. x 22-ft. hemlock at $22 per M.
Lumber, Marsh & Bingham, Chicago:
  10x10-in. x 10 to 14-ft., l. l. y. p., at $28 per M.
  10x10-in. x 16 to 20-ft., l. l. y. p., at $32 per M.
  12x12-in. x 10 to 14-ft., l. l. y. p., at $30 per M.
  12x12-in. x 16 to 20-ft., l. l. y. p., at $31 per M.
  12x12-in. x 26-ft., l. l. y. p., at $33 per M.
Lumber, W. H. Bromley, Canada:
  12 x 12-in. x 12 to 20-ft. red pine, at $25 per M.
Lumber, Colonial Lumber Co., Canada:
  12 x 12-in. x 12 to 20-ft. red pine, at $25 per M.
Lumber, Long Lumber Co., Hamilton, Ontario:
  10 x 10-in. x 12 to 18-ft. hemlock, at $16 per M.
  12 x 12-in. x 16 to 18-ft. hemlock, at $20 per M.
Lumber, R. Laidlaw & Co., Canada:
  12 x 12-in. x 16-ft. Georgia pine, at $35 per M.
  12 x 12-in. x 18-ft. Georgia pine, at $37 per M.
Lumber, W. B. Crane & Co., Chicago:
  12 x 14-in. x 14 to 16-ft. white oak, at $38 per M.

PRICES OF MISCELLANEOUS MATERIALS.

The following prices were paid for the miscellaneous materials listed in the accompanying table:

Reinforcing steel, per 100 lbs.	$ 1.45
Drifts, sheared points without head, per 100 lbs.	1.90
Ship spikes, per 100 lbs.	2.90
Anchor straps, each	4.80
Machine bolts:	
⅞x15-in., per 100	29.40
⅞x24-in., per 100	42.00
½x15-in., per 100	10.62
Nose plates, each	7.15
1 10-in. sheave block	14.00
1 10-in. sheave snatch block	7.50
1 8-in. triple wood block	1.95
Steam hose, per lin. ft.	.93
Suction hose, per lin. ft.	1.33
1 3-ton M. J. duplex block	72.00
1 1-ton M. J. differential block	7.00
1 3-ton Harrington block	67.50
Dynamite, f. o. b. factory, per lb.	.19
Amazon 3-ply roofing paper, per square	2.25
1 motor boat, 18-HP.	600.00

SCALE OF WAGES.

The prices paid for the various classes of labor were as follows:

	Cts. per hr.
Common laborers	22½
Handymen	25
Carpenters	35 to 40
Labor foreman	35
Carpenter foreman	40
General foreman	60
	Per day
Superintendent	$8.00
*Driver	6.50
*Helper	4.50

*Furnished by railway company.

EQUIPMENT FURNISHED BY SUBSTRUCTURE CONTRACTOR.

The accompanying table gives the equipment, and its value, furnished by the J. S. Metcalfe Co., the substructure contractor:

6 Hudson, V-shaped, 1-cu. yd. cars	$ 390
Track	40
1 No. 3 Gould trench pump	25
1 ½-cu. yd. Cube mixer	1,600
1 ½-cu. yd. Smith mixer (old)	400
1 Pulsometer pump	100
1 Emerson, Jr. B pump	120
1 No. 2 Emerson steam pump	368
2 No. 2 Wood electric drills	250
Motor and fittings	600
2 locomotive boilers	800
1 vertical boiler	300
[illegible] wheelbarrows	45
Total	$5,038

WORK DONE BY BRIDGE AND BUILDING DEPARTMENT.

The data given in Table II refer to materials furnished and work done by the Bridge and Building Department of the Grand Trunk Ry.

*The [illegible] and 36 ft. long, notwithstanding a statement to the contrary on p. 543 of our Dec. 9, 1914, issue. As constructed, the booms between protection cribs shown in Fig. 2 of that issue were built up of five parallel lines of 12x12-in. timbers.

[illegible] COST DATA ON WORK DONE BY BRIDGE AND BUILDING DEPARTMENT.

—Piles and Pile Driving—

[illegible] ft. piles (4,670 lin. ft. at 15 cts. per ft.)	$700.00
Motive Power Department	329.61
1[illegible] days' pile driving at $8 per day, labor $16[illegible]	232.00
Labor, cutting piles east abutment	[illegible]
Labor, cutting piles by driver, average about 8 piles per day at $17 per day; total, 62 piles in 8 days	136.00
Freight	80.00
Overhead charges	79.00
Total cost (per ft., 33.8 cts.)	$1,576.61

—Supporting Track by Bridge Dept.—

Labor	$ 415.20
Material	500.00
Freight	120.00
Overhead charges	71.00
Total	$1,106.20

—Work Done by J. S. Metcalfe Co. as Part of B. & B. Dept.'s Work—

This work consisted of changing trestle bents, supporting track and removing old trestle piles.

Labor	$1,411.57
Material	200.00
Transportation	23.44
Freight	70.00
Overhead charges	159.00
Total	*$1,864.01

*$1,864.01 — $20.00 = $1,844.01.

Table III gives cost data on work done by the diver, such as cutting off piles and excavation for caissons and protection cribs.

TABLE III.—COST DATA ON WORK DONE BY DIVER.

Cutting off piles in foundation:	
Labor	$100.00
Material	20.00
Freight	2.75
Overhead charges	13.25
Total	$136.00
Cutting of old trestle piles inside and outside of caissons:	
Labor	$100.00
Material	20.00
Freight	2.76
Overhead charges	13.75
Total	$136.51
Excavation by diver: This account was for excavation work for caissons (area, 6,400 sq. ft.; cost, $1,975), and for protection cribs (area, 2,000 sq. ft.; cost, $851).	
Labor	$2,400.00
Material	130.00
Freight	10.00
Overhead charges	286.00
*Total	$2,826.00
*Cost per square foot (8,400 sq. ft.) =	34 cts.
Grand total	$3,098.51

DATA ON OPEN TIMBER CAISSONS.

Table IV gives the quantity of timber required for the caisson of each pier and abutment, the various labor and material costs of each caisson, the cost of miscellaneous items, the total cost of each caisson, and its unit cost. The contractor's percentage and the engineering costs are not included (for contractor's percentage of the total cost of each pier and abutment see Table VII). The overhead charges given include: construction buildings, timekeeper, tool boys, watchman, shoveling snow, superintendence, general expense, equipment, labor and material.

TABLE IV.—COST DATA ON OPEN TIMBER CAISSONS. ENGINEERING AND CONTRACTOR'S PERCENTAGE NOT INCLUDED.

Timber used, ft. B. M.

	Double wall caisson: Permanent wall	Double wall caisson: Temporary wall	Double wall caisson: Total	Single wall caisson: Permanent wall	Single wall caisson: Temporary wall	Single wall caisson: Total
West abutment						
Pier No. 1				5,520	3,680	9,200
Pier No. 2				3,000	3,680	6,680
Pier No. 3				5,520	3,680	9,200
Pier No. 4	15,818	6,480	22,298			
Pier No. 5	[illegible]	6,480	[illegible]			
Pier No. 6	[illegible]	[illegible]	[illegible]			
Pier No. 7	11,700	40,440	52,140			
Pier No. 8	91,000	18,000	109,000			
Pier No. 9	[illegible]	[illegible]	79,440			
Pier No. 10	[illegible]	[illegible]	16,125			
Pier No. 11	[illegible]	[illegible]	18,13[illegible]			
Pier No. 12				9,200	3,680	12,880
Pier No. 13				5,520	3,680	9,200
East abutment						
Total	344,727	64,200	408,927	28,760	18,400	47,160

Total costs of various items

	Labor: Framing	Labor: Setting single wall	Labor: Rip-rap	Labor: Unloading material	Material: Timber	Material: Tools	Material: Rip-rap	Material: Iron	Material: General	Miscellaneous: Fuel	Miscellaneous: Freight	Miscellaneous: Transportation	Miscellaneous: Overhead charges	Total	Cost per M ft. B. M., total
West abutment	.....	.....	$108	.....	.....	.....	$170	.....	.....	.....	$80	$51	$39	$448	
Pier No. 1	$274	$20	.....	$[illegible]	$157	$15	.....	$22	$[illegible]	$5.00	22	6	60	598	$65.0[illegible]
Pier No. 2	220	20	35	[illegible]	93	8	55	15	[illegible]	4.00	16	17	43	535	80.39
Pier No. 3	246	20	31	[illegible]	160	15	49	24	[illegible]	5.00	22	17	60	665	72.28
Pier No. 4	[illegible]	36	37	[illegible]	380	31	60	54	18	11.00	33	27	145	1,665	74.67
Pier No. 5	[illegible]	36	63	[illegible]	560	45	102	78	27	16.00	80	43	216	2,287	68.57
Pier No. 6	1,605	36	76	[illegible]	650	51	118	90	32	20.00	90	49	249	3,104	80.81
Pier No. 7	1,478	41	44	[illegible]	880	71	55	122	44	30.00	124	12	338	3,321	63.69
Pier No. 8	3,131	[illegible]	47	101	1,830	148	122	255	92	54.00	260	90	705	6,956	63.8[illegible]
Pier No. 9	1,9[illegible]	41	38	69	1,000	80	64	140	50	26.00	112	48	384	3,958	66.5[illegible]
Pier No. 10	1,786	41	12	48	780	64	70	108	39	23.00	112	45	300	3,457	74.9[illegible]
Pier No. 11	1,066	41	12	18	810	64	18	115	40	24.00	116	30	312	2,696	56.0[illegible]
Pier No. 12	266	20	32	9	230	16	48	30	11	6.00	31	19	83	801	62.1[illegible]
Pier No. 13	165	20	15	6	160	15	25	22	8	5.40	3[illegible]	11	60	534.40	58.09
East abutment	.....	.....	109	.....	.....	.....	171	.....	.....		80	52	39	451	
Total	$13,881	$459	$724	$469	$7,690	$623	$1,127	$1,075	$382	$229.40	$1,250	$[illegible]45	$4,033	$31,478.40	$69.02*

*Average.

CONCRETE WORK.

Table V gives the quantity of concrete placed in each pier, the various labor and material costs for each pier and abutment, the cost of miscellaneous items, the total cost of the concrete in each pier and abutment, and its unit cost. The contractor's percentage and the engineering costs are not included. (For contractor's percentage of the total cost of each pier and abutment see Table VII.) The overhead charges given in Table V include: construction buildings, shoveling snow, timekeeper, tool boys, watchman, superintendence and general expenses. The concrete was deposited under water by bucket, the average depth of water being about two-thirds of the height of the pier. In connection with the concrete work the pumping account amounted to $1,676.92, divided as follows: labor, $797.92; fuel, $530; miscellaneous material, $200; and overhead charges, $149; this amounts to 31.2 cts. per cubic yard of concrete in the piers and abutments.

CRIB PROTECTION WORKS, BOOMS AND WALING.

Table VI gives the quantities of materials used in the cribs, booms and waling, the labor and material costs of various items, the cost of miscellaneous items, the total costs of the cribs, booms and waling, and their unit costs. The engineering costs are not included. The cribs were filled with stone from barges, the rip-rap being unloaded from trains. The total cost of the wing cribs and booms was $15,594, and that of removing the old protection work $1,190.98 (for details see Table VIII).

SUMMARY OF PIER COSTS.

Table VII gives a summary of the cost of the various items of each pier and abutment, the total cost of each pier, and the total cost of all piers and abutments. The engineering costs are not included. The total cost given for the pivot pier, No. 8, does not include the cost of removing the old pier, which was $900. Cofferdams were constructed for the two abutments and for pier No. 1. The cofferdam used for pier No. 1 consisted of 2-in. sheeting driven flush with the outside of the caisson and banked with clay on the outside. The cost given for this cofferdam, $306.98, includes the cost of excavating for the pier and that of puddling. The excavation work for the two abutments (cost, $300) was done by ordinary labor; that for pier No. 11 (cost, $600) was done by a dredge; and that for the remaining piers (cost, $1,975), by divers. The cost given for the caisson of the west abutment ($448) was for rip-rap only. The distance from the top of masonry to the base of rail is 8 ft. 6 ins.

The itemized costs of the pivot pier (No. 8) are shown in Table VII. These costs do not include engineering nor removal of old pier, the costs of which were $416 and $900, respectively. If these items are included, the total cost of the pivot pier is $24,517.96. The total cost of the concrete work for this pier was $11,827.45, which was divided as follows:

12-in. wall, 140 cu. yds. at $9.00	$ 1,260.00
Encasing walls and concrete burlap bags, 650 cu. yds. at $8.46	5,500.00
Rubble grouting, 450 cu. yds. = 150 cu. yds. grout at $12.00	1,800.00
Pier top, 360 cu. yds. at $8.22	2,969.45
Forms	298.00
Total	$11,827.45

The estimated cost of the pivot pier, including its proportion of the general charges, freight, interest and depreciation of plant, contingencies, engineering, and superintendence, was $31,250. The difference between this and the actual cost of about $24,500 is due mainly to: (a) Grouting the rubble filling instead of removing it and replacing with concrete (approximate saving, $3,500); (b) lower cost of concrete than estimated (approximate saving, $2,500); and (c) lower cost of caisson (approximate saving, $750).

Table VIII gives a general summary of the costs of the bridge, the tabulation being made in such a manner as to show the costs of labor and superintendence, material, transportation, fuel, freight, overhead charges and totals for each item of the work. It will be noted that the cost of labor and superintendence was about $9,600 in excess of that of materials.

In Fig. 11 the heights of the piers have been platted as ordinates and their total costs as abscissas. Excluding the rest piers, which are special cases, it is seen that the height-cost curve is practically a straight line.

UNIT COSTS.

The following unit costs include, in addition to the items noted, train service and freight, but do not include contractor's percentage nor engineering charges:

Timber in Caissons. The average cost of the timber in the caissons, framed, erected and in place, including materials, tools, equipment and labor, was $69.02 per M ft. B. M.

Timber in Protection Crib Work. The average cost of the timber in the crib protection work, framed, erected and in place, including materials, tools, equipment and labor, was $60.87 per M ft. B. M. The prices of the various kinds of timber, per M ft. B. M.,

TABLE V. COST DATA ON CONCRETE WORK—ENGINEERING AND CONTRACTOR'S PERCENTAGE NOT INCLUDED.

Structure	Quantity, cu. yds.: In pier exclusive of 12 in. walls	Between caisson walls	Total	Total costs of various items. Labor: Mixing and placing, excluding 12-in. walls	Mixing and placing between 12 in. walls	Unloading material	Forms	Temporary tracks	Equipment	Material: Cement, sand and crushed stone	Forms	Temporary track	Equipment	Tools	Miscellaneous material	Miscellaneous: Transportation, watching	Freight	Fuel	Overhead charges	Total cost	[illegible]
West abutment	262	.....	262	$283.00	.....	$139.11	$359	$15	$13	$727	$95	$12	$21	$16	$55	$29	$282	$39	$150	$2,275.11	$8.68
Pier No. 1	109	....	109	92.00	...	58.00	125	24	19	302	22	18	10	6	27	13	118	16	64	914.00	8.39
Pier No. 2	78	...	78	147.00	...	42.00	78	33	13	213	22	25	7	5	19	9	88	12	60	770.00	9.87
Pier No. 3	113	....	113	175.00		92.00	130	42	15	314	25	32	11	7	28	13	123	17	65	1,093.00	9.67
Pier No. 4	225	32	257	286.00	$47	136.00	158	51	42	712	25	40	22	15	54	30	283	38	147	2,096.00	8.16
Pier No. 5	260	40	300	353.00	90	159.00	139	60	49	832	25	15	26	17	75	37	325	44	172	2,446.00	8.15
Pier No. 6	283	42	325	411.00	59	172.00	180	69	73	900	25	52	27	19	80	38	352	48	186	2,674.00	8.23
Pier No. 7	512	83	595	614.00	209	310.00	221	78	97	1,650	32	60	50	35	148	70	645	88	349	4,647.00	7.81
Pier No. 8	1,460	140	1,600	1,854.45	286	848.00	245	95	262	4,548	53	78	131	93	400	185	1,639	240	870	11,827.45	7.39
Pier No. 9	470	74	544	847.00	253	290.00	234	100	89	1,515	32	90	41	32	134	65	590	80	312	4,704.00	[illegible]
Pier No. 10	310	53	363	582.00	180	190.00	221	51	59	1,014	25	10	30	22	90	42	392	53	210	3,201.00	[illegible]
Pier No. 11	320	55	375	527.00	186	198.00	188	39	62	1,045	25	32	32	22	93	43	407	56	216	3,171.00	8.46
Pier No. 12	159	.....	159	125.00	......	84.00	136	30	26	440	22	25	14	9	39	18	172	24	91	1,255.00	7.90
Pier No. 13	113	....	113	225.00	.....	60.00	113	21	19	314	22	19	10	7	28	13	123	17	65	1,056.00	9.34
East abutment	188	.....	188	360.00†	..	100.00	414	12	20	522	100	12	10	11	17	22	204	28	108	1,980.00	10.52
Total	4,862	519	5,381	$6,884.45	$1,310	$2,878.11	$2,941	$720	$882	$17,048	$550	$580	$412	$316	$1,337	$625	$5,740	$800	$3,056	$44,109.[illegible]	$8.20*

*Average. †Mixed by hand.

were: hemlock and spruce, $20, red pine $25; long leaf yellow pine, $26 to $31.

Concrete.—The average cost of the concrete, including materials, equipment, tools and labor, was $8.20 per cubic yard. The prices of the materials were: crushed stone, $1.10 per cubic yard ($0.75 per ton); sand, $0.32 per cubic yard; and cement, $1.25 per barrel, net.

Excavation.—The cost of the excavation, including equipment and labor, for the three classes of work was as follows:

By diver, preparing bottom, 8,100 sq. ft., $0.[illegible] per square foot.

By orange-peel bucket, 200 cu. yds., $4.[illegible] per cubic yard.

By ordinary labor, 200 cu. yds., $1.50 per cubic yard.

Rip-rap.—The cost of the 1,300 cu. yds. of rip-rap, including equipment and labor, was $2.25 per cubic yard.

Stone Filling.—The 5,400 cu. yds. of stone filling, including equipment and labor, cost $1.69 per cubic yard.

Piles.—The 40-ft. hardwood piles cost, in place, $13.48 each, or $0.34 per linear foot.

TABLE VI. COST DATA ON CRIB PROTECTION WORKS, BOOMS AND WALING—ENGINEERING NOT INCLUDED.

Structure	Quantities: Timber, ft. B. M.	Rubble stone, cu. yds.	Total cost of various items. Labor: Unloading material	Framing	Excavation by diver	Filling cribs	Rip-rap	Material: Timber	Iron	Rubble stone	Tools	Miscellaneous: Transportation	Freight	Fuel	Overhead charges	Total cost	Unit cost, per M. ft. B. M.
[illegible]	550,316	[illegible]	[illegible]	$7,085.00	$505	$1,737.15	$135.09	$11,080	$1,000	$3,500	$250	$200	$1,310	$140	$3,106	$30,128.21	$55.30
[illegible]	15,300		20	793.72				195	55	......	35	14	80	......	169	1,661.72	108.45
Booms	4,696		16	[illegible]				705	35	......	40	16	120	......	248	2,417.00	97.87
Waling [illegible]	[illegible]	[illegible]	200	[illegible]	346	806.00	101.00	1,260	490	1,213	100	100	690	60	1,216	13,177.00	68.37
Total	[illegible]	[illegible]	[illegible]	$12,690.72	$851	$2,543.15	$236.09	$16,540	$1,5[illegible]	$4,713	$125	$330	$2,200	$200	$4,739	$47,683.96	$60.87
Contractor's percentage.....																$3,097.00	
Grand total cost...																$50,780.96	$64.85

TABLE VII. SUMMARY OF COSTS OF PIERS AND ABUTMENTS OF RICHELIEU RIVER BRIDGE.

	West abutment	Pier No. 1	Pier No. 2	Pier No. 3	Pier No. 4	Pier No. 5	Pier No. 6	Pier No. 7	Pier No. 8	Pier No. 9	Pier No. 10	Pier No. 11	Pier No. 12	Pier No. 13	East abutment	Total
Height [illegible], ft.	14.5	11.5	11.5	11.5	20.5	26.0	28.5	30.5	35.5	33.0	31.0	32.5	18.7	14.5	9.[illegible]	
Piles			...	...	....	....	....	....	....	....	....	....	$485.00	$185.00	$606.61	$1,[illegible]
Excavation	$150.00		$140.00	$140.00	$150.00	$150.00	$150.00	$190.00	$420.00	$190.00	$150.00	$600.00	155.00	140.00	150.00	2,875.00
Caisson	148.00	$598.00	537.00	665.00	1,665.00	2,287.00	3,104.00	3,321.00	6,956.00	3,958.00	3,457.00	2,696.00	801.00	534.40	451.00	31,478.40
Pumping	59.00	90.00	96.00	90.00	125.00	125.00	125.00	150.00	201.92	150.00	125.00	125.00	90.00	90.00	50.00	1,676.92
Concrete and forms	2,275.11	914.00	770.00	1,093.00	2,096.00	2,446.00	2,674.00	4,647.00	11,827.45	4,704.00	3,201.00	3,171.00	1,255.00	1,056.00	1,980.00	44,109.56
Cofferdam	230.00	306.98													520.00	1,356.98
Soundings	26.00	21.60	10.90	14.50	10.90	12.70	12.70	27.45	81.10	11.50	10.90	14.50	59.60	50.20	....	361.45
Grillage beams					....	....	....	....	2,460.59	......						2,460.59
Contractor's percentage	225.00	125.00	100.00	139.50	262.00	325.50	394.00	541.90	1,255.00	586.00	451.00	430.00	185.00	152.00	244.00	5,415.00
Total	$3,098.11	$2,055.58	$1,647.90	$2,142.00	$4,308.90	$5,346.20	$6,459.70	$8,876.45	$23,201.96	$9,692.50	$7,394.90	$7,036.50	$3,030.60	$2,507.60	$1,001.61	$91,310.51

TABLE VIII.—GENERAL SUMMARY AND CLASSIFICATION OF COSTS.

Item.	Superintendence and labor.	Material.	Transportation.	Fuel.	Freight.	Miscellaneous.	Overhead charges.	Total.
Construction buildings	$1,660.53	$1,562.80	$ 50.00	$ 400.00	$200.00			
[illegible] tool boss, watchman	2,500.00							
[illegible]	64.00							
Equipment, handling and repairing	2,922.00	1,306.00						
General expense		[illegible]						
Superintendence	[illegible]							
Caissons	11,800.00	[illegible]	[illegible]	229.10	1,050.00		$2,701.00	$28,795.40
Crib protection work, etc.:								
[illegible]	[illegible]	585.00	14.00		80.00		169.00	1,661.72
Booms	1,253.00	780.00	16.00		120.00		248.00	2,417.00
Wing protection	3,795.00	4,850.00	100.00	60.00	500.00		1,000.00	10,305.00
Center protection	7,465.90	12,330.00	200.00	140.90	[illegible]		2,490.00	23,775.00
Concrete:								
Concrete	11,792.[illegible]	17,281.00	600.00	640.00	[illegible],580.00		4,036.00	39,929.56
Forms	2,941.00	550.00	25.00	160.00	160.00		344.00	4,180.00
Rubble stone:								
Riprap	960.00	1,127.00	300.00		200.00		332.00	2,919.09
Filling cribs	2,543.15	4,713.00			350.00		832.00	8,438.15
Removing old structure:								
Old rest piers	1,010.00							
Old center pier	600.00							
Old wing protection	576.00	1,041.29					432.00	4,518.27
Old center protection	614.98							
Old wharf	244.00							
Removing under water by diver	2,148.38	130.00			20.00		258.00	2,556.38
Cutting trestle piles inside caisson	100.00	20.00			2.76		13.75	136.51
Cleaning bottom of piers and cribs by diver	2,400.00	130.00			10.00		286.00	2,826.00
Excavation and cofferdams E. & W. Abut. and Pier No. 1	1,232.98	235.00			20.00		169.00	1,656.98
Unwatering all piers	797.92	200.00		30.00			149.00	1,676.92
Sounding	256.00	50.00					55.45	361.45
B. & B. Dept. work.								
By J. S. Metcalfe Co.—	1,411.57	200.00	23.44		70.00		159.00	1,864.01
Resupporting track								
Removing old trestle piles	100.00	20.00			2.75		13.25	136.00
Cutting foundation piles								
By G. T. Ry. B. & B. Dept.								
Resupporting track	415.20	500.00			120.00		71.00	1,106.20
Pile driving	232.00	700.00			80.00		79.00	1,191.00
Motive Power Dept						$ 329.61		329.61
Grillage beams						2,460.59		2,460.59
Dredging:								
Excavation						600.00		600.00
Removing old structures						245.00		245.00
Damage to barge						75.00		75.00
Engineering						2,705.85		2,705.85
Contractor's percentage						9,188.80		9,188.80
Total	$68,234.20	$58,668.09	$1,573.44	$2,159.40	$9,715.51	$15,604.85		$155,955.49

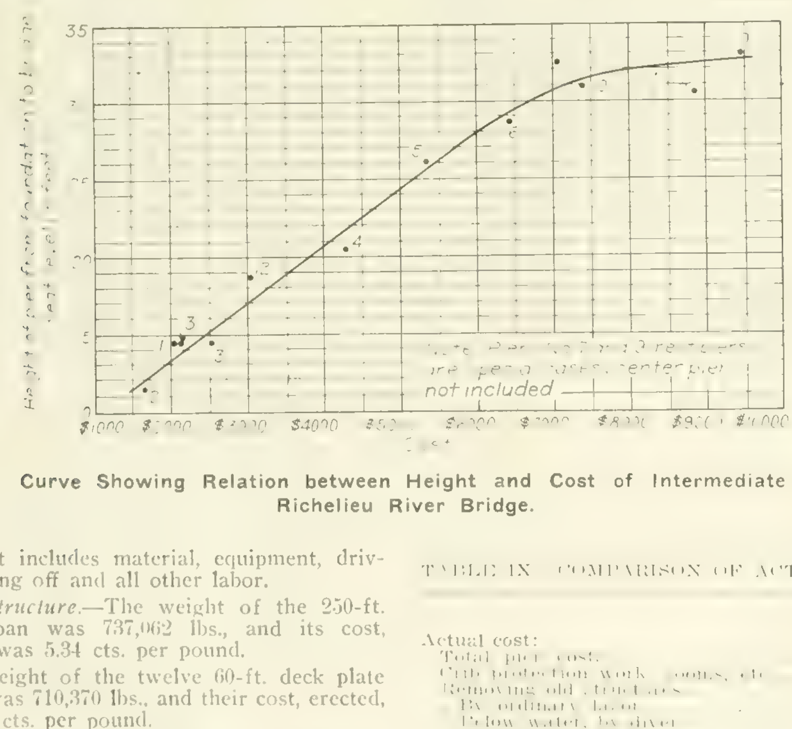


Fig. 11. Curve Showing Relation between Height and Cost of Intermediate Piers of Richelieu River Bridge.

about 14 per cent of the actual labor cost alone.

Superstructure.—The following are the percentages of the total superstructure cost ($77,877.79) of several items:

	Per cent.
Engineering, including shop and field inspection	0.78
Bridge and Building Dept.—run-outs	0.96
Bridge and Building Dept.—new floor	6.50

COMPARISON OF ACTUAL AND ESTIMATED COSTS.

Table IX gives a comparison of the actual and estimated costs of this bridge. The data are of particular interest in that they show the value of the percentage basis of payment to the contractor. It will be noted that the "extras" required in the substructure amounted to about 16 per cent of the total cost. The estimated total cost of the substructure plus the cost of extras was $157,688.39, while its actual cost was $155,955.49, a difference of $1,732.90. The estimated total cost of the superstructure was $72,615.00, while the actual cost was $77,877.79, a difference of $5,262.79.

This cost includes material, equipment, driving, cutting off and all other labor.

Superstructure.—The weight of the 250-ft. swing span was 737,062 lbs., and its cost, erected, was 5.34 cts. per pound.

The weight of the twelve 60-ft. deck plate girders was 710,370 lbs., and their cost, erected, was 3.27 cts. per pound.

The power plant and machinery cost $5,167.

PERCENTAGES OF TOTAL COST OF VARIOUS ITEMS.

Substructure.—The following are the percentages of the total substructure cost ($155,-955.49) of some of the principal items:

	Per cent
Engineering, including preliminary surveys, plans and field inspection	1.73
Bridge and Building Dept., including changing of trestle bents, re-supporting track, etc.	1.90
Sounding	[illegible]
Contractor's superintendent	1.66
General charges, including overhead charges, engineering, contractor's superintendence, and contractor's percentage	
Freight	[illegible]
Transportation	1.01

The substructure contractor's percentage was 6½ per cent of the total cost (exclusive of engineering) of the substructure, which equals

TABLE IX.—COMPARISON OF ACTUAL AND ESTIMATED COSTS OF BRIDGE.

SUBSTRUCTURE

		Total cost.	Contractor's 6½ per cent.	Grand total cost.
Actual cost:				
Total pier cost		$ 85,895.51	$5,415.00	$ 91,310.51
Crib protection work, booms, etc.		47,683.96	3,097.00	50,780.96
Removing old structures—				
By ordinary labor	$1,518.27			
Below water, by diver	2,556.38			
Below water by dredge	245.00			
Trestle piles, by diver	136.51			
		7,456.16	484.80	7,940.96
Bridge and Building Dept. work not included in pier costs—				
By J. S. Metcalfe Co., $1,864.01 — $20.00 =	$1,844.01			
By G. T. Ry. B. & B. Dept	1,106.20			
		2,950.21	192.00	3,142.21
Damage to barge		75.00		75.00
Engineering		2,705.85		2,705.85
Total actual cost		$146,766.69	$9,188.80	$155,955.49

Estimated cost:	
Estimated according to plans, Engineering Dept.	*$136,500.00
Extra wing cribs and booms: Cribs, $13,177.00; booms, $2,417.00	15,594.00
Removal of old wing cribs: $870; by diver, $300	1,170.00
Extra wall	830.86
Extra piling, 2 piers and 1 abutment	1,576.61
Extra extension of center crib work 3,760 ft. at $55.30	207.92
New crib E: 22,000 ft. at $55.30 = $1,216; removal by dredge and diver, $300	1,516.00
Removal of old wharf	293.00
Total estimated cost plus extras	$157,688.39

SUPERSTRUCTURE

Actual cost	$77,877.79
Estimated cost: Steel, $67,755; floor, $4,860	72,615.00

*Contractor's estimate, $141,000.

This difference was due mainly to freight charges which were not included in the original estimate.

PERSONNEL.

The work described in this article was in charge of H. R. Safford, chief engineer, Grand Trunk Ry.; R. Armour, masonry engineer; H. B. Stuart, structural engineer; and A. Larsen, assistant engineer, to whom we are indebted for the data upon which this article is based. The substructure was built by the J. S. Metcalfe Co., the remaining work being done by the forces of the railway company.

A Discussion of the Administrative and Design Features of Highway Bridge and Culvert Work.

The establishment of state highway commissions, in centralizing authority for state and bridge design and construction, has made possible a more thorough and systematic study of this subject than has heretofore been possible. The work is still in its infancy, but the possibilities of enormous savings are already apparent. That highway bridge work has been inefficiently carried on in the past cannot be denied. To avoid future mistakes will require that the subject be carefully studied, beginning with fundamentals, as past practice offers little of value. The following discussion is of particular value in that it discusses broadly the design, construction and administrative features of highway bridge work. The discussion is taken from a paper presented at the first Northwestern Road Congress, in Milwauwee, Wis., by A. Marston, of the Iowa State Highway Commission.

State highway commissions and engineers must take up the question of highway bridge and culvert work from many other standpoints than that of structural design. The broad general question of administration of the work must be properly solved. There are many general features of highway bridge and culvert construction which must be settled by central state authority.

The local engineer and road officer also must determine many important general features of highway bridge and culvert work.

ADMINISTRATION OF HIGHWAY BRIDGE AND CULVERT WORK.

The best system of general administration of highway bridge and culvert work is still open to discussion. In fact, a system suitable to one state may be unsatisfactory to another. However, the principles which determine the best system are general in character.

Former Practice. On one point all investigators of highway bridge and culvert work are agreed, namely, that the former practice of administration of such work throughout the United States has been extremely unsatisfactory. Literally millions of dollars have been wasted in the work.

There has been a general lack of centralization of authority and responsibility in the past. The countries, the townships, and the cities have had partial and illy-defined authority in connection with the work. The men having charge had no training whatever for the work, and with the best of intentions could not avoid waste of funds and failure to secure satisfactory structures.

The highway bridge contractor, who did the work in the majority of cases, had no assurance whatever of securing contracts by low prices and excellent work. For most of the work contracts were let and orders given privately by county supervisors and township trustees without public competition, and without definite plans and specifications. Very generally the state was divided by agreement between different contractors.

Proper Practice.—In proper highway bridge and culvert administration probably the first question to be settled is the proper unit. For small states this may perhaps properly be the state, but in the central west the county is undoubtedly the proper unit of administration. The county should employ a thoroughly competent engineer to have full charge of this work as an employee of the supervisors. Definite and complete plans and specifications should be placed on file in advance. Competitive bids should be secured for all contract work of importance. Construction should proceed under skilled inspectors. Absolutely complete records of cost should be kept.

All this work should be carried out under the general supervision of a state highway commission with real authority. Only in this way can the work of one county properly fit that of others surrounding, and only under active and competent central supervision can highway bridge and culvert work be carried on efficiently and economically.

Contract vs. Day Labor Work.—Another question of general administration is whether it is advisable to construct all or any part of the highway culverts and bridges by day labor. Without discussing this important question in detail it may be said that the experience of the Iowa State Highway Commission indicates that the best results in all ordinary culvert construction, both as to cost and quality, are being secured by day labor wherever competent foremen can be secured to take charge of the crews. By this plan the high overhead costs of private contractors are practically eliminated, and this without any corresponding overhead cost to the county, which must in any case employ a county engineer and county auditor.

On the other hand, it is altogether probable that few if any counties can afford the large amount of contractors' appurtenance or secure the services of the high grade contractors' foremen required to build large and important highway bridges. Undoubtedly such structures can best be erected by contract.

GENERAL FEATURES OF HIGHWAY BRIDGE AND CULVERT WORK.

Definitions.—Any law using the terms culvert and bridge should define them. The new Iowa road law, for example, prescribes a 20-ft. roadway for culverts and only 16-ft. for bridges, without defining these terms. Originally a culvert was a structure to carry a waterway under something else. Thus the meaning had no reference to the size of the structure.

In common parlance, however, a culvert is only a small bridge. In supervising the execution of the Iowa Road Law the State Highway Commission has defined culverts to mean all structures of 16-ft. span or less.

Standardization of Plans.—It evidently is quite important that there should be a general standardization of plans for highway bridges and culverts throughout the entire state. There is just as much need for this as for a standardization of railway plans on a particular system of railways. Otherwise it would be impossible to construct a proper system of highways covering the state.

Loadings.—The size of loads in highway traffic has been greatly increased of late years. A large proportion of the highway bridges formerly built in Iowa are at present unsafe under the heavy tractors now so extensively used in road construction. The use of auto trucks is likely to increase greatly the size of loads in the near future. All highway bridges and culverts should be designed for very heavy loadings. At least a 15-ton traction engine should be provided for.

Permanent Culverts.—Permanent culverts will ordinarily include only those made of very permanent masonry. Probably considerably more than 90 per cent of the permanent culverts in the middle west are being made of reinforced concrete, of the box type. This type is especially adapted to foundations of ordinary soils. However, the arch type of culverts, made of concrete, or stone, or brick, is especially well adapted to high embankments where the foundations are solid and are entirely safe against undermining. All permanent culverts should be fully provided with wing-walls, thoroughly protecting the embankments, and with proper cut-off walls, safely guarded against undermining.

Semi-Permanent Culverts.—Semi-permanent culverts include those made of corrugated pipe, boiler pipe, reinforced concrete pipe, and vitrified pipe. The corrugated pipe is especially well adapted to work of a temporary nature, such as replacing an old broken-down or washed-out culvert, until a permanent masonry structure can be constructed. The durability of corrugated and boiler pipe culverts is a subject which should be thoroughly investigated, together with the preparation of standard specifications for the quality of the materials. The question of whether head-walls and wing-walls should be provided for semi-permanent pipe culverts is mainly one of probable length of time which the culverts will remain in place. Manifestly, if they are to be replaced within a very few years by masonry structures, it will be good engineering to keep the cost of the temporary structures as low as possible. On the other hand, if it is expected that these pipe culverts shall remain in place 20 to 30 years, then head-walls and wing-walls should be constructed, in order to protect the embankment and the culvert against washing out and the culvert against dislodgment.

Temporary Culverts.—Temporary culverts have heretofore been built in large numbers of wood. The high cost of lumber and the short life of the structure now make these the most expensive of all types. The only proper use for temporary wooden structures in connection with culverts is to provide temporary passageway during construction.

Special Culvert Problems.—It takes a good engineer to proportion properly, locate and design a highway culvert. It is work which should never be given over to a mere instrument man. Careful study should be given in each case to the special conditions. One of the most difficult special culvert problems is the proper design of the structure where the difference in level between the upstream and downstream sides of the road is great. In such cases the water is almost certain to undermine the structure unless the latter is so designed as to break up entirely the force generated by the fall. The writer has seen places in Iowa where gulches 30 to 50 ft. in depth have been formed within a few years. He has seen individual instances where thousands of dollars have been spent in the attempt to stay the progress of such a ditch, and he has noted other cases where a township road had practically to be abandoned. In all such cases the water should be conveyed from the upper level to the lower level entirely within the culvert structure and discharged at the lower end with its velocity mainly destroyed.

Permanent Bridges.—Permanent bridges include both those built entirely of masonry and of very heavy steel constructed on masonry piers and abutments.

Reinforced concrete is the material most extensively used where the span is not too great. The different types of structures include reinforced concrete slabs, reinforced concrete girders, and reinforced concrete arches. The slabs and girders are especially well adapted to foundations on ordinary soils, and arches to rock foundations.

Arches may, of course, be built of stone and brick, as well as concrete, but these materials are not nearly so extensively used.

Permanent steel bridges should in all ordinary cases be placed on heavy permanent masonry foundations. These should be extended well below the bed of the stream and should be supported on piling in ordinary cases, unless they reach to rock. It does not pay to take chances of the undermining or settlement of an expensive masonry abutment for the sake of saving a few hundred dollars' expense for piling. High masonry abutments are especially expensive features for permanent bridges. However, a saving as much as 50 per cent of the cost of the abutments can often be made by adopting pedestal types of structures, constituting a sort of an approach span at each end whereby the earth is allowed to assume its natural slope.

The steel bridge standards of the Iowa State Highway Commission provide for: I-beams, carrying reinforced concrete floors, for spans of 16 to 32 ft., inclusive; riveted pony trusses, carrying reinforced concrete floors, for spans of 35 to 100 ft., inclusive;

and riveted through trusses, carrying reinforced concrete floors, for spans of 100 to 150 ft., inclusive. It is believed that the above steel structures, if properly maintained by painting and repairing of floors, should be practically indestructible. Above 150-ft. span, special designs will, of course, be made, and pin-connected bridges may be used.

Temporary Bridges.—The permanent bridges mentioned above are, of course, somewhat expensive to build, and, in view of the large number of highway bridges in the ordinary county, it seems manifest that a considerable number of structures must still be reconstructed of cheaper material for a number of years. For such cases, a good, heavy wooden pile bridge is still about the most satisfactory structure. Such bridges should be made heavy enough to carry 15-ton traction engines.

Sizes of Highway Bridges and Culverts.—The question of the proper size of a highway bridge, or a culvert, is an exceedingly important one. In former practice the size adopted depended upon the notions of untrained men and frequently varied 100 per cent within a distance of a mile on the same stream, with the small structure downstream. The writer has seen a $4,000 permanent bridge spanning a ravine with only a few acres drainage area, where a 24-in. pipe would have carried all the water. With a proper organization of the highway bridge and culvert work of the state, however, all this should be changed.

Under the direction of the State Highway Commission a state-wide study of the run-offs of the important streams of the state should be undertaken from their mouths to their sources, which will afford in the end reliable data for determination of the sizes of structures needed. The same wide study should be made of the size of culverts needed for different drainage areas in different portions of the state.

In such a state-wide study accurate data should be kept of every culvert and bridge which washes out. It has been stated that it would not be good engineering to build culverts so large that they would never wash out, for the reason that excessive storms come at very long intervals, which would call for structures so large that the interest on their increased cost would more than pay for all the damage and the cost of renewal. The establishment of highway commissions makes it possible, for the first time, to conduct a thorough and systematic study of this subject.

In the meantime, most of the states in which the highway bridge and culvert work has been properly organized are using empirical formulas to determine the size of waterway required. The use of such formulas, coupled with close watch of the behavior of the structures built during excessive floods, affords a good method of attacking the subject until the state-wide study above recommended is completed. In the end, however, the proportioning of culverts and highway bridges should be reduced to a question of cubic feet per second of run-off, as compared with area of waterway and allowable velocity without undermining.

GENERAL FEATURES OF HIGHWAY BRIDGE AND CULVERT WORK

The details of highway bridge and culvert work must fall mainly to the county or to the local engineer in charge.

Preliminary Work.—The entire season's work of a county, in the construction and repair of culverts and bridges, should be planned in advance, so far as practicable. During the winter months the county engineer, accompanied by the various supervisors, should go over the entire territory, selecting the structures to be built or repaired. The plans and specifications for all the work should be prepared and filed. All contract work should be advertised and let by competitive bids before the season opens.

In the preliminary work the engineer should carefully determine the drainage area and other special conditions for each structure. Excellent engineering judgment will be required in doing this work properly.

Relations of Engineer and Supervisors to Citizens Interested.—Our American system of government is such that very properly each individual citizen feels that he has a right to some direct say concerning public work in which he is immediately interested. Also, citizens living in the immediate vicinity of the proposed culverts and bridges can sometimes give valuable information concerning high-water marks and floods. For these reasons the county engineer should consult, to a reasonable extent, in advance with the citizens living in the immediate vicinity of the proposed structure. The engineer will need, however, to be a man of much good sense, and with a large amount of backbone as well as diplomacy, to avoid serious mistakes and harsh criticism. Very often, perhaps usually, the farmer who lives nearest will advise twice the size of structure which is thought necessary by those at a distance.

Plans and Specifications.—For most culvert work all that will be necessary in the way of plans is a blue print from a standard tracing, filling in properly the location, the drainage area, and perhaps special dimensions. For larger work a special plan must be prepared, either by the county engineer or by the highway commission at his request. Even for the culverts, however, the county engineer has responsible work to do in connection with the preparation of the plans. After he has determined the necessary waterway, for example, he must still settle the proportion of span to height. The writer has seen highway culverts designed by engineers where flat land adjoining would be flooded 4 ft. in depth before the culvert was filled, whereas the high-water mark was not more than a foot above the surface of the ground. The upper part of such a structure was worse than useless, for the whole grade of the road had to be raised to get over the culvert. In general, the county engineer should remember that the desired waterway must be secured with very little additional backing-up of the water on adjacent lands.

Another important matter to be considered in preparing culvert plans is the exact location and direction of the structure. The ideal water channel and roadway each should be straight, which would call for skew culverts in a large number of cases. Moreover, very frequently the culvert should not be located directly in the old channel, but a new channel should be provided on each side. Where the culvert is placed on the skew, special care should be taken to protect the embankment at the acute angle between channel and structure on each side.

In a large number of cases special construction must be undertaken to prevent danger of washing out of culverts by undermining when they discharge under a head.

Staking Out and Inspection.—The county engineer is an extremely busy man, but he should not entrust the staking out of his culverts to an inexperienced man. He will find it practically impossible to be actually on the ground during much of the culvert construction, but every structure should be inspected by him a reasonable number of times. For important bridges he should have a competent inspector constantly on the work.

Records.—The county engineer should be required by law to keep and to file detailed records of the cost of such structure, both with the county authorities and with the State Highway Commission. The county should be required by law to have a bridge book, in which an account is kept for each individual structure.

The statement has been made in this paper that, in the past, literally millions of dollars have been wasted in the highway bridge and culvert work in the United States. It is believed that this statement can be verified within a few years by actual records showing the difference in cost between work done under former practice and that done under proper practice. In Iowa it is still too early to give definite figures of saving which shall eliminate all factors except that of difference in administration. An approximate preliminary study of prices for 1912 and 1914 indicates that those for 1914 may be as much as 30 per cent lower. Part of this is due to the fall in the price of steel, although, on the other hand, the new system is not yet working at its best.

ROADS AND STREETS

Maintenance and Repair of Pennsylvania State Roads.

Specific directions concerning the maintenance and repair of state roads are given in a pamphlet of instruction recently issued by the Pennsylvania Highway Department, a part of which is given below. The standards of this department and an abstract of the specifications in use were given in ENGINEERING & CONTRACTING for August 19, 1914.

MAINTENANCE AND REPAIR OF EARTH ROADS.

The ordinary dirt or country road should be thoroughly drained in order that it may be kept dry and solid. Any spongy or loose places should be removed by placing proper sub-drainage. All depressions or ruts should be filled up immediately and an even surface maintained. Operations incident to the maintenance of an earth road may be classified under the following headings: Drainage and sub-drainage. Grades and slopes. Surface.

Drainage and Sub-Drainage.—In the matter of drainage it is necessary, first, to ascertain the nature of the soil and drainage area of the locality under consideration. This necessary information should be forwarded to the assistant engineer who will make calculations to determine the necessary waterway required. Proceed then by placing culverts of suitable size and construction, specified by the assistant engineer, at the proper locations. In the construction of drains and culverts care should be taken to avoid the use of unsatisfactory material. Where a pipe or drain can be placed having a covering of 30-in. or more, vitrified pipe should be used. Less covering will indicate that approved metal, concrete or cast iron pipes be placed. The joints of pipes must be made water tight by calking with portland cement mortar, 1 part cement and 2 parts clean sand. All pipes will be furnished the superintendent upon requisition to the department, approved by the assistant engineer of the district. Concrete has proved very satisfactory and economical in the construction of culverts and bridges and plans, specifications and material for structures of this character will be furnished upon requisition from the assistant engineer to the department. In placing pipes or drains, care also must be exercised in giving them sufficient grade, as a fall of at least ½-in. to 1 ft., where possible, is most necessary. The inlet and outlet of all drains and culverts should be kept open and clean at all times and where the soil is of a silty nature or liable to wash, paving at the ends of the pipe and part of the way up the side slopes becomes necessary. Standard plans showing form of construction at pipe ends will be furnished the superintendent. Ditches should be maintained at the sides of the road of sufficient width and depth to dispose properly of the drainage. Water should be taken off the road at intervals by cutting ditches into the fields where possible, especially on steep grades and where it is not possible to do this, the ditches should be paved with cobble or rubble.

Frequently it is found necessary to carry the water from one side of the road to the other and in many instances this has been done by raising the surface of the road transversely or by placing mounds in the road in an inverted "V" shaped form commonly called "breakers." In no case should this be resorted to. A catch basin on one side of the road with a suitable size drain crossing the highway is the correct solution to the problem and at the same time a great improvement to the road surface. Ditches should be kept open and free from obstructions at all times. Where approaches or entrances to fields, barns, buildings, property, etc., obstruct the drainage of the side ditches, the superintendent will be permitted to perform the necessary labor to place pipes through said approaches or entrances, provided the owner or owners of the property so affected furnish said pipe or pipes free of cost to the state highway department and provided, further, that, upon refusal or neglect of said owner or owners of said property to furnish the necessary pipes within fifteen days from written notice from the superintendent, the superintendent must proceed to open the ditch through the approaches or entrances. When pipes of this character are contemplated, the superintendent must specify the size or sizes thereof in his notice to the owner or owners of the property and the owner or owners of said property thereafter must maintain said pipe or pipes in good condition or they will be removed by the superintendent. Where there is no natural under-drainage in the soil and the water lies below the surface to a depth of several feet or less, it is necessary to place some form of artificial underdrainage. Porous drain tile and French drain when properly placed will give excellent results. Drain tile is manufactured in short sections with collars to place over the joints and it is laid in a ditch dug for that purpose under the surface of the road to a grade of not less than 3 ins. in 100 ft. The tile is then covered to a depth of 2 ft. with broken stone and the ditch is filled with some other porous material. A French drain is constructed by digging a trench of the same character as that used in placing drain tile and filling the same with broken stones of various sizes. In spring thaws the road will dry and become solid more quickly and on an earth road with bad underdrainage money cannot be expended to a greater advantage.

Grades and Slopes.—The grade of a road should be given special attention, as steep grades increase the cost of maintenance. It is obvious, therefore, that they should be reduced when possible and when the funds and conditions will permit. The impression must not be entertained that roads should be level and have no grade. This is entirely wrong, as the minimum grade of an earth road should not be less than 1 ft. in 125 ft. After having decided the proper grade and width of a proposed change of a highway, begin by making the cuts full width, with the sides perpendicular, taking care of the side slopes afterward. In filling up the low places in the road, care should be taken to see that the sub-surface is free from water and spongy places, and the surface is cleared of rubbish or vegetable material of any description. The filling should be deposited in layers of 1 ft. in depth the full width of the roadway and thoroughly rolled with a 10-ton roller. Any spongy or loose places showing up under such rolling should be dug out and refilled with suitable material and rolled until thoroughly compressed.

In the grading of roads it becomes necessary to provide new side slopes, sufficient to eliminate the danger of slips or slides. Different soils require different slopes. For ordinary soils, such as a mixture of sand and clay, compact clay and compact stony soil, a slope of 1 ft. base to 1 ft. perpendicular is generally given, but 1½ ft. base to 1 ft. perpendicular will give more satisfactory results. Gravel may require a greater slope. Solid rock, which does not disintegrate upon exposure to the atmosphere, might be made perpendicular, but it is better practice to give it a slope of 1 ft. base to 1 ft. perpendicular, or even 1½ ft. base to 1 ft. perpendicular, as the side being perpendicular in some degree excludes the action of the sun and air upon the road. Slopes on the south sides of highways should be greater in order to expose better the road surface to the sun's rays. Slopes, when necessary, and upon the approval of the department, may be protected by paving with cobble or rubble or by sodding. In some instances it is found necessary to build small retaining walls at the foot of the slope. When possible a ditch may be dug at the crest of the slope, parallel to the road, and the water carried to the natural drainage course without coming in contact thereon with the actual road drainage. If springs are found in the side slopes they should be drained by means of tile, French drain, or otherwise, according to the nature of the conditions and good judgment. If the whole surface of the slope appears saturated it is evident that the source of the water or spring is not isolated and can be readily drained by digging a trench in the side slopes and placing therein a line of drain tile or French drain and discharging the same in the side ditch or most convenient place. If the source is unknown careful study and experiment are necessary. Slopes of embankments are also governed by the material used. Loose earth, sand and gravel require 1½ ft. base to 1 ft. vertical; sand, 2 ft. base to 1 ft. vertical; soft, greasy clay, 3 ft. base to 1 ft. vertical, and rock, if sound, 1½ to 1 ft. vertical. Embankments of greater depth than 4 ft. should be protected by guard fence.

Surface.—The surface of the roads should be kept smooth at all times by filling up the depressions and maintaining a suitable crown. This can best be accomplished by the proper and liberal use of the road drag.

Dragging should be begun on a road as soon as the frost leaves the ground in the spring and continued throughout the season at the proper time after rains. This time is when the surface is still plastic and the mud does not adhere to the drag. All minor ruts and depressions can be evened up in this manner. In dragging a road the drag should be started along the one side near the ditch and the return trip should be made along the other side, always working toward the center. A last trip should be made up the center of the road with the cutting edges of the drag at right angles to the center of the road.

If the road has become so badly rutted that dragging will not suffice, the sides should be rooted or plowed up and combed down with a road machine or grader and, if possible, rolled with a 10-ton road roller.

Crowns should vary with the nature of the soil. For ordinary soil a rise from the side of the road to the center of ¾ in. to 1 ft. is sufficient; sandy loam, 1 in. to 1 ft.; clay, 1¼ in. to 1 ft.

A curve of suitable convexity can be obtained as follows: give ⅞ of the total rise at ¼ the width from the center of the road to the side and ⅝ of the total rise at ½ width.

When a road is composed of clayey or loamy soil, it will prove most troublesome from the point of maintenance and eventually it may become necessary to remove several inches of the top soil and replace it, if possible, with sand, good gravel or crushed stone. This can be done only upon the approval of the department.

MAINTENANCE OF BRICK ROADS.

The brick road, when approved block and material are used and constructed on a concrete foundation for some time, seldom needs attention other than sweeping of the surface, which may be done with mechanical sweepers or hand brooms; trimming berms and opening the side ditches and drains. It becomes necessary, however, when bricks are cracked or distorted to remove the same and replace with good sound specimens of approved manufacture.

Brick roads are usually constructed on a concrete, crushed stone; slag or gravel foundation. Concrete is most preferable, as in the point of maintenance it will be found that brick roads laid on anything but concrete usually present an uneven and wavy surface, which surface is quick to show the effects of wear.

When a brick road develops deep depressions not from the abrasive action of the travel on the brick, it is evident that the cause is faulty foundation and this should be attended to at once by removing the necessary amount of bricks and sub-base in order to prepare a proper foundation and, if found necessary, to place sub-drainage therein. The trench should then be filled in layers with good solid loose stone and earth thoroughly tamped until it presents an unyielding surface. This surface should then be covered with a slab of concrete, mixed in proportions of 1-3-5, at least 12 ins. in thickness with a bearing of 9 ins. on solid foundation which has not been disturbed. When concrete has set sufficiently, the brick may be replaced upon a sand cushion, allowing due amount for settlement. The surface should then be thoroughly rolled or tamped and a cement grout, mixed in the proportions of 1 part cement to 2 parts sand, swept into the joints and the work barricaded until the cement has thoroughly set and hardened.

REPAIR AND MAINTENANCE OF BITUMINOUS MACADAM ROADS.

Before going into the actual description of the proper methods of repair and maintenance of bituminous roads, we believe it will be of advantage to make some mention of the small tools, etc., used in this work.

The ordinary tools used in the preparation and construction of bituminous macadam or similar types of bituminous roads consist of ordinary picks and shovels, cutters, tampers, smoothers and special asphalt rakes. It is not necessary to dwell upon the use of ordinary picks and shovels other than to speak of their care and the tool dressing of picks and such other tools as are termed "hot tools" and used while hot. Care should be exercised in overseeing the use of these tools and to prevent their being overheated.

The only time that it will be necessary to use tampers and smoothers is when the mixture requires compression and surface dressing in places that cannot be reached by the roller and at joints and gutters prior to the rolling.

The tines of the rake should not be used in the raking of bituminous mixes wherein the large screened stones are incorporated in the mixture. To do so separates the graded proportions and forces the larger stones to the surface of the road. After the bituminous mixture is properly forked into place, a competent raker will spread the material evenly over the entire surface of the roadway by using the back of the rake. Should depressions appear after the initial compression, the tines of the rake should be used to loosen up the mixture before any additional hot stuff is placed to eliminate depressions.

In preparing hot mixtures, such as asphaltic concrete or sheet asphalt, the same care must be exercised for repair work as when it is being furnished for repaving or resurfacing.

The foundation and drainage should be perfect. If not, the bituminous surface will soon disintegrate and crumble into dirt regardless of the utmost care and supervision that might be secured in the handling of the best materials that might enter into the mixture.

Too much cannot be said relative to the constant use of thermometers the care in handling bituminous cement and mineral particles during the process of drying, heating, melting and proportioning the proper quantities of mixture. Excessively high temperature burns and destroys the bonding powers of the various materials that may enter into the construction of the mixed bituminous pavements.

Maintenance of Asphaltic Concrete Road.—When the road indicates the need of repairs in the nature of ruts, disintegrated spots or any other form of deterioration, they should be repaired by cutting out the area affected down to the sub-base, being careful to secure a vertical joint between the remaining original road surface and the material applied. If the depression was caused by faulty under-drainage, see that this is remedied before any new material is placed. Prepare this area by cleaning all dust and foreign matter from the top of the sub-base, painting the vertical joint with hot bituminous cement and deposit, spread and roll a layer of properly prepared asphaltic mixture, tamping the material especially at the joint with heated tampers, weighing 25 lbs., until thoroughly compressed. The whole surface then should be rolled thoroughly with a 10-ton power roller. Over this surface will be applied a light treatment of bituminous mixture and covered with torpedo sand, stone chips or fine gravel, which should be rolled again until compact and solid. All excess disintegrated material should be removed immediately from the road in order to maintain ease of travel.

Maintenance of Bituminous Macadam by the Penetration Method. — Penetration treated highways which are in need of repair, should be treated according to the nature of the condition.

Depressions or ruts which show up under heavy travel and not from the disintegration of the bituminous material may be repaired by cleaning thoroughly the surface affected and applying thereto sufficient clean stone to produce a surface when thoroughly rolled, with a 10-ton roller, true to lines and grades and cross-sections, applying to the surface approved bituminous material in the specified quantity.

When a highway of this character shows a disintegration of the bituminous material and the surface has become loose or ravelled, it will be necessary to remove all the material from the surface of the road down to the subbase, screening the same if possible, in order to remove the disintegrated bituminous material. Suitable stone taken therefrom should be replaced together with additional stone sufficient to bring the surface of the road true to lines, grades and cross-section. After it has been thoroughly rolled with a 10-ton roller, a specified quantity of approved bituminous material should be applied to the surface and treated according to the specifications for this class of road.

Maintenance of Surface Treated Roads Surface treated roads should be kept true to lines, grades and cross-sections by filling up the ruts and depressions as mentioned under the heading of "Maintenance of Water Bound Macadam Roads." After the surface has become compacted thoroughly, proceed by sweeping it clean of all dust, dirt, vegetable matter and all rubbish of any character, and this can best be accomplished by the liberal use of fiber brooms, mechanical sweepers or other apparatus. Great care should be exercised in doing this work so as not to displace any of the stones of the top course, thereby breaking the bond of the roadway. Designated bituminous material should then be applied in quantity and manner as stipulated in specifications covering this class of work.

MAINTENANCE AND REPAIR OF THE CONCRETE ROADS.

A few causes for the failure of wear showing on the face of the concrete road in the nature of uneven or pitted places will first be mentioned. The spots may be the result of either or both poor mixing and placing which develops the segregation of the ingredients and, once started, the affected spot grows rapidly. The abrasive action of the traffic will attack and grind out even the good concrete. Exposed faces of large pieces of aggregate sometimes become loosened and displaced by horses' calks and abrasion of travel, producing the same result. The geological difference between the aggregate and the other ingredients, especially so where hard stone such as trap rock, granite, gneiss and other stones of similar character are used, is advanced by some engineers as the cause for the separation of the mass under the action of traffic, causing irregularities in the surface. Such places should be repaired by cutting them out for a depth of at least 3 ins. and thoroughly cleaning out the section before any new material is placed. This area should be filled with cement or bituminous concrete, being careful to secure a perpendicular joint between the new material and the surrounding surface. The material should then be thoroughly rolled or tamped into place.

Places where cracks have formed should be given very close attention, since the edges of the cracks and the surface adjacent to them soon wear away. They may be filled with a bituminous filler which will protect the edges and prevent the water from seeping into the foundation. The same care should be given all expansion joints. It is readily seen that an application of some bituminous treatment to roads of this character is most essential.

MAINTENANCE OF THE WATER BOUND MACADAM ROADS.

Distributed over the state today are numerous stretches of state-built and other improved roads, which, in many instances, form a part of the present main highway system. From lack of attention on the part of the local road authorities, most of these roads, especially the water bound macadam construction, have become considerably worn and on many the macadam is entirely worn out and nothing left but the sub-base. Recent laws passed by the legislature place the care and maintenance of these roads, under certain regulations, on the state highway department. It is necessary, therefore, that those traveled most and in the worst condition should receive attention at the earliest possible date. Roads of this character should be resurfaced in the following manner:

When a road is entirely worn down to the sub-base, it should be reshaped by adding the required amount of crushed stone in the following manner after having constructed the proper shoulders or berms:

Upon the sub-base, wihch must be free from all rubbish, vegetable matter, etc., must be spread a layer of approved broken stone. The stone must be of such size that it will pass through a 2½-in. ring and not through a 1-in. ring. This layer of stone must be of such thickness that when it has been rolled thoroughly with a 10-ton roller, it will have a thickness of at least 4 ins. All depressions must be filled with stones of the same size and quality and again rolled thoroughly until the surface is true and even. The surface voids must then be filled with smaller sized stones and dry screenings and thoroughly rolled. The road should be then alternately sprinkled and rolled, applying additional screenings when necessary. This surface should be sprinkled and rolled until water flushes to the top and the passage of the roller causes no wave in the surface. The surface of the road after final rolling must be true to established lines, grades and cross-sections. If the road, in the spring following, shows the effect of a frost, it should be thoroughly rolled and covered with a light sprinkling of screenings. Where a macadam road surface presents a series of ruts and depressions not entirely worn down to the sub-base the depressions should be thoroughly cleaned out and the surface loosened or scarified and sufficient broken stone of the size and quality hereinbefore mentioned, added to crown the surface true to lines and grades in the same manner as specified above. Slight ruts or depressions can be loosened and patched in the same manner.

In patching the superintendent should use no stone 50 per cent of which shall not be less than 1½ ins. in diameter, excepting as a binder course. Small stones used in repairs of this character wear and crush into dust quickly, making it difficult to maintain an even surface. Macadam roads should be kept free from loose stone, rubbish, vegetable matter, etc., at all times, and during dry weather, where possible, the surface should be sprinkled.

Where highways are so situated as to be exposed to wind action you will find that considerable raveling will result which may be remedied temporarily by covering the surface with a layer of stone chips or coarse sand ½ in. in depth and about 10 ft. wide in the center of the road.

Where possible and practicable, piles of crushed stone should be maintained at convenient intervals along the road. All berms or shoulders should be kept free of weeds and rubbish of any character and trimmed true to grade and cross-sections. The ditches, culverts, drains, etc., should be kept clean and free from obstructions at all seasons of the year.

Guard rails should be kept in good condition and where state highway standard or similar guard fence has been placed, it should be painted with approved white paint from time to time when found necessary.

MAINTENANCE OF FLINT AND GRAVEL ROADS.

The methods of repair and maintenance of both types of road are similar in a great measure. However, the materials are dissimilar in almost every particular.

The flint of Pennsylvania, as referred to in these instructions, is found in broken formations ranging in size from ¼ in. to 5 and 6 ins. in the longest diameter.

These roads are usually constructed as follows: Upon the foundation properly prepared a layer of flint 8 ins. in depth should be deposited and thoroughly compacted. Upon this surface is spread a layer of the smaller sized material properly graded and thoroughly compacted in a manner similar to the water bound macadam construction.

In the spreading of flint it is often found convenient to deposit the load in the center of the road, spreading into place with a road machine or grader. However, good results are secured with hand rakes and care should be exercised in that all larger particles are raked out and deposited in the foundation ahead.

When a flint road has become partly worn and rutted, if there is sufficient depth of material remaining, it is necessary and essential that the surface should be broken up or scarified and reshaped, applying sufficient material to produce the proper crown and cross section.

When applying material, it is important to see that it is furnished in small sizes from 2 ins. down to ⅛ in., with sufficient clay or loam to insure proper bonding. Larger material may be used where it is possible to secure the services of a road roller. In the passing of the roller over the road, the flint becomes crushed and packed, insuring a more solid mass, alternating with sprinkling, and in this way very satisfactory roads may be made and maintained for light vehicular travel.

In the construction and repair of roads of this type, there are many important points to be considered before beginning work. The character of the material should be thoroughly studied (1) with an idea as to its adaptability for the road under consideration, (2) accessibility, and (3) nature of the traffic.

It is well in the construction of flint roads to give first thought to the materials easy of access, as long hauls materially increase cost and it may be possible to procure a material of a more permanent nature, such as crushed stone, more economically by shipping.

In considering the nature of the traffic, the width of the road is an important factor, as if much teaming is done on a narrow road all vehicles will track and considerably increase the maintenance cost.

Sufficient crown should be provided to discharge the surface water into the ditches quickly. Three-fourths of an inch to 1 ft. is most desirable for roads of this type. When a flint road begins to show signs of wear, the depressions should be cleaned out and loosened up with a pick and more material added, being careful to see that the new material forms a proper bond with the old. The use of a heavy road drag to repair the surface of a road of this type will give very excellent results. Likewise, the services of a roller, if available, will compact and lengthen the life of the same.

The gravel road is constructed in layers practically in the same manner and the several points for consideration prior to maintenance and repair apply likewise to this type of road.

The selection of the material should be given careful consideration and it may be necessary to have it screened in order to properly grade the same. All flint or gravel for the top surface should contain about 60 per cent of material ½ in. and under; no particle being larger than will pass through a 1-in. ring. It is important to see that there is not too great a percentage of sand, as an excess of sand will not permit of proper bonding, in which case it is necessary to incorporate a small amount of clay or loam.

In resurfacing both flint and gravel roads where automobile traffic is heavy, it is essential that some bituminous material be incorporated or applied to the surface, and it is also necessary in this connection that the road be as solid and compact as possible before bituminous material is applied, which necessitates the use of a road roller or a period of wear to insure a solid surface.

REPAIRING ROADS OVER TRENCHES.

In making repairs to ditches and openings such as are made by water, gas or electric light companies and other service corporations, attention is called to the cardinal points relative to the back filling, repairing foundation, preparing the joints of the surface of road and relaying surface material over the cuts. The back filling should be of suitable material that will pack and readily settle. The necessary tamping should be enforced as the back filling material is being shoveled into the ditch. Back filling should be brought to the surface of the road and the necessary time allowed to elapse in natural settlement or compression by traffic before being concreted.

When thoroughly settled, the main body of the ditch should be excavated to the depth of 1 ft. and concrete placed over the original ditch and lapping the sides and ends with the required thickness to equal that of the original foundation. When this surface has become hardened thoroughly and set, the surfacing of the road may be replaced in a manner similar to that used in the original construction. All repaired joints shall be tamped before final compression with roller. Careful supervision should be exercised in order that no depressions on raised sections exist about the joints and that the repairs in general correspond to the methods used in the original construction.

Patching Sheet Asphalt with Bituminous Concrete.

(Staff Article.)

In Benton Harbor, Mich., a town of 14,000 population, there are 13 miles of paved streets constructed largely of asphalt. Traffic there is heavy at certain seasons of the year due to the heavy fruit shipments from that point. Produce sufficient to load three ships a day is hauled over one street leading to the docks. An interesting feature connected with the wear of the pavement on this street is the much greater wear on the side of the street traveled by the loaded wagons on the way to the docks than on the side on which the empty wagons return.

For the purpose of repairing streets out of guarantee and, by arrangement with the contractors, those still under guarantee, the city acquired the following plant: A ¼-cu. yd. Smith hot mixer, a 7-ton tandem roller, a 2-bbl. portable tar tank with heating attachment, and the necessary small tools—tampers, smoothing irons, valves, shovels, etc. This equipment was used for a large amount of patching work. In operating this equipment and executing the work the following crew was used: One engineer who fired and operated the mixer, 1 kettle man who attended to the delivery of asphalt to the mixer and who handled, hoisted and emptied the tar barrels, 1 steam roller man, 1 man who loaded the mixer skip with aggregate, 2 men who wheeled mixed material from the mixer to the work, and 1 man spreading and tamping.

The method employed was to heat the old asphalt, picked up to provide a firm bond for the patch, with new aggregate and asphalt added to make the proper proportions. This material was placed, tamped and rolled as in placing an asphalt patch.

The cost of similar work with approximately the same plant in Dayton, Ohio, as determined by the Bureau of Municipal Research of that city, is given in Table I, the amounts of material used being shown in Table II. The report on the Dayton work states that the disadvantage of the machine is the impossibility of obtaining a definite proportion of aggregate and bitumen in the mixture when remelting old material. How long patches will wear made from such material is conjectural. Analysis of remixed material at first showed it to contain 17 per cent bitumen. The amount of new asphaltic cement was cut down to secure a 10 to 13 per cent mixture, but no tests were made to see if the right mixture was obtained after the completion of those given in Tables I and II.

TABLE I.—OPERATION STATISTICS AND UNIT COSTS OF HOT MIXING WORK.

Date.	Hours worked.	Batches mixed.	Time per batch, mins.	Cu. ft. per batch.	Sq. yd. laid.	Total cost.	Cost per sq. yd. laid.
Nov. 24..	2½	4	37	60	9	$ 66.00	$7.33
Nov. 25..	7	19	22	285	97	72.50	.74
Nov. 26..	1	2	30	30	[illegible]	....	.....
Dec. 1..	5	14	21	210	84	32.60	.39
Dec. 2..	9	32	17	480	194	70.35	.36
Dec. 3..	9	39	14	624	197	84.10	.43
Dec. 4..	9	40	135	640	203	88.10	.43
Dec. 5..	6	29	14	464	1,115	57.30	.51
Dec. 6..	9	34	15	544	173	69.40	.40
Dec. 8..	9	24	22	384	123	59.35	.48
Total ...	56	212	..	3,316	[illegible]	$461.90	.....
Avge.		35	15	557	181	[illegible]	$0.42

Notes.—Nov. 24, plant put in operation; entire gang cutting out. Nov. 25, cutting out 2 hours. Nov. 26, rain. Dec. 1, rain. Material, rock.

The first three days' operation were not included in total or average owing to peculiar conditions.

Dec. 5, all new material was used, the cost per square yard of surface being $1.63.

The test on new material was made under very unfavorable conditions, with an unnecessarily large force of laborers and the unit costs of laying new pavement can be materially reduced. Analysis of the mixture showed 10 to 13 per cent bitumen. The cost of laying asphalt with this machine, using old material, can be greatly reduced in the summer with a more efficiently organized force.

TABLE II.—MATERIAL USED IN REMELTING OLD ASPHALT AND WITH ALL NEW MATERIAL.

Date, unit.	Coal, tons.	Asphalt cement (Trinidad), lbs.	Asphalt cement (Standard) pav. asphalt.	Fuel oil, gals.	Crushed stone, cu. yd.	Marble dust, lbs.
Nov. 24.......	[illegible]	100		[illegible]		10
Nov. 25.......	[illegible]	800	300	40	...	90
Nov. 26.......	[illegible]	..				5
Dec. 1.......	½	..	370			100
Dec. 2.......	¾	.	[illegible]		.	200
Dec. 3.......	1¼		1,010			210
Dec. 4.......	1		740			215
Dec. 5.......	½		235		2.7	150
Dec. 6.......	½	...	300	.	2.6	200
Dec. 8..	¾	.	200		2.5	180
Total	6¾	900	3,915	48	[illegible]	1,360
Using all new material: December .	1¼	.	[illegible]		3.2	750

Notes on Sand-Clay Road Construction in Virginia.

Sand-clay roads have been extensively built by the Virginia state highway commission. In a paper before the Virginia Road Builders' Association F. D. Henly, an engineer of the commission, commented on methods of construction in vogue in that state, a part of which paper is given here.

Equipment.—An ideal equipment for one force, in my opinion, consists of 16 to 20 mules, 1 six-horse grader, 1 two-horse grader, 1 rooter plow, 1 four-horse turn plow, 2 two-horse turn plows, about 7 wheel and drag scrapers, 1 wagon with slat body to each two-horse team, 1 heavy disc harrow, 1 plank or split-log drag for every 4 or 5 miles of road, 1 spike-tooth drag and hand tools, such as picks, shovels, etc. An equipment of this size will cost about $6,000, and depreciates at the rate of about 15 per cent per year.

Two foremen and 20 to 30 laborers are usually required to operate this equipment, and the cost per month will average about $1,200 to $1,500. Where convicts are used the cost to the county is about one-half of this amount. A force of this size will build from one three miles of road per month.

Materials.—As the name implies, sand and clay are required and the coarse the sand and the more tenacious the clay the better the results. In some instances both elements are found in roadbed, and with this condition it is only necessary to mix the two and shape the road. In other instances it is necessary to haul either sand or clay onto road-bed and mix with material in subgrade, and some conditions require both sand and clay to be hauled. On sections of road where drainage has been poor for a long time it is always advisable to haul both sand and clay. Whenever a natural mixture of sand and clay in the right proportions can be found it is advisable to use this for surfacing in preference to hauling either sand or clay separately. When the last mentioned method is adopted we designate it a top soil road. This type of road is being built in a great many counties of Virginia, and is giving general satisfaction. The top soil road is a very near relative of the gravel road, and, as a matter of fact, is a gravel road in which the gravel is either small in size or percentage. As no rule has been formulated for determining the size or percentage of gravel in a gravel road, I would suggest that a more appropriate name for a top soil road would be a No. 2 gravel road.

If it is necessary to ship surfacing material I would recommend the gravel (or possibly the macadam road if in a section financially able to build and maintain the macadam), as with our conditions I do not believe the gravel road would be much more expensive than the

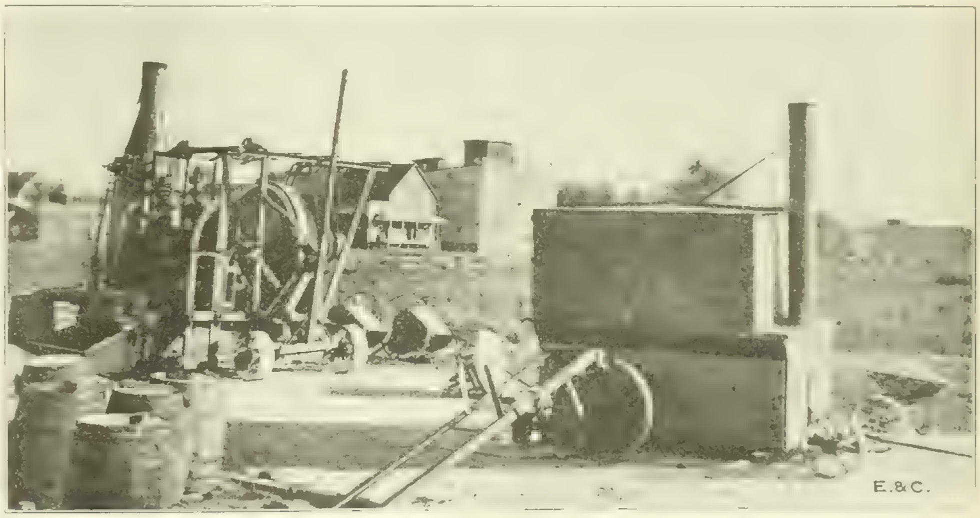


Fig. 1. Hot Mixing Plant Used for Repair Work.

sand-clay, and would be well worth the difference in cost; but on roads that do not carry a very heavy traffic, and where gravel or (with the above modification), suitable rock for macadam, can not be had locally, I would recommend the soil or sand-clay road.

The two elements, sand and clay, in the proportion of 75 to 85 per cent sand and 15 to 25 per cent clay should be thoroughly mixed until of uniform color. For the best results only sufficient clay to fill the voids between the sand grains should be used, the clay serving as a binder to prevent the sand grains from moving under traffic. On sections of road that are not exposed to the sun and wind, as in a dense woods or deep cut, or on low, boggy sections, a very small percentage of clay is required, as water will partially fill the voids between the sand grains. On heavy grades, especially when exposed to the sun and wind, the percentage of clay should be greater than with any other condition.

Placing and Finishing.—The sand-clay mixture should be 8 to 12 ins. in thickness at the center and run to a feather edge at ditch line, and 20 to 26 ft. in width, depending on the traffic and local conditions. In roads built as above described the crown is made altogether of the surfacing material.

Unless the surfacing material has been thoroughly mixed before being dumped on the road, the sand and clay should be spread before mixing to roughly conform to the proposed finished cross-section, which should be not less than three-fourths and not more than 1 in. to the foot, with side ditches somewhat deeper than for macadam.

The sand-clay or soil road should not be considered completed the first time the surface is put in good shape, as bad sections often develop after the surface has been in shape for as long a period as one year. The real tests for the sand-clay road are the protracted droughts and the long-continued, slow, dribbling rains, followed by freezing. The last mentioned condition is very trying on sand-clay and soil roads and the best constructed road will very probably require some attention after such weather. If the road has been properly constructed, a split-log drag used at the right time will put the surface in good shape.

The use of a roller on this class of road is not necessary, and may be a decided disadvantage. The surface should, like a wound in the flesh, heal from the bottom. It is not practical to lay surfacing in more than two courses, and use of the roller case-hardens the surface, and this case-hardened surface will often carry the traffic until the first protracted wet or dry spell of weather, when it will cut through or break up. While this probably would not be a permanent injury to the road it would certainly be very inconvenient to the users of the same.

Cost.—On account of the great variance in local conditions, it is impossible to set one price for sand-clay work. However, I would say, that under the most favorable conditions a fair sand-clay road may be built for about $400 per mile, and when built under adverse conditions may cost as much as $1,600 per mile. About 270 miles of sand-clay road were built during the fiscal year ending Sept. 30, 1913, by the Virginia highway commission, at an average cost of about $885 per mile.

Poles Used in Canada.—In 1913, according to a report by the Forestry Bureau of Canada a total of 534,592 poles were purchased in that country by railway, telephone and telegraph companies. The total number purchased in 1912 was 608,556. In 1913, 49.4 per cent of the poles purchased were of white cedar; 27.2 per cent, red cedar; 21.6 per cent, tamarack; while, 1 per cent or less of each were of spruce, jack pine, balsam fir and white pine.

The Life of English Roads as Determined from Traffic Statistics.

In England statistics of the tonnage of traffic passing over various roads have been kept for a number of years. Information concerning this traffic collected by W. de H. Washington and published in the last annual report of the New York State Highway Commission is given here.

RESULTS OF EXPERIENCE IN LIVERPOOL

J. A. Brodie of Liverpool has very carefully recorded his experience there for ten years with various types of roads. In some ways his data are among the most important road statistics yet gathered, and are given in Table I.

With an actual traffic of 120,000 tons per yard width per year, he gives the life of waterbound macadam at one year; tar sprayed at two years; and pitch macadam at eleven years.

Engineer Brodie also measured the wear of various experimental surfaces by a wire running over pulleys at both sides of the road, stretched across and kept at a constant curve by a standard weight. The fall in the height of the road was originally measured by a steel rule, but this method is now improved upon.

OTHER TESTS.

On the Norfolk macadam roads, where the total weight of traffic is about 9,023 tons per day, or 3,300,000 per year, the average cost of maintenance per ton mile of traffic was .16 pence or .32 of a cent.

On the Warwickshire macadam roads, with about the same total traffic, the cost of maintenance per ton mile of traffic was .27 pence or .54 of a cent.

TABLE I.—TRAFFIC STATISTICS FROM LIVERPOOL, ENGLAND.

Type of road.	Tons traffic per yard width per annum.	Life, in years	Life tonnage per yard width	Cost a square yard.	Total maintenance square yard.	Ton miles a yard width a cent.	Cost of maintenance for a 100-ton mile of traffic.
6-in. blocks	521,000	18	9,432,000	$2.40	$0.175	17	$0.06
Soft wood	204,000	18	3,672,000	2.04	.15	8	.13
Pitch macadam	120,000	11	1,320,000	.72	.066	10	.095
Tar-sprayed macadam	120,000	2	240,000	.24	.12	5.7	.18
Water-bound macadam	120,000	1	120,000	.18	.18	3.8	.264

Costs.—His experience gives the cost per square yard of surface, considering waterbound macadam as already laid, at $.24 for tar-sprayed macadam; $.72 for pitch macadam; $2.04 for soft wood blocks; $3.24 for hard wood blocks, and $2.40 for 6-in. granite blocks.

Life Tonnage.—His records show the total tonnage which can be borne by a yard width of surface before wearing out, as 120,000 tons for waterbound macadam; 240,000 tons for tar-sprayed macadam; 1,320,000 tons for pitch macadam; 3,672,000 tons for soft wood blocks; 2,754,000 tons for hard wood blocks, and 9,452,000 tons for 6-in. granite blocks.

Annual Cost.—He gives the annual cost per square yard, including capital charges, as $.18 for waterbound macadam; $.12 for tar-sprayed macadam; $.066 for pitch macadam; $.15 for soft wood blocks; $.25 for hard wood blocks, and $.175 for granite 6-in. blocks.

Cost Per Ton Mile.—The number of ton-miles of traffic per yard width which 1 ct. of total expenditure yields is 3.8 for waterbound macadam; 5.7 for tar-sprayed macadam; 10.3 for pitch macadam; 7.8 for soft wood blocks; 3.7 for hard wood blocks, and 17 for granite 6-in. blocks.

IN IRELAND.

Engineer Gullan of Belfast gives as the results from different types of roads throughout 20 years, all on existing foundations, the figures shown in Table II.

Engineer Hatfield of Sheffield gives the results of two tar macadam roads as compared with two paved roads as shown in Table III.

TABLE II TRAFFIC STATISTICS FROM BELFAST, IRELAND.

	Life, years.	First cost per square yard	Capital charge per year.	Maintenance per square yard per year.	Renewal cost per square yard per year.	Total cost per square yard per year.	Total cost per square yard twenty years.
Water-bound macadam	1-4	....	.....	$0.04-$0.24	....	$0.06-$0.42	$1.20-$8.40
Tar macadam	6	$0.48-$1.08	.....	.02- .06	$0.03-$0.06	.09- .25	1.84- 4.94
Stone blocks	15	1.80	.....	.04	.03	.22	4.40
Asphalt	10	2.16	$0.26	.12	Out of cap. charge.	.42	8.40
Hard wood	1[illegible]	2.88	.25	.06	Out of cap. charge.	.39	7.80
Soft wood	12	2.16	.22	.03	Out of cap. charge.	.39	7.86

The cost per 100-ton miles of traffic is $.264 for waterbound macadam; $.18 for tar-sprayed macadam; $.096 for pitch macadam; $.128 for soft wood blocks; $.272 for hard wood blocks, and $.06 for 6-in. granite blocks.

TABLE III.—TRAFFIC STATISTICS FROM SHEFFIELD, ENGLAND.

	Tons traffic per yard width per annum.	Cost per yard	Probable total life.	Est. cost per yard per year
Six-inch granite blocks on concrete	73,180	$3.66	27	$0.15
Redwood blocks on concrete	11,983	2.40	14	.23
Tar macadam, No. 1	[illegible]	.66	21	.12
Tar macadam, No. 2	35,939	.1[illegible]	21	.06

Some Requirements of Road Specifications with Special Reference to Rolling.

Constructive criticism of road specifications is of great value. In a paper before the American Association for the Advancement of Science, W. W. Crosby commented upon some of the failings of modern road specifications and his paper is given here.

Before proceeding to details, it seems necessary, for the sake of clearness, to state certain general principles in regard to specifications. Recognition of them may be more general than the writer is aware, but it seems to him their emphasis at the expense of reiteration is demanded by the apparent frequency of their neglect.

In the first place, while it may be necessary sometimes to restrict in details the methods to be followed, generally it will be found more satisfactory to specify the results to be obtained rather than one exact method for reaching the result. Elasticity for meeting variations in conditions encountered will then not be wanting. This is especially true as regards rolling. Also, where necessary the methods of producing the result may be limited by specific description, but this should be done only when unavoidable for the insurance of proper results and for preventing the production of a result which will be offered for acceptance as "just as good." Furthermore, for economic reasons as much elasticity in the provisions for limits, in the descriptions of the machinery or tools allowed for use, should be given as is practicable. Finally, the specification of the result to be secured should be absolutely definite, clear, and as brief as may be consistent. The specification should so describe the product that no more room for argument as to the fulfillment of the specification will exist than will be occupied by a few questions whose answers can and must be determined by scientific methods, such as physical or chemical analyses and arithmetical calculations or measurements.

With specifications drawn on the basis of the foregoing, most of the difficulties complained of in many specifications would be obviated. The ever-recurrent question of the interpretation of the clauses would have eliminated from it such extremely annoying and indefinite factors as the meaning of many phrases, the permitting of substitute methods, the personal equation in interpretations under varied conditions and be resolved into one simple one—that of what will be the maximum allowance of variation from the specified results for the actual result secured to be acceptable.

Assuming now that the specifications in regard to the road crust have been drawn in all other points on the basis outlined, the clauses regarding the rolling will be considered.

As a horrible example of *what not to do,* the following may be cited:

"Immediately after the application of the refined tar, a layer of dry No. 1 broken stone, not to exceed three-eighths (⅜) of an inch in thickness, shall be spread and broomed, as directed by the engineer, over the surface of the refined tar, and shall be at once rolled as directed by the engineer with a roller weighing between eight (8) and fifteen (15) tons."

To the speaker this is indeed a "Monstrum horrendum, informe, ingens, cui eunen ademptum." And yet it is not a "creature of the imagination" of the speaker, but really a verbatim extract from a set of specifications recently adopted as "standard" (save the mark) by a national society of considerable pretensions. It is not the intention of the speaker to digress into a discussion of either the syntax of the clause or the propriety of the expression "as directed by the engineer." Familiarity of his audience with the discussion on the latter will be assumed. He merely wishes to illustrate by the quotations his remarks here. Take the foregoing as the "before" picture. Let us see how an "after" looks:

Immediately after the application of the refined tar dry No. 1 broken stone shall be spread and broomed into an even layer that will not be over three-eighths (⅜) of an inch in thickness over the surface of the refined tar, and this layer shall be at once rolled with a roller of not less than eight (8) nor more than fifteen (15) tons in weight. The rolling shall be continued until the surface shall be compact and even.

Which form of the clause is more definitely interpreted? Under which are opportunities for the display of idiosyncrasies by engineer and contractor greater? Under which form is it more easily possible to estimate the cost of doing the work and there to name a definite price for it?

For further illustration of the theme, let us consider for a moment a clause from the printed specifications for macadam used by a large English city:

"The second coat (of metal) shall then be uniformly applied and the whole surface shall receive a thin coating of fine granite chippings or other binding material, approved by the city surveyor, not exceeding one inch in size, which shall be well rolled again three times."

Now regardless of whether this city surveyor, who it may be supposed is at least not expected to be of extraordinarily small size in any particular, would lose his job and be supplanted (by what) if the commas were lost, is not the inelasticity or unnecessary and inconvenient rigidity of the clause as it stands apparent?

Again, from a large city's (in this country) specifications:

"When the grading has been completed the entire surface shall be rolled not less than three times with a steam roller weighing not less than 350 pounds per lineal inch of roller, or, if in the opinion of the engineer the use of a steam roller is impracticable he may permit the use of a horse-roller weighing not less than 250 pounds per inch."

Will any two contractors estimate the cost of the rolling under this clause at the same figures?

One more illustration, this time from specifications of one of the so-called progressive cities of the country. (It now has a city manager who seems to be doing good work, and perhaps it should be presumed that the city's specifications have since been brought beyond criticism. Nevertheless, the counterpart of the clause may also be found elsewhere.)

"The surface must be made perfectly even by heated smoothers and be rolled with a steam roller weighing not less than 250 lbs. to the inch run; the rolling must be continued for not less than five (5) hours for each 1,000 square yards of surface."

Does this mean that the roller must be constantly moving, and, if so, at what rate of speed? Will rolling part of the 1,000 sq. yds. of surface once and the rest of it as many more times as may be necessary to occupy the period fulfill the specification?

Various other questions are easily framed and it is necessary to consider the possibilities for all such questions, as have been suggested, because of the conditions usually present in connection with public work—the great bulk of highway work. It is futile to argue that the clauses as they stand should be readily understood to mean this or that by all competent highway engineers and contractors, and that the questions above suggested are rarely captious criticisms. Contractors for public work of this character are not yet usually selected and invited to bid. The bidding is open to all—experienced or inexperienced, intelligent or stupid, straight or crooked, who can fulfill certain (generally) financial requirements, and the difficulties in the way of rejecting the low bid are well known. Many green contractors have to be educated, many would-be sharp ones held in the narrow path, and competition in bidding must be encouraged. Again this supply of inspectors, as well as of contractors, must be examined through the same glasses, and provisions be made for using that material also to the best advantage considering its likely qualities. Friction as well as lawsuits must be avoided.

As far as possible, there should be left no opportunity for such questions as are indicated above, and the specifications may, to this end, even express, to a certain extent and as before suggested, definite methods to be followed in the productions of results, as well as describe exactly the results themselves.

Having criticised what has been done, it is but fair that the writer should specifically propose something to be done in this line and illustrate his proposition also. To attempt to cover here every case of rolling the road crust would be probably as unnecessary as it would be uninteresting to the audience. Hence the speaker will limit his suggestions to what he considers a few, but perhaps typical, exhibit.

He thinks the "rolling" clauses should be as follows:

WATER-BOUND MACADAM.

Sub-Grade.

"The portion of the road-bed prepared for the crust shall be ——— feet wide, be brought to the grades and cross-sections shown on the plans, and be rolled with a self-propelled roller until firm and solid. All depressions that may appear during the rolling shall be filled with approved earth and rerolled until a firm, even surface with a proper grade and cross-section shall be obtained."

First Course.

"After the layer of broken stone, slag, gravel, shells, or other metal for the first course shall have been spread uniformly to the proper cross-section it shall be rolled with a three-wheeled, self-propelled roller, weighing not less than ten tons, until the layer shall be compacted to form a firm, even surface. Should any serious difficulty in compacting such metal as certain granitic rocks be experienced while rolling, lightly spreading the layer with sand or other material or sprinkling it with water, all as may be approved by the engineer, shall be employed. The rolling shall begin at the sides and work toward the center of the roadway, thoroughly covering the space with the rear wheels of the roller. The rolling shall be discontinued before the pieces of metal lose their angular character.

"Should any unevenness or depressions appear during or after the rolling of the first course, they shall be remedied with fresh metal of the same kind as previously used, and the rolling shall thereupon be resumed and continued until a firm, uniform and even surface shall be obtained. Should sub-grade material appear at any time to have churned up into or mixed with the metal of the first course the contractor shall, at his own expense and without extra compensation, dig out and remove the mixture of sub-grade material and metal and replace the same with clean, fresh metal of the same kind as previously used in this course, thoroughly rerolling and compacting the fresh metal so that the first course shall finally be firm, uniform and even on its surface with the latter at the proper grades and cross-sections."

Second Course.

"After the metal for the second course shall have been spread to the proper thickness and cross-sections, it shall be rolled as hereinbefore provided under the head of "First Course," except that water, in connection with the rolling, shall be used as follows: When the rolling shall have been carried on to the point where the metal of the second course will not push or "weave" ahead of the roller and any depressions or unevennesses have been properly remedied as provided, the rolling shall be interrupted and a thin layer of sand, screenings, or other approved binding material shall be evenly spread over the surface of the second course metal with as little disturbance of the latter as possible. The quantity of fine material so applied shall be just sufficient to cover the metal and care shall be exercised to avoid its use in excess. Water shall then be sprinkled on the roadway surface and the rolling at the same time resumed, the quantity of water used being such as will prevent the fine material from sticking to the wheels of the roller. The combined watering and rolling shall be continued until the voids of the metal shall become so filled with the finer particles as to result in a wave of water being pushed along the roadway surface ahead of the roller wheel. The watering and rolling shall then be discontinued until the macadam shall have dried out. If then the metal shall begin to loosen and to appear on the roadway surface, or if the voids in the metal shall appear to be not properly filled, the watering and rolling shall be resumed with the application of only as much additional fine material as may be necessary. Any depressions or unevennesses appearing during the above operations shall be remedied by the contractor, as hereinbefore provided, and when completed the macadam shall be uniform, firm, compact and of at least the thickness required and shall have an even surface, nowhere departing by more than one inch from the grades and cross-sections shown on the plans."

BRICK PAVEMENTS

"The portion of the road-bed prepared for the crust shall be ——— feet wide, be brought to the grades and cross-sections shown on the plans, and be rolled with a self-propelled roller until firm and solid. All depressions that may appear during the rolling shall be filled with approved earth and rerolled until a firm, even surface with a proper grade and cross-section shall be obtained."

Cushion.

"After the material for the cushion shall have been evenly distributed to the proper thickness and spread to the proper grades and cross-section it shall be thoroughly, firmly and evenly compacted by rolling. The roller shall

Fig. 1. Half Section of One-Course Alley Pavement.

weigh not less than ten (10) pounds per inch in length, should approximate twenty-four (24) inches in diameter, and shall not be more than thirty (30) inches in length. After rolling until the cushion shall be properly compacted as above prescribed, the template shall be applied, and if the surface of the cushion shall be found to be not uniformly parallel to the surface re-

Fig. 3. Street Paving Showing Lower Course and Reinforcing in Place.

quired for the finished pavement nor at the proper grades, the defects shall be remedied and the rolling shall then be repeated."

Rolling the Brick.

"After the bricks for the pavement shall have been laid as provided and the surface of the roadway swept clean, the brick shall be rolled

Fig. 5. Method of Placing Expansion Joint.

with a self-propelled tandem roller, having a weight of not less than three (3) nor of more than five (5) tons. The first passage of the roller over the brick shall be at a slow pace, shall be begun at the curb, and the rolling shall, by means of overlapping passages parallel to the curb, proceed to the center of the street. The rolling shall then proceed from the other curb to the center in the same manner. The roadway shall thereafter be rolled transversely by parallel overlapping passages from curb to curb at each angle of forty-five (45) degrees, with the curbs, and finally by passages of the roller parallel to the curbs as at first above described. When the rolling shall have been completed, as above described, the surface shall be even at the proper grades and cross-sections. Any depression exceeding one-quarter (¼) of an inch in depth under a ten (10) foot straight edge laid on the surface of the brick parallel with the curb shall be properly remedied by

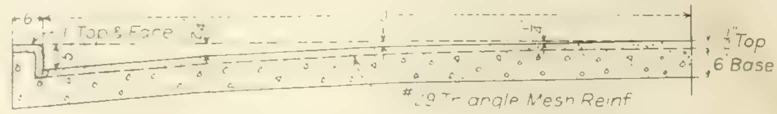


Fig. 2. Half Section of Two-Course Concrete Pavement.

the contractor, at his own expense and without extra compensation."

Concrete Street and Alley Paving and Curb Construction in Gary, Indiana.

To the Editors:

In the current number of Engineering and Contracting (Dec. 9) I note an article on the construction of concrete curbs integral with concrete pavements. Having had some experience and some trouble with this work, and having overcome our troubles, I submit the enclosed sketches and photographs with hope that some one may be saved the trouble we had.

During the past season we have constructed by contract 4,850 lin. ft. of concrete pavements in alleys and 3,500 sq. yds. of concrete pavement (¼ mile) on street improvement. On all this work the curbs were built as part of the pavement and paid for as such. A comparison of cost to abutting property for concrete pavements and brick pavements in alley improvements shows a saving in favor of the concrete of $0.25 per lineal foot, under very similar conditions as to competition in bidding and work to be done.

In construction of these pavements grade stakes were set for top of curb and reference line established for back of curbs. From these the contractor set form boards for back of curbs. Templates were used for striking off subgrade and top of concrete surface—the back form board was used as a runway for the templates. After striking off the surface of the pavement the face board for the curbs was set and clamped to the back board. The face of this board was plastered with a 1-in. facing of 1:1 mortar using Portland cement and torpedo sand, and after placing and tamping the concrete backing, a 1-in. top of same mortar was applied. The finish of the curb was then the same as for ordinary curb construction.

In the alley improvements a one-course pavement was used. Concrete was a 1:2:3 mixture composed of washed and screened gravel and torpedo sand for the mineral aggregate and Universal Portland Cement, used almost exclusively. On one small contract, crushed limestone, screenings and Marquette Portland cement was used and at present seems to be equally as good as the balance. The bid prices on the several contracts varied from $1.32 per square yard to $1.40 per square yard. Payments were to be made in bonds worth about 93 cts. on the dollar.

On the street work a two-course reinforced section was used. The lower courses in this work was a 1:2¼:4 mix, and top or wearing surface a 1:2 mix, using equal parts of sand and pea gravel for the fine aggregate. The pea gravel was obtained by screening a portion of the small material out of the gravel supplied for the lower course.

We experimented on the finish for all pavements and adopted a finish obtained by working the surface with a wooden float by hand and then brooming the surface lightly to give uniform appearance. Expansion joints were placed at intervals varying from 25 to 35 feet apart. All joints were made by using Kahn armor plate and Carey elastite.

Local conditions make it unnecessary to provide for subdrainage as all our work is on sand sub-grade. The sub-grade was compacted by sprinkling ahead of the concrete work and this watering served to prevent a drying out of the concrete. Sprinkling was also resorted to in curing the pavement in its early stage. A sand cover was placed on the surface during the night by the watchman and

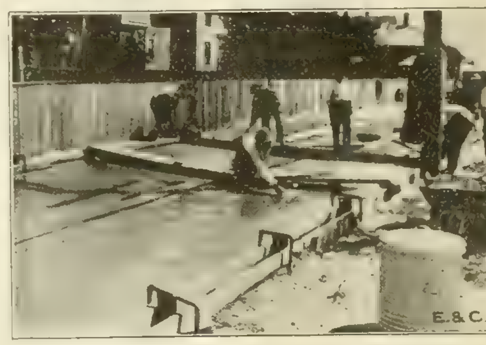


Fig. 4. Method of Constructing Curb.

Note. Usually the finisher was not so far ahead of the curb work or so near the newly struck concrete.

kept moist by him for ten days. Plans and specifications for this work were prepared by the engineer's office, and construction was under our supervision.

I trust that this information will help some one in the construction of concrete pavements

Fig. 6. Method of Floating the Surface at Intersection.

and will be glad to explain further any point I have not made clear.

Very truly yours,
W. P. Cottingham,
Assistant City Engineer.

Gary, Ind., Dec. 10, 1914.

Quantity of Water Required for Street Sprinkling.—The last annual report of the Street Commissioner of Newport, R. I., gives the quantities of water required for street sprinkling as shown in Table I.

TABLE I.—GALLONS OF WATER USED AND AREA SPRINKLED BY YEARS IN NEWPORT, R. I.

		Gallons of water used			Mileage of streets watered			Sq. yds. of street watered		
Year.	[illegible]	Fresh.	Salt	Total	Fresh.	Salt	Total.	Fresh.	Salt.	Total.
1907	1973-2	29,622,914	2,245,075	[illegible]	14,918.61	1,812.24	16,730.85	202,983,848.35	30,867,466.84	233,851,315.19
1908	1977-2	30,353,738	1,861,038	32,214,776	15,007.45	1,768.34	16,775.79	204,414,971.15	30,205,186.88	234,620,158.03
1909	1771-3	26,931,452	1,661,662	28,593,114	13,352.02	1,539.13	14,891.15	182,801,035.46	26,490,043.34	209,291,078.80
1910	1526-6	24,412,998	2,221,574	26,634,572	10,943.16	1,393.47	12,336.63	156,700,279.00	31,087,143.00	187,787,422.00
1911	1469-5	17,868,183	1,647,200	19,515,383	9,336.57	978.39	10,314.96	135,028,329.00	18,377,241.00	153,405,570.00
1912	1674-1	20,084,089	1,865,650	21,949,739	11,251.24	1,067.77	12,319.01	163,772,122.00	20,480,371.00	184,252,493.00
1913	1446-1	17,881,771	1,477,500	19,359,271	9,563.45	678.33	10,241.78	142,389,874.00	13,846,873.00	156,236,747.00

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design and to Methods and Cost of Construction

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Traffic and Road Maintenance.

It may perhaps be thought that the discussion of road maintenance is somewhat beside the point since, for the most part, the primary consideration in this country is that of constructing rather than maintaining roads. On the contrary, maintenance when considered with traffic is a matter of primary importance in road construction. It is the factor upon which hinges the selection of the type of pavement, a matter most intimately related to that of first cost.

In the article on another page of this issue is given a complete discussion of this subject based on actual experience and data. Some of the conclusions reached in this article are worthy of emphasis. It is stated that economical maintenance is only possible on properly constructed roads. The reconstruction of a poorly built road is not maintenance, it is merely the correction of a blunder. Where motor traffic is heavy a water-bound macadam road will be destroyed within a month unless treated with some sort of binder. With certain volume and weight of traffic granite block on a concrete base is believed to be the most economical type of pavement. In round numbers the cost of road maintenance will approximate a cent a vehicle per mile each year. With a certain volume of traffic the use of a concrete base, or in some places a concrete road, becomes economical. In England a rough estimate of the increased cost of maintenance on water-bound macadam roads due to each motor vehicle traveling the road per mile for each mile traveled is as follows: Motor bus of 6 tons and load, 4 cts. a mile; small touring car, ¼ ct. a mile; large touring car, 1 to 2 cts. a mile; and a heavy horse-drawn vehicle with a 3-ton load, 1 to 2 cts. a mile.

It would perhaps be well to call attention to distinction between the two uses of the term "ton-mile." When referring to the cost of hauling "ton-miles" are obtained by multiplying the weight of the load by the distance hauled. This product divided into the total cost of hauling the load the whole distance is "cost per ton-mile." With reference to cost of maintenance the "ton-mile cost" of maintenance is obtained by dividing the product of the tonnage passing and the distance into the total cost of maintaining the length of road. It is readily seen that confusion may exist unless this distinction is carefully borne in mind.

As to the actual cost of maintenance on country roads paved with various types of macadam it will be noted that French roads approximate $770 a mile a year, English roads $1,100 a mile a year, and Massachusetts roads $850 a mile each year. It must be borne in mind, however, that these figures include the cost of resurfacing; the life of the road thus being, in a measure, unlimited.

Accurate data concerning other types of pavements used on country roads considered in connection with traffic do not exist. It may be stated, however. that the resurfacing of concrete roads, if necessary after a period of years, whether with brick, asphalt or additional concrete must be considered as maintenance. Also the interest on the additional cost of the investment, if any, must be included.

The important relation between the first cost of road improvement and the cost of maintenance is shown indirectly by the fact that from 60 per cent of the total cost in the case of gravel, to 85 per cent in the case of brick roads is expended on the paved surface. The relative costs of maintaining various types of surface considered in connection with interest charges is undoubtedly the economic basis upon which pavement types should be selected.

Some Facts Emphasized by the Edison Fire.

The destructive fire which occurred on Dec. 9, 1914, at the plant of Thos. A. Edison, Inc., West Orange, N. J., again emphasized some essential requirements of so-called fireproof buildings. Although this fire may well be carefully studied by engineers and architects we do not believe that the findings of investigating bodies will disclose facts which are not already known. They will, however, undoubtedly disclose facts which continue to be disregarded by some engineers and architects. In the area affected by the fire two types of buildings were in general use—brick buildings with steel beams and roof trusses, and those constructed of reinforced concrete. The building in which the fire started was used for film testing, and it was a one-story structure with sheet metal covering. The buildings were closely spaced, which made it impossible to stop the progress of the fire with the inadequate facilities at hand. As a result of this disaster the brick buildings and their contents were completely destroyed, while the contents of the reinforced concrete ones were destroyed, and in many cases the columns either were destroyed or were badly damaged. In general, the reinforced concrete floors and walls effectively resisted the fire, the damage to them being due mainly to the failure of the columns.

Even though the contents of the buildings were highly inflammable, and the fire hazard was known to be great, the reinforced concrete buildings were fitted with wooden sash and plain glass windows. Moreover, there was a decided lack of fire walls, the buildings generally being open and with few partitions. There were also open elevator shafts and some unprotected stairs. Finally, the buildings were not equipped with automatic sprinkler systems, even though the water supply for fire-fighting purposes was inadequate.

It has been demonstrated by many fires that no building which contains wooden frames and plain glass windows should be classed as a fireproof structure, as it neither can retard a fire originating from within nor can it offer protection in case of a fire in adjacent buildings. The automatic sprinkler system has also proved its worth as an effective means of protecting the contents of a building. Furthermore, the value of fire walls and of enclosed shafts for elevator openings is generally recognized. In view of these facts any engineer or architect is inviting severe and merited criticism if he does not strongly impress upon the prospective builder the need of recognized fire protection measures for buildings of this character.

As has been pointed out in previous issues of Engineering and Contracting the so-called rodded columns, without spiral reinforcement or efficient hooping, should not be classed as a reinforced concrete member. The columns in the reinforced concrete building affected by this fire were of the rodded type, as is evidenced by photographs of numerous columns which failed. The vertical rods simply bent out of line as soon as the outside layer of concrete fell away and thus left nothing to resist the horizontal shear—the characteristic failures being along diagonal lines. Although the water thrown upon these columns undoubtedly contributed to their failure, total failure would not have occurred in at least some of them if horizontal shear reinforcement had been provided. It should be remembered by designers that the columns are the vulnerable points in destructive fires, and that the utmost care should be exercised in their design and construction.

The prominence given to this fire may well be taken advantage of by engineers and architects in emphasizing the lessons pointed out by it, even though these lessons are not new and should need no reiteration.

Inscriptions on Engineering Structures.

It has long been the custom to place inscriptions on engineering structures, either on corner stones, on stones over main entrances to buildings, or on metal plates placed on the end posts of bridges or on the inside walls of buildings. The custom has not been general by any means, but it should become so.

There are substantial reasons why this custom should become general. In the first place there is the item of public interest to be considered, and in the second place the fixing of individual responsibility. The official name of every important engineering structure should be permanently inscribed at some conspicuous point on the structure's exterior. This is for the information of citizens, particularly of visitors who have occasion to inspect the public works of a city. Moreover, this appears to be a suitable and efficacious method of calling the attention of the public to the work of engineers, a consideration of great importance to both engineers and laymen. To illustrate our thought in this matter, let us take the case of a modern rapid sand water filtration plant. The official name of the plant should be cut into the stone over the entrance to the head house or administration building. Inside the same structure a metal plate bearing appropriate data, in raised letters, should be prominently displayed on a wall. The data given should comprise the dates significant in the history of the inception and construction of the plant. The names of the city officials who had to do with the plant's installation should also be given, as should the names of consulting engineers, resident engineers, architects and contractors engaged upon the design and construction of the plant.

The fixing of responsibility for the structure in this manner is sure to have a salutary effect upon its serviceableness. What is probably more to the point, in most cases, it will give credit for the work to those to whom credit is due. The public is quick to forget the names of its servants unless they are kept before it in some manner.

Incidental to the foregoing matter we take this opportunity to record a protest against the use of the letter v for the letter u in such inscriptions. The architect frequently collaborates with the engineer in the design of engineering structures of which buildings form a part. He brings his entire "box of tricks" along with him, including the alphabet from which the perfectly good letter u is missing. The origin of this custom is doubtless familiar to our readers; we are not concerned with its origin, however, but with its finish. It is a custom which has nothing to commend it. At best it is only a superficial indication of a certain form of culture. It causes momentary confusion to the reader, and if he be a plain man, and most readers are plain men, it causes him a certain feeling of irritation.

WATER WORKS

The Protection of Riveted Steel Pipe.

Contributed by Leonard Metcalf, of Metcalf and Eddy, Consulting Engineers, Boston and Chicago.

During a recent engagement in California, the writer had an opportunity of obtaining some information regarding the experience of the Spring Valley Water Co., of San Francisco, with pipe coatings, which is of particular value on account of the length of the period these coatings have been in satisfactory service. The work has been under Mr. Hermann Schussler as chief engineer for many years. The available information on the preservation of riveted pipe is so limited and the subject is so much more important in the case of a pipe of thin steel than where a thick cast-iron shell is used, that the writer believes the readers of Engineering and Contracting will be interested in what he learned. This is still more the case because the Californian experience shows that under the soil and climatic conditions of that section of the country a coating has given a good account of itself which was unlike those of several pipe lines recently constructed in the Eastern states.

In the case of the Spring Valley Water Company's pipes, a barrel of coal tar is poured into a kettle under which a fire has been started; the barrel contains about 50 gals. Into this heated tar are tossed pieces of crude asphaltum from 2 to 4 ins. in greatest dimensions. This material is obtained from Santa Barbara. As it melts and is stirred into the tar, more of it is added until about 3,000 lbs. has been placed in the kettle. In order to make the coating as tough and tenacious as possible, a second barrel of coal tar is then added very slowly and when all has been placed in the kettle it is allowed to boil for about four hours without stirring. Throughout this refining process the kettle is kept at a temperature of about 300° F., and when the boiling is finished, after about twelve hours from the initial charging of the kettle with tar, the coating is ready to be skimmed to remove the dross and to be baled into a pipe-dipping trough. About 650 lbs. of sand, gravel and bituminous material are left in the bottom of the kettle after each melting and are cleaned out before the melting of another charge is begun. In a long coating operation, it was found that about 1,400 lbs. of crude asphaltum were used with each 50-gal. barrel of oil.

It is considered necessary to have the coating at a somewhat higher temperature, 360 to 400° F., for dipping than for refining; this is maintained by a fire under each end of a long dipping trough, or, if two troughs are used, the second is kept at a temperature about 30° below that of the first. The consistency of the dip is tested frequently by dipping a stick into it, allowing the skin collected in this way to cool, and then observing its resistance to the point of a knife. If the dip needs to be corrected, the proper amount of coal tar or of refined asphaltum from the kettle is placed in the trough. Contrary to the usual practice elsewhere, the pipes are not heated before being coated, but are immersed cold and allowed to remain in the bath until they attain the temperature of the dip. When this condition is reached a bar will slide freely along the immersed pipe, but it will drag if the metal is not yet at this temperature. This procedure takes more time than dipping a hot pipe. For example, a 54-in. pipe of steel 0.275 in. thick must be immersed in the tank about 25 minutes, and before it is removed the dip is stirred and the pipe rolled over in order to have it uniformly coated. It is then raised above the tank and held at an angle of about 45° to drain and cool. When the coating ha become firm but is still very sticky, the pipe is dipped again, either in the first tank or in another. It is rolled over in the tank and after remaining there from three to five minutes, it is lifted out, held at an angle of 45° again, and when cool is removed on skids covered with sand.

The writer examined a large number of pipe coated in this way, which had been in service for various periods up to 47 years. The interior coating generally adhered tenaceously to the metal and could be dented by pressing the finger-nail against it. It was smooth and unbroken and could be pushed aside by a slow, hard pressure without cracking. When this was done the metal below the coating was found to be clean, even where mill scale covered it. There was very little blistering, tuberculation or pitting, and it is doubtful if the carrying capacity had been reduced by these causes more than 10 per cent on the average. In the worst case found, which was also the oldest pipe, the reduction in capacity was shown by tests to be under 20 per cent. The condition of the exterior coating was hardly so good, but was satisfactory except where the soil conditions were unusually destructive on account of organic or acid substances in them.

In the Los Angeles district, the writer found that steel pipe were not dipped but were painted in a manner which had given satisfaction during an experience covering many years. The paint used was the pitch left in making gas from the local asphaltic oils, which is a very different material from the tar produced in the Eastern states. Ordinarily the material is applied with a brush without any preparation, but in cold weather it is heated and thinned with distillate.

This Pacific Coast experience is instructive in view of the experience in the Eastern states, where difficulty has been experienced in producing satisfactory pipe dips from mixtures of asphalt and tar. The Pioneer pipe dip, which has been much used, was produced by a process kept a trade secret until the time of the litigation to establish the validity of the Byerly blown-oil patents a few years ago. Testimony was then given that this dip was a mixture of 28 per cent of gilsonite and 72 per cent of petroleum residuum, made by melting the materials and blowing air through them for 35 to 37 hours. This is manifestly a much more expensive and complicated process than that followed by Mr. Schussler's staff, but the actual dipping in the material has been carried on more rapidly. A vertical tank has been used, as a rule. The pipe is heated to the temperature of the bath, plunged in the latter for a few minutes, and then removed to drain and cool. In actual practice, there has been some difficulty in heating the pipe uniformly to the temperature desired and in keeping the bath in a uniform condition. These difficulties have proved more troublesome with coal tar dips than with those containing asphaltum, and the writer is inclined to believe that some of these troubles may be due to departures from the pipe coating methods recommended by the originator of the tar varnish, Dr. Angus Smith. The earliest statement concerning these methods published in this country was probably written by him to James P. Kirkwood in 1850 and printed in a report of 1858 on the Brooklyn water-works. It reads as follows:

The pipe is made clean, free from rust and earth which clings to it in coming from the molds. The cleaning is a very important thing, as the success very much depends upon it. The surface is then oiled with linseed oil in order to preserve it until it is ready to be dipped; when the coating is to be made the pipe is heated in an oven to about 300° F. It should also be managed in such a manner as to prevent soot from settling on it. It is then dipped into a pan of gas pitch and kept in it for some time until it has taken up the pitch as intimately as possible. The pitch should not be too hard, so hard as to be brittle; nor should it be too soft, so as to adhere to anything. When it becomes too hard it may be softened by adding more oil; when the pipes are taken out they are covered with a fine black varnish and look exceedingly well.

An oven is made to heat the pipe in, and from it they are transferred to the pitch vessel; they are dipped vertically, slowly removed, the liquid running off very clear, leaving a very thin coating. . . . I do not know if you have any distilleries of tar in New York, but, if so, you will readily obtain the proper pitch; we like it distilled till the pitch is about the consistency of wax in our climate. If hard, the mixture of 5 or 6 per cent of linseed oil is a great advantage, or even if not very hard.

This coating was made with coal-gas tar, and was used on a large part of the Scotch iron pipe imported into this country many years ago. As made at Manchester, England, in 1849, the pitch or tar was free from naphtha compounds and was cut with enough mineral oil to give it the required consistency. The oiling of the pipe, mentioned in Dr. Smith's letter, was considered an important detail at Manchester. The late George W. Rafter was informed in 1895 (Trans. Am. Soc. C. E., xxxi, 293) that these pipe were believed to be practically as clean then as when first laid. The imported pipe in Brooklyn and Boston remained in good condition for a long period of years, and the early coating honestly applied in this country under the same or equivalent specifications gave excellent results. With changes in the gas industry there came corresponding changes in tar. Higher temperatures were used, different materials were employed and the tar produced in most gas works today resembles the tar recommended by Dr. Smith in little more than color and odor. It is not surprising, therefore, that a great deal of dissatisfaction has been experienced for a number of years with tar pipe coatings, and in view of the great demand for old-fashioned coal-gas tar road binders it will probably prove troublesome to get the proper material for pipe preservation. Unless it is obtained, however, it is questionable if the tar coating will be reliable. Furthermore, experience indicates that a tar pipe dip must be heated more carefully than an asphaltic dip, for the range of temperature within which it is in a suitable condition to receive the pipe is smaller. This concerns not only the bath but also the heating of the pipe before it is immersed. It is improbable that as good coating with a tar preparation cannot be done today as was done 50 years ago, but it evidently cannot be done without much care in the selection of the materials and the conduct of the work.

In order to reduce the danger of rusting and tuberculation to a minimum, the sections of pipe have sometimes been brushed clean, then dipped in weak acid and finally dipped in lime water to neutralize the acid, before they were dipped in the coating. Theoretically this is a desirable procedure, but it has been found in practice to be difficult of execution, owing to erratic changes in the acid and alkaline baths. Where a high degree of protection is desired for many years, the money needed for this pickling process may possibly be better spent in wrapping the pipe with burlap dipped in the coating. This has been done on a number of pipe lines in the last two years, and the burlap undoubtedly protects the coating from many of the scratches and bruises it would suffer during transportation and handling, if not covered in this way.

The importance of the subject can only be appreciated by making an actual comparison of cast-iron and steel pipe for a given case. The life of the steel pipe will be found to be a very important factor and this life depends very largely on the pipe coating. If a steel pipe begins to leak in seven years on account of pitting, as was the case with the Rochester conduit of 1893-4, and costs annually thereafter from $300 to $400 per mile for stopping leaks through pitted plates, this peculiarity must be taken into account. In the case of the Portland, Ore., riveted steel pipe laid in 1895-6, the rust caused leakage in 0.2-in. plates in about nine years and a few years later in 0.25-in. plates. On the other hand no leakage due to corrosion appeared in the Newark, N. J., conduit laid in 1890-1 until 1909. Such pitting does not appreciably injure the strength of the pipe so long as the spots are not close together. It is not difficult to repair the early leaks due to such corrosion. But it is very difficult to reach a satisfactory estimate of the life to be assumed for such a pipe in making estimates of the relative total annual cost of steel and cast iron. It is certain that steel has not shown such durability as wrought iron. Wrought-iron pipe are practically all there are to furnish long-time records of service, so in order to be able to use the wrought-iron data in connection with steel, the greatest care must be taken in preparing and applying a coating, and the hasty methods now followed in some cases should be regarded very critically before they are permitted.

Design Features of the Proposed Water Supply and Purification Works, Corpus Christi, Texas.

(Staff Article.)

The new water supply of Corpus Christi, Texas, will be drawn from the Neuces River. The hydraulic elements of the proposed works are shown herewith in the diagrammatic profile, Fig. 1. This is the form of sectional profile which we commended in an editorial some months ago. The reader's attention is directed to the fact that a single glance at Fig. 1 shows all the hydraulic elements of this interesting design. The relation of parts is at once made apparent and subsequent reference to more detailed drawings is greatly facilitated by the form of diagram shown in Fig. 1. As indicated the raw water passes through the grated opening in the intake structure to the 20-in. cast iron intake pipe leading to the suction well in the lower portion of the combined suction well, sterilization and coagulation chamber and wash water tank structure. The water is then picked up by the high lift pump and sent through the pressure filters. Thence the water passes through a surge tank to the 20-in. main leading to the city.

The plant as designed has a working capacity of 1,500,000 gals. daily, which can for short periods be increased to a maximum rate of 3,000,000 gals.

Intake.—In order not to interfere with the continuous operation of the existing plant, and also for economic reasons, it was thought best to construct a new plant entirely independent of the existing pumping plant. The new intake consists of a 20-in. cast iron pipe line about 60 ft. long. At its outer end this pipe line is protected by a concrete structure 9 ft. square and 10 ft. 8 ins. deep, carried up to mean water level. A screen, consisting of 2½x¼-in. bars placed ¾ in. on centers, protects the intake pipe from drift and other foreign matter. The detail of the intake is shown in Fig. 2.

Suction Well, Sterilization and Coagulation Chamber and Wash Water Tank.—The details of this structure are shown in Fig. 3. The 20-in. cast iron intake pipe discharges into the suction well, a circular reinforced concrete structure 15 ft. in diameter carried down 12 ft. below the river level. Cast iron suction pipes extend down into this well and convey the water to the high-lift pumps. At elevation 18 the suction well is covered with a reinforced concrete floor, on which are located the sterilization and coagulating apparatus. Both the sterilizing medium and the coagulant are introduced into the water at the suction well. Directly above the suction well at a height of 16 ft. above the floor there is to be erected a reinforced concrete water tank 15 ft. in diameter and 18 ft. deep, holding 25,000 gals. of filtered water. The water thus stored is to be used for washing the filters.

High Lift Pumps.—The high-lift pumps are housed in a brick building 24 ft. 6 ins. wide and 72 ft. 6 ins. long. Within this building are to be installed the two 750,000-gal. triplex pumps direct connected to 50-HP. De La Vergne oil engines which were purchased last year by the city for the temporary relay pumping station. There is to be purchased an additional 1,500,000-gal. triplex pump, direct connected to a 100-HP. De La Vergne oil engine of the F. H. type. The F. H. type of engine is only manufactured in the larger sizes. It is very much more economical than the smaller units. An oil engine of this type develops a brake horsepower at about 60 per cent of the fuel consumption required by the smaller units.

Filtration Plant.—The waters of the Nueces River from a sanitary standpoint are, as shown by analysis, of fairly good quality. However, the mineral constituents of the water vary widely with the flow and are present at times in such large quantities as to render the water objectionable for a domestic supply. The quality of the water will be improved by the ultimate construction of a storage reservoir outside of the river basin. In such a reservoir the flood waters of the river will be stored, which are of far better quality than the water which ordinarily flows in the Nueces River. There will be no objection to the turbidity in these flood waters, as this will be removed to a very great extent by sedimentation in the storage basin, and the water is to be purified by rapid sand filtration.

This Calallen site possesses an advantage over the Tule Lake site, formerly considered, in that the city, not being entirely dependent upon storage, can select water of a better quality for storage; whereas, at Tule Lake the reservoir must be replenished at every opportunity when the river water is free from salt irrespective of its other qualities.

Pressure filters are preferred to the open type of sand filter, for economical reasons and no others. A pressure filter installation, where properly designed and equipped with a system of flow controls, will give almost, if not entirely, as good results as the open type at about one-quarter the cost for maintenance and operation. The plans provide for the immediate installation of three 500,000-gal. units; further units can be added from time to time as necessary.

The total quantity of water pumped to the city is to be measured, after being filtered, by means of a recording Venturi meter.

Surge Tank.—It is not advisable to pump directly through the 15-mile pipe line to the city, as the strains set up in the pipe line by direct pumping are considerable and may be as much as 50 lbs. or more per square inch over and above the hydrostatic pressure. By

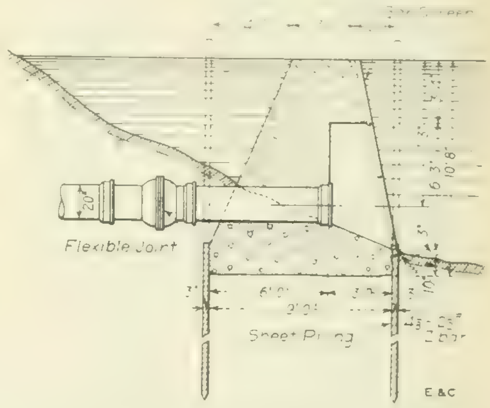

Fig. 2. Sectional Elevation of Proposed Water Supply Intake Structure Corpus Christi, Texas.

pumping into a standpipe at Calallen and not directly into the main leading to the city, all strains in the pipe line due to pumping will be practically eliminated. A standpipe, to be efficient, would have to be about 250 ft. high. Because of this height its construction is not recommended, especially in view of the fact that it is possible to build an air-tight tank which will perform the work equally well at less than one-fifth of the cost of such a standpipe. Such a tank is shown in Fig. 4. It is 8 ft. in diameter and about 22 ft. high. Under operating conditions it will be filled about half full of compressed air, which absorbs practically all of the shocks from the pumps. When the water rises in the tank to a predetermined level thereby reducing the air space, a float operating an electrical connection will ring an alarm in the pumping station. A tank such as described is advisable irrespective of the material of which the pipe line is to be constructed.

Pipe Line.—The most important element entering into the proposed improvement, from the standpoint of cost, is the 15-mile pipe line from the surge tank to the city. The cost of this line will be fully 75 per cent of the total expenditure. The line is designed to carry 3,000,000 gals. daily, which it is estimated will serve the city for the next 20 years. The materials available for this line are cast iron, steel, concrete and wood. The bidding prices will govern the choice of material. The first two and the last type of pipe are of the usual standard construction

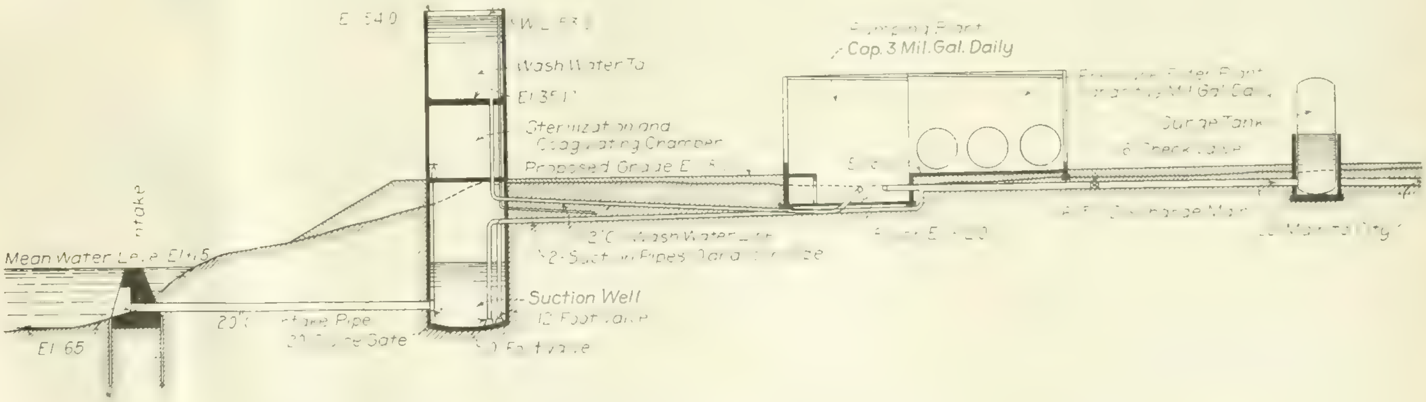

Fig. 1. Diagrammatic Profile of Proposed Water Supply and Purification Works, Corpus Christi, Texas.

and call for no further comment here. The consulting engineer's comments on reinforced concrete pipe, however, are of special interest and are here given in substance.

On his trip to Europe in the winter of

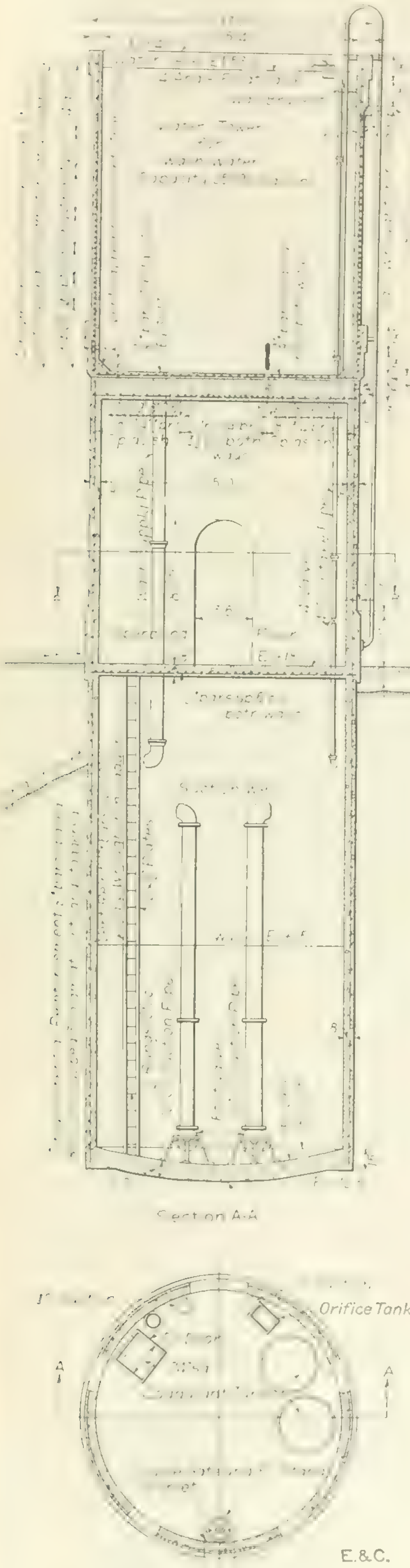

Fig. 3. Sections of Proposed Combined Suction Well, Sterilization and Coagulating Chamber and Wash Water Tank, Corpus Christi, Texas.

1913-14, Mr. Potter investigated a type of reinforced concrete pipe manufactured by the "Jagger" process, a process in which the concrete is compacted by oscillation and vibration to an extent which renders it absolutely watertight under the pressure to which such a pipe line would be subjected at Corpus Christi. The interior of pipe manufactured by this process is as smooth as glazed stone ware, which gives it about the same carrying capacity as cast iron pipe of the same diameter five years old. The physical life of this pipe is fully as long as that of cast iron, in the judgment of the consulting engineer. Pipe manufactured by the "Jagger" process designed to resist a working pressure of 100 lbs. can be manufactured at Corpus Christi for about $1.50 per foot. The cost of laying such pipe in the trench is said to be no greater than that of laying cast iron pipe.

PERSONNEL.

Mr. Alexander Potter of New York City is Consulting Engineer on this work, and Mr. H. A. Stevens is City Engineer. The information here presented has been compiled from the plans, specifications and report prepared by Mr. Potter.

A Discussion of the Elements of Water Works Accounting.

There are three interests involved in every water plant: (1) the proprietors, (2) the public, (3) the managers. It is a function of accounting to present a history of a plant in such form that it may be used as a guide by each interest in its future conduct toward the plant. This history should be presented periodically and at least annually. Mr. R. A. Stevenson, instructor in accounting at the Iowa State University, recently had the opportunity of investigating the report of a large number of water plants, both municipally and privately owned. He read a paper, from which the following matter is taken, at the second annual field day, Nov. 20, 1914, in which he discussed the evidences of poor and indifferent accounting by water utilities. He also discusses the elements of the subject and gives examples to illustrate the system of accounts that he recommends.

The report of a certain municipally owned plant of a city with a population of about 20,000 is taken as typical of the reports issued by all of them. For the purposes of this discussion we may call the city "Urban." Although this was a municipal plant, the questions involved are applicable to both private and municipal plants. The only difference between the two being that in a municipal plant the public assumes the interest of proprietor as well as consumer. The report investigated continued the figures for the two years 1912 and 1913. The first question concerned the investment. How much was the public's investment in the plant? A thorough investigation of the report brought forth the following information: From the Annual Report of the Urban Water Department for 1912:

1. Taken from page 3 of the report

Old water works plant	$ 15,000.00
Lake Michigan pumping station	219,410.00
Water main extension to date	512,975.72
Total	$747,385.72
Water department supplies on hand	$ 22,375.99

2. Taken from page 10 of the report:

Bonded indebtedness:

Water works bonds, series of 1890, 5 per cent	$140,000.00
Water works bonds, series of 1911, 4½ per cent	145,000.00
Lake Michigan water works ref. bonds, 5 per cent	20,000.00
Lake Michigan water works ref. bonds, 4 per cent	25,000.00
Water main extension bonds, 4½ per cent	300,000.00
Total	$630,000.00

This is all the information that it was possible to obtain from the report and it is all that the state required the cities to keep. An investigation was made into all the city records and another one of the physical property to determine its present value. It was found that the old water works plant had been abandoned 20 years before and was worthless and that the plant in use had depreciated by $30,000. The unpaid water accounts were $7,550. There was a sinking fund of $15,000 accumulated to retire the bonds. The cash balance for the water department was $23,-

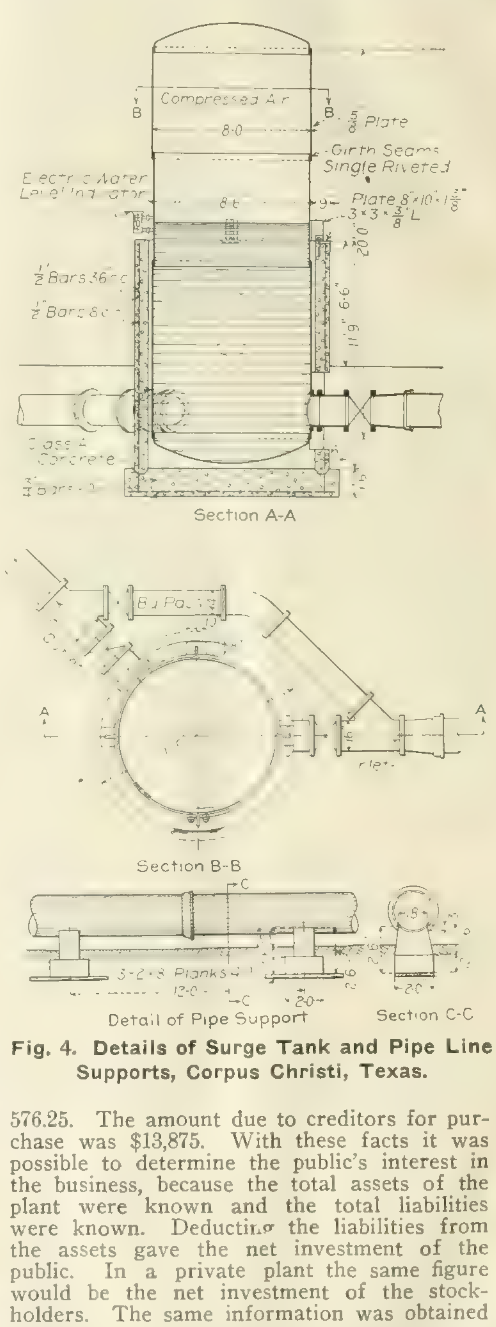

Fig. 4. Details of Surge Tank and Pipe Line Supports, Corpus Christi, Texas.

576.25. The amount due to creditors for purchase was $13,875. With these facts it was possible to determine the public's interest in the business, because the total assets of the plant were known and the total liabilities were known. Deducting the liabilities from the assets gave the net investment of the public. In a private plant the same figure would be the net investment of the stockholders. The same information was obtained

TABLE 1. COMPARATIVE BALANCE SHEET FOR 1913 AND 1912 OF THE "URBAN" WATER DEPARTMENT REARRANGED FROM WATER DEPARTMENT REPORTS:

Assets	1913.	1912.
Pumping station	$ 19,110.00	$219,110.00
Distribution system	612,975.72	512,975.72
Water property used for—		
Municipal service	40,000.00	35,000.00
Water dept. supplies	44,754.15	22,375.99
Sinking fund	[illegible]	15,000.00
Accounts receivable	12,950.00	7,550.00
Cash	10,955.76	23,576.25
	$971,045.63	$835,887.96

Liabilities.	1913.	1912.
Bonds:		
Series 1890, 5 per cent	$140,000.00	$140,000.00
Series 1911, 4½ per cent	145,000.00	145,000.00
Series 1913, 4½ per cent	100,000.00	....
L. Mich. W. W. Ref. 5 per cent	20,000.00	20,000.00
L. Mich. W. W. Ref. 4 per cent	25,000.00	25,000.00
Water main extension	300,000.00	300,000.00
Creditors' accounts	15,450.00	13,875.00
Depreciation reserve	40,000.00	30,000.00
Sinking fund reserve	30,000.00	15,000.00
Surplus	155,595.63	147,012.96
	$971,045.63	$835,887.96

for the year 1913 and the net investment determined. The information for the past two years was set down in a comparative balance sheet as shown in Table I.

This statement shows the net investment to be $147,012.96 for 1912 and $155,595.63 in 1913, plus the sinking fund reserve of $15,000 in 1912 and $30,000 in 1913. The sinking fund created through a reserve is simply a means of purchasing the plant from the bondholders. If the plant is maintained and a sinking fund reserve created besides, the reserve becomes part of the net investment of the proprietors when the bonds are retired. It is sometimes argued that the net investment figure for a municipal plant is of no significance. That figure represents the financial advantage to the present water consumers of owning the plant assuming that it could be managed just as efficiently under either form of management. The plant of the city of "Urban" is valued at approximately $805,000 (after deducting the depreciation from the cost figure); the indebtedness amounts to $658,000, leaving a net surplus of approximately $147,000. The consumers of water must pay in rates enough to cover all expenses and the interest on the interest-bearing liabilities, while if the plant were owned by a private corporation, the consumer would have to pay enough in rates to cover all expenses and a return on the whole value of the plant, $805,000. Therefore, the surplus or net investment in a municipal plant is a measure of the financial advantage to the present consumers from the public ownership of the plant. What further information may be gathered from this balance sheet? The net investment (surplus) has increased $8,582.67, besides the $15,000 added to the sinking fund revenue during the year; $100,000 has been expended on the distribution system, that amount having been raised through the issuing of $100,000, 4½ per cent bonds. The plant has also depreciated $10,000 during the year as shown by the depreciation reserve. This comparative balance sheet then is sufficient for showing the condition of the plant. It gives an analysis of financial changes in the business. The balance sheet should be the cornerstone of any accounting system. While the balance sheet gives a comparison of net results, it does not show any of the causes for the increase or decrease in surplus. What were the causes for the increase of $8,582.67 in the surplus item in the balance sheet? It is the function of the income statement to analyze that increase. An examination of the water department report of Urban showed that the nearest approach to an income statement was an extract from the City Treasurer's report for 1913, referring to the finances of the water plant. That report follows:

From the Treasurer's Report of the City of Urban for the year 1913:

Receipts and disbursements of the water department.

1. Receipts:		
Assessed rates	$33,967.36	
Metered water	29,722.48	
Mason work	836.04	
Metered rentals	172.44	
Miscellaneous	71.75	
Total receipts		$64,770.67
2. Disbursements:		
Operation—		
Pumping station expense	$ 6,40[illegible]	
Fuel	5,061.16	
Office and distribution expenses	10,678.64	
Maintenance—		
Repairs to distribution system	607.35	
Repairs to pumping station	644.19	
Repairs to meters	581.50	
Interest on bonds and sinking fund	45,000.00	
New extension	8,000.00	
Insurance	414.88	
Total disbursements		77,390.56
Deficit		12,620.49

On the basis of these figures, the City Treasurer reported that the water plant had been operated at a loss of $12,620.49, and recommended that the rates be raised. Can we base a judgment as to the reasonableness of rates from the above statement? It is true that the treasurer paid out $12,620.49 more of water department funds that he took in. What were the payments for? Were they all expenses? An income account is necessary to answer those questions. The income statement should first show the total revenue for the year, whether actually collected in cash or still owed by the consumer. The above report shows only the amounts actually collected in cash. What constitutes revenue to the plant? A certain amount of water is pumped into the mains at the pumping plant and is all consumed for different purposes and by different parties. A benefit is conferred to each party for all the water consumed. The return for those benefits constitutes the revenue. The revenue should be classified as to source somewhat as follows:

1. Private service; (a) metered; (b) unmetered.

2. Municipal service; (a) fire protection; (b) parks; (c) public buildings, etc.

3. Miscellaneous service; (a) meter rentals.

It may seem like taking money from one pocket and putting it into another to call municipally consumed water a revenue to a municipally owned plant. There is no necessity for the transfer of money by the Treasurer to treat that service as revenue, and in order to determine the reasonableness of rates, it is necessary to so treat it, and to charge it to the department of the city which receives the benefit. It is the policy of cities to use the property tax receipts for the cost of protecting property and special taxes for parks and public buildings. If no charge is made against the funds created for these purposes and credited to the revenue of the water department for water consumed for the city use, the water users are paying part of the expenses for the protection of property, the beautifying of parks and the maintenance of public buildings which should be apportioned to the citizens on some other tax basis. The revenues of the "Urban" water plant should be shown as follows:

1. Private Service: There was $29,722.68 received in cash for metered service as shown by the Treasurer's report. There was $8,740 more of water used during the year and still due from consumers which also constitutes revenue. The unmetered was $33,967.36. The total private service, therefore, was $72,429.84.

2. Municipal Service: An investigation of the water used by the city showed that the water for fire protection if charged on the meter rates would amount to $10,000, for parks $5,000 and for public buildings $9,500, making the total municipal revenue $24,500.

3. Miscellaneous revenue amounted to $1,080.23.

The total revenue then amounts to $98,010.07, although the receipts as shown by the treasurer's report were only $64,770.07.

The expenses should next be deducted from the revenue to find the net revenue. The total disbursements of cash are not always all expenses. Some cash may be paid out for new property or improvements, etc. On the other hand, there may be some expenses which are not represented in cash disbursements, such as depreciation. The expenses must represent the actual cost of bringing in the revenue represented in the revenue section. They may be classified as follows:

1. General Expenses. (a) Administration. (b) Insurance, etc.

2. Operating Expenses. (a) Operating management. (b) Collection system. (c) Distribution system.

3. Maintenance Expenses. (a) Collection system repairs. (b) Distribution system repairs. (c) Depreciation on plant.

The item of depreciation on plant must be included in expenses to obtain the correct cost of operation. In spite of all repairs, the plant is gradually wearing out and at some time will have to be replaced. The cost of the plant throughout its life is just as much a cost of operation as materials furnished which are used up within the year. If we were to make out an income account to cover the whole life of a plant, one of the expense items would be Cost of Plant. Since we are making our income accounts several times during the life of the plant, we must apportion to each year the value of the plant used up during the year. If this were not done, at the end of the life of the plant, the expenses of one year would be burdened with the whole cost of the plant. The expenses should be restated as follows:

1. General Expenses. These had been lumped in with the operating expenses so that insurance was the only item under this head which could be found—$414.88.

2. Operating Expenses. These were correctly stated at $22,142.64.

3. Maintenance. The repair items were correctly stated in the report. Depreciation on the plant for the year was estimated by engineers at $10,000, making the total maintenance $11,869.88.

The total expenses amounted to $34,427.40, although the total disbursements were $77,390.56. Deducting the expenses from the revenue gives the net revenues $63,582.67. All expenses having been met, the next thing is to meet the capital charges, that is the payments made for the use of capital. In this case it is interest on bonds and amounts to $30,000, included in the item interest on bonds and sinking fund in the treasurer's report. The other $15,000 is an allowance for sinking fund and is not a deduction from revenue. It is not a cost of the capital, it is simply a saving of a part of the profits with which to buy out the bondholder's interests. It is estimated, however, that $2,000 of the unpaid accounts will prove worthless and these must be charged against the net revenue. The total charges of $32,000, deducted from the net revenue, gives $31,582.67, the net profit on the plant, instead of a deficit of $12,620.49, which is the excess of expenditure over receipts. Of the $31,582.67 net profit, $8,000 was used to extend the plant and $15,000 was put into the sinking fund to retire the bonds. The remainder, $8,582.67, was carried to surplus and explains the increase in the surplus as shown by the comparative balance sheet for the two years. The reconstructed income statement is as follows:

Reconstructed Income Account of the City of Urban for the year 1913:

1. Private Service:		
(a) Metered (cash), $29,722.48, plus charged $8,740.	$38,462.48	
(b) Unmetered	33,967.36	$72,429.84
2. Municipal Service:		
(a) Fire protection	$10,000.00	
(b) Parks	5,000.00	
(c) Public buildings, etc.	9,500.00	24,500.00
3. Miscellaneous Revenue:		
(a) Special work	$ 836.04	
(b) Meter rentals	172.44	
(c) Sundry accounts	71.75	1,080.23
Total revenue		$98,010.07
EXPENSES.		
1. General Expenses:		
(a) Administration.		
(b) Insurance, etc.	$ 414.88	
2. Operating Expenses:		
(a) Operating management.		
(b) Collection system	$11,464.00	
(c) Distribution system	10,678.64	$22,142.64
3. Maintenance Expenses:		
(a) Collection system repairs	$ 644.19	
(b) Distribution system repairs	1,225.69	
(c) Depreciation on plant	10,000.00	11,869.88
Total expenses		$34,427.40
Net revenue		$63,582.67
Deductions from net revenue:		
1. Interest on bonds	$30,000.00	
2. Bad debts charged off	2,000.00	$32,000.00
Net profit		$31,582.67
Distribution of profit:		
1. New construction	$ 8,000.00	
2. Sinking fund	15,000.00	$23,000.00
Net amount carried to surplus		$ 8,582.67

If the public were to base its judgment on the treasurer's report, the rates would have to be raised, whereas the plant is actually operating at a profit and in a condition to reduce the rates. The reports then in which the proprietors and public are interested are the balance sheet and income statement. With these two statements which give a concise history of the business, they are both able to form an accurate judgment as to the condition of the plant and as to the reasonableness of rates. Having formed a judgment, they can exercise their power of control to the best interests of the public and the plant. The special problems of the manager are: (1) the determining of causes behind the expense account; (2) the determining of unit costs of distribution; (3) unit costs for laying pipe, installing meters and services; (4) the location of waste from the loss of water and other sources; the results obtained from the installation of meters or from the installation of any new device. The manager demands statistics which will serve as an aid in the direct management of the plant. In order to give the needed information to the managers it is necessary to develop a set of statistics which will keep a record of each class of information desired. These statistics may be entirely separate from the financial accounts. An account should be opened for laying of pipe to get the unit costs of laying pipe. The same should be done for installing meters. There would have to be kept statistics showing the amount of water pumped and then the amount recorded by the consumers' meters to locate lost water, etc. What should a water plant which has no adequate accounting system do to install one? There are just four steps to be taken:

1. Take a complete inventory of all the assets and liabilities to construct a balance sheet as a starting point.

2. Employ a bookkeeper to look after all the accounts of the department, including the manager's statistics.

3. Study the plant with the view of having the necessary forms ruled to fit the needs of the plant. The forms to be used are of the utmost importance. They should be ruled so that the information may be readily entered on them and readily collected from them. Too many forms will make a person feel that the information is not worth the bother or that there is too much "red tape."

4. After the system is installed, keep it working—give it a chance.

BUILDINGS

Design Features of the Open Hearth Building of the Pennsylvania Steel Co., Steelton, Pa.

(Staff Article.)

The Pennsylvania Steel Co., in carrying out a general program of reconstruction, has built a centralized steel making plant, which affords it excellent facilities for producing any grade of Bessemer or open-hearth steel by any known process. The company's original Bessemer converter plant, which was built in 1867, consisted of two small converters, while the original open-hearth plant was [illegible] another location and was [illegible] improved. The main [illegible] of the present open-hearth plant, which was built in 1905-1906, is 448 ft. long by 172 ft. 5 ins. wide, with gas producer buildings and a calcining plant. During 1913 an extension 392 ft. long was built at the west end of the main building, and another extension 84 ft. long was constructed at the east end, the minor buildings also being extended where necessary.

GENERAL FEATURES.

Figure 1 is a plan of the main section of the open-hearth plant and of the east and west extensions, and Fig. 2 is a cross section of the main and auxiliary buildings. These drawings give the locations of the converters [illegible] and indicate the type of con[illegible]

[illegible] consists of two parallel crane runways, one of which is 72 ft. and the other 65 ft. wide. The 72-ft. runway is equipped with two 50-ton "Morgan" cranes and one 75-ton "Alliance" crane, with 25-ton secondary hoists. The 65-ft. runway is served by two 115-ton "Morgan" ladle cranes equipped with 25-ton auxiliary hoists and one 150-ton "Alliance" crane with separate auxiliary hoists having capacities of 40 tons and 25 tons respectively.

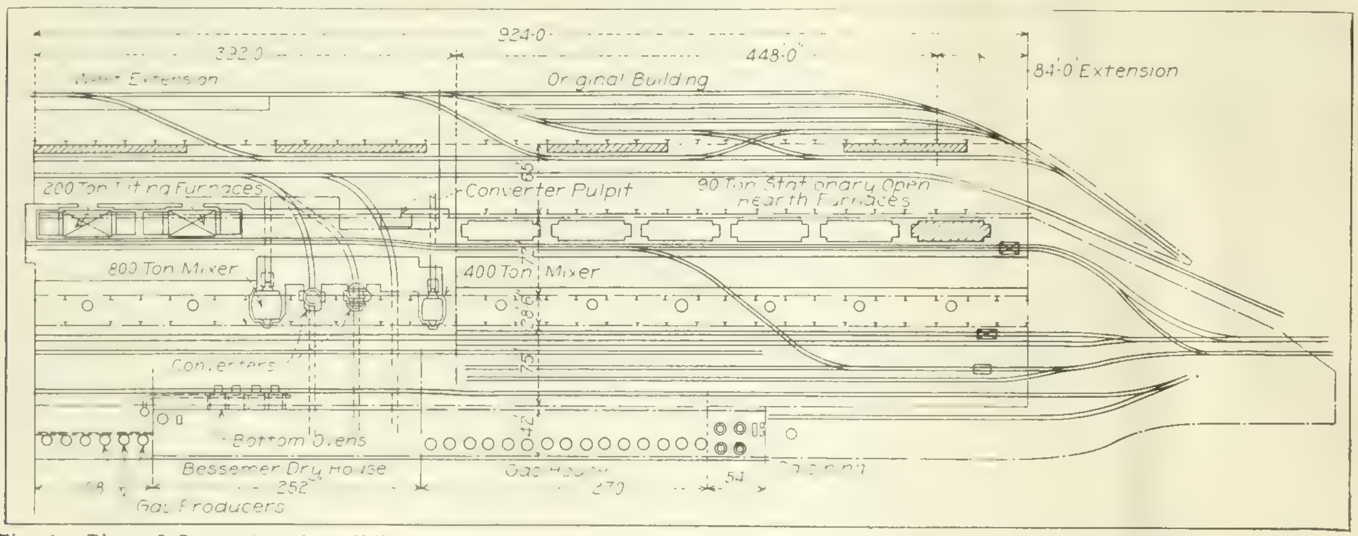

Fig. 1. Plan of Open Hearth Building of Pennsylvania Steel Co., Steelton, Pa., Showing Layout of Furnaces and Equipment.

The six 90-ton stationary open-hearth furnaces (see Fig. 1) are modern in all particulars, and have a total capacity of 25,000 tons of ingots per month, using hot pig iron and cold scrap. These furnaces are fed by two "Wellman" charging machines.

At the west end of the building there are two 200-ton tilting furnaces. The fronts of

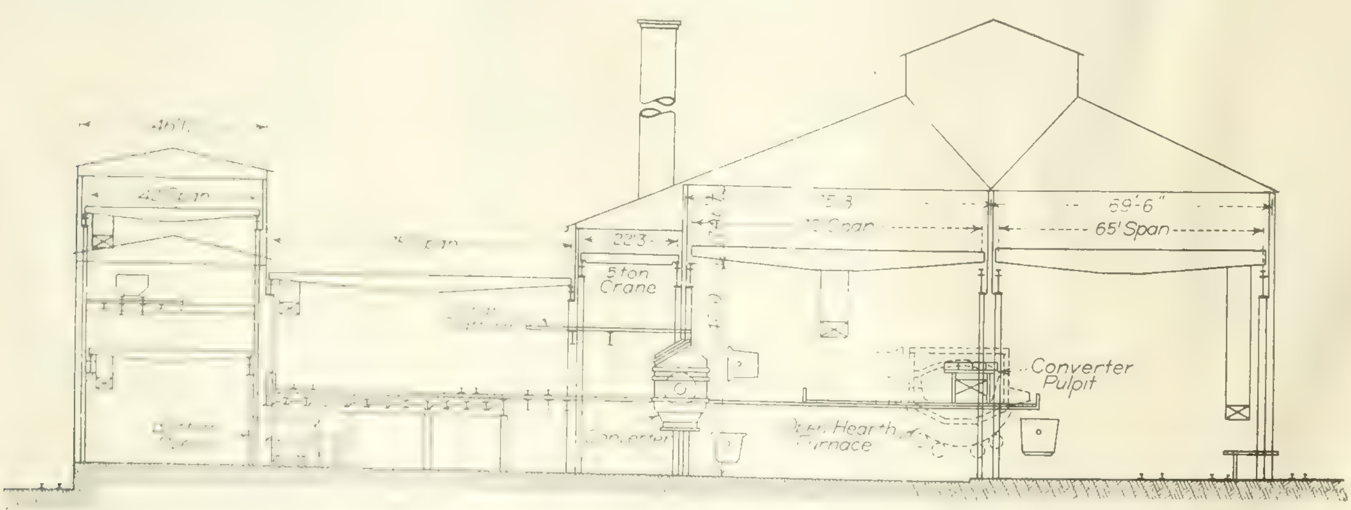

Fig. 2. Cross Section of Main and Auxiliary Buildings of Open Hearth Plant of Pennsylvania Steel Co.

these furnaces are water cooled, and the doors are electrically operated. The estimated capacity of each of these furnaces is 6,000 tons per month when using a mixture of pig iron and scrap, and 20,000 tons per month when duplexing. These furnaces are also charged by a "Wellman" machine.

The metal mixers, one of 800 tons and the other of 400 tons capacity can be fired either with tar or oil.

The Bessemer converters (see Fig. 1) are tilted electrically and are arranged so that they can be charged and poured from the same side. The cranes from the stock yard place the scrap on a platform, from which it is dropped into the converters by a 5-ton "Shepard" crane.

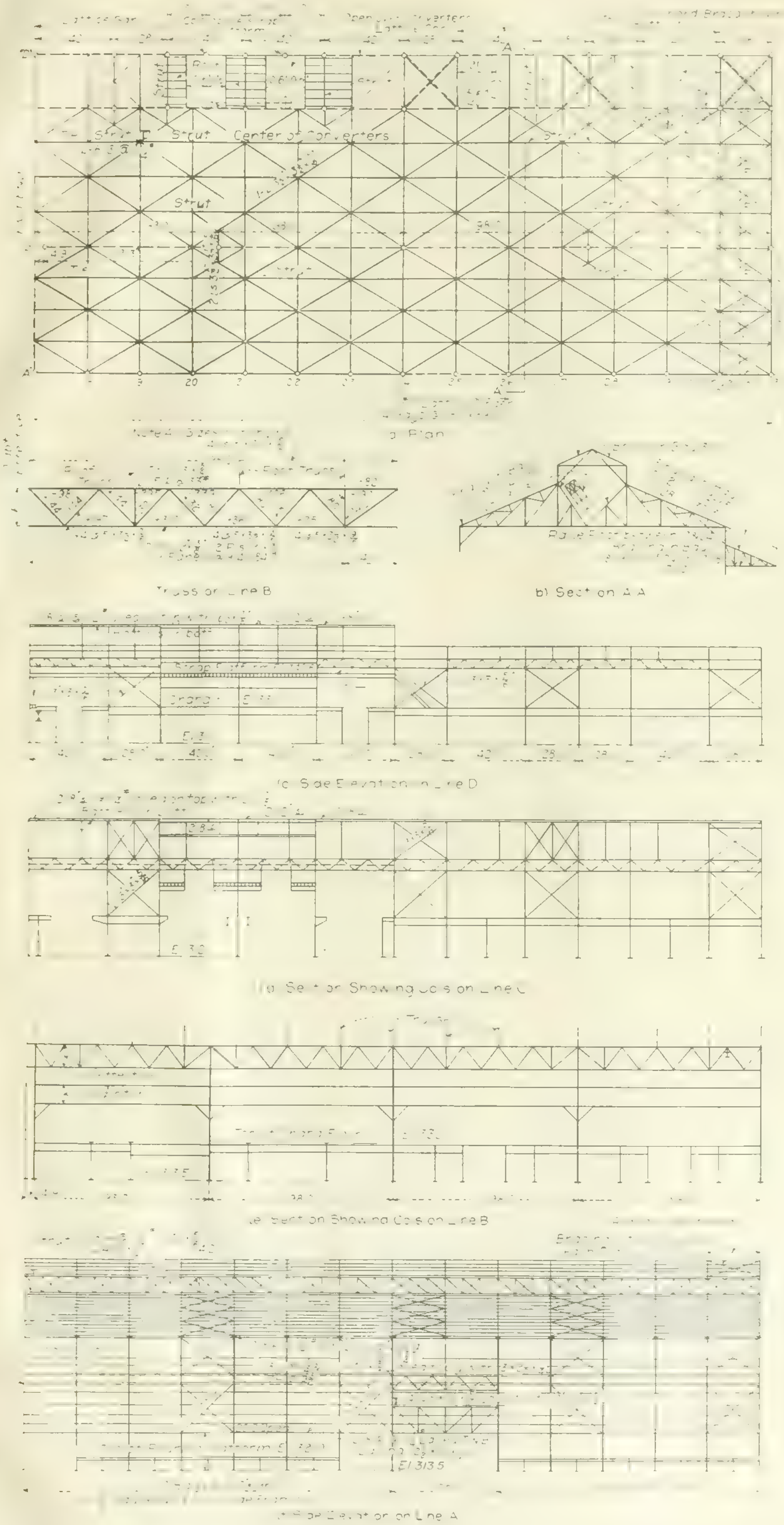

Fig. 3. Framing Plan and Elevations of Main Building of Pennsylvania Steel Co.'s Open Hearth Plant.

The heats are generally handled in 110-ton capacity ladles, which are equipped with the usual bottom stopper arrangement.

The stock yard is 924 ft. long by 75 ft. wide, and it is served by two 75-ft., 10-ton "Morgan" cranes, equipped with lifting magnets, and one 25-ton "Alliance" crane, the latter being used principally to handle the iron ladles from the blast furnaces, where they are emptied into the metal mixers. These iron ladles have a capacity of 235 cu. ft. each, and they are moved along a track at the side of the stock yard.

STRUCTURAL FEATURES.

To illustrate the structural features of the open-hearth building the west 392-ft. extension will be considered, this extension consisting of fourteen 28-ft. bays. Figure 3 (a) is a framing plan of the bracing used in this extension, and Fig. 3 (b) is an elevation of the trusses, the bracing in the plane of the lower chord being shown by full lines and that in the top chord by dotted lines. It will be noted that no lateral bracing is used in the planes of the top chords of the trusses in the main part of this addition (the north 145 ft. 2 ins. of it). The "lean-to" at the south side has both top and bottom lateral bracing, as shown. The purlins, which are not shown in this drawing, give sufficient bracing to the top chords of the trusses.

Along the line marked *"D"* in Fig. 3 (a) the main column spacings are 28 ft. and 42 ft. The spacing of these columns and the framing in the plane of them are shown in Fig. 3 (c). The framing along the line marked *"C"* in Fig. 3 (a) is shown in Fig. 3 (d); that along the line *"B"* is shown in Fig. 3 (e); and that along the line *"A"* in Fig. 3 (f). These drawings show clearly the general structural features of the building. By referring to Fig. 3 (e) it will be noted that the transverse trusses are supported by 98-ft., parallel-chord, longitudinal trusses, and that the crane runway girders along this line of columns also have a span of 98 ft.

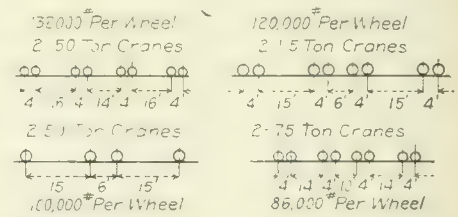

Fig. 4. Spacing of Wheels and Wheel Loads of Heavy Cranes Serving Open Hearth Building of Pennsylvania Steel Co.

The 98-ft. trusses have an effective depth of 12 ft. and are of the type shown in Fig. 3 (e). Each of these trusses supports three sets of transverse trusses at intermediate panel points. The maximum stress in the upper chord of a 98-ft. truss is 339,000 lbs., and its section consists of two 15-in., 33-lb. channels and one 23 x ⅝-in. cover plate. The maximum stress in the lower chord is 361,000 lbs., and its section consists of four 5 x 3½ x 9/16-angles and two 12 x ⅜-in. plates. All web members are composed of four angles.

The 98-ft. crane runway girders (see Fig. 3e) in the 69-ft. 6-in. bay are of the plate girder type, their depth, back to back of angles, being 9 ft. 11½ ins. These girders carry 150-ton and 115-ton cranes, the loadings being as shown in Fig. 4. The maximum moment in these girders is 22,645,000 ft.-lbs., and the maximum end shear is 980,000 lbs.; these values include 25 per cent impact. The bottom flanges of the girders have the following composition, the total net section being 129.22 sq. ins.:

2 8x8x13/16-in. angles.
2 18x⅞-in. flange plates.
1 25½x¾-in. x 77-ft. cover plate.
1 22x¾-in. x 68-ft. cover plate.
1 22x¾-in. x 59-ft. cover plate.
1 22x11/16-in. x 48-ft. cover plate.
1 22x11/16-in. x 35-ft. cover plate.

The composition of the top flange is the same as that of the bottom, except that the cover plates are full length. The web plate is 119 ins. wide and ⅞ in. thick. The end stiffeners consist of four 6 x 6 x ¾-in. angles, with 20 x ⅞-in. fills for the flange plates and 16¼ x 13/16-in. fills for the flange angles. The 19 pairs of interior stiffeners are each composed of two 6 x 4 x ½-in. angles.

The 98-ft. crane runway girders in the 75-ft. 8-in. bay, which carry 75-ton and 50-ton cranes, have a depth of 10 ft. 0 13/16 ins., back to back of angles. The maximum moment in these girders is 15,330,000 ft.-lbs., and the maximum shear is 668,000 lbs. The web plate is 120 ins. wide and 9/16 in. thick, and the net area of the flange is 90.96 sq. ins.

The 28-ft. girders in the 69-ft. 6-in. bays have a depth of 5 ft. 0½ in., back to back of angles, while the 28-ft. girders in the 75-ft. 8-in. bays have a depth of 6 ft. 1⅞ in., back to back of angles. The 42-ft. girders in the 75-ft. 8-in. bays have a depth of 6 ft. 0½ in., back to back of angles.

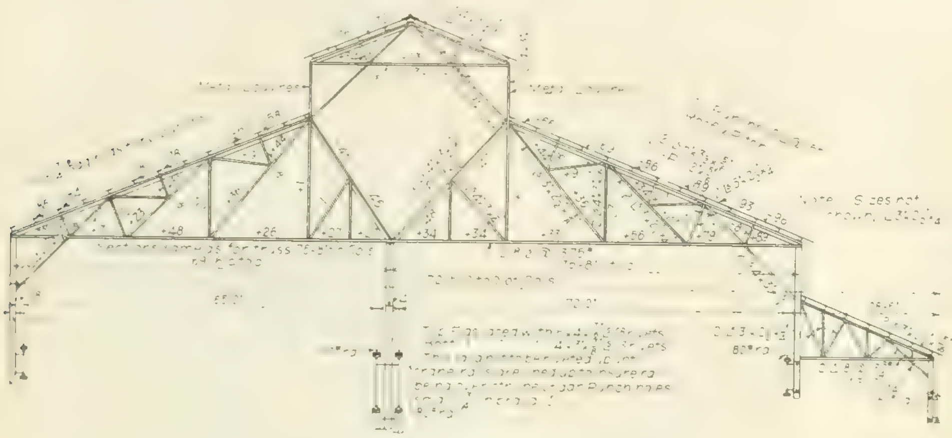

Fig. 5. Roof Trusses Over Bays of Open Hearth Building of Pennsylvania Steel Co., Showing Stresses and Sections.

Figure 5 gives the stresses and sections of the members of the trusses and monitor over the 69-ft. 6-in. and 75-ft. 8-in. bays, and the "lean-to." No lateral bracing is used in the planes of the top chords of the 69-ft. 6-in. and 75-ft. 8-in. trusses as the closely spaced 10-in. channel purlins afford sufficient bracing. The bracing in the plane of the lower chords is shown in Fig. 3 (a).

The height of the columns, and the heavy roof and crane loads carried by them, required the use of several types of column sections. Figure 6 (a) gives the height, loading and composition of the outside columns, Nos. 24 and 26 in line *"A"* (see Fig. 3a). These columns support one end of the 69-ft. 6-in. truss, and carry a line of 98-ft. crane runway girders. Figure 6 (b) gives essential data on the composition and loads carried by the interior columns along line *"B,"* the maximum load on which, including the crane thrust, is 3,318,000 lbs. These columns carry the adjacent ends of the 69-ft. 6-in. and 75-ft. 8-in. trusses and also two lines of 98-ft. crane runway girders, in addition to floor loads. Figures 6 (c) gives data on column No. 21 in line *"C,"* and Fig. 6 (d) gives data on columns Nos. 25, 26½, 28½ and 30 in line *"C."* These columns are along the south side of the 75-ft. 8-in. bay—between that bay and the "lean-to." Figure 6 (e) shows the composition of and the loading on column No. 21 in line *"D,"* the outside line of columns of the "lean-to."

PERSONNEL.

The work described was designed by the Bridge and Construction Department of the Pennsylvania Steel Co., Charles H. Mercer, chief engineer.

Dustless street cleaners, operated on the combined vacuum and sweeper principle, are in use in a number of cities in the United States and Canada. It is claimed that their work is entirely satisfactory, and that after cleaning no sprinkling is necessary, as the dust has been thoroughly removed.

Methods Used to Protect the Foundation of a Heavy Tower During Adjacent Subway Construction in Boston, Mass.

The construction of subways often involves the use of special construction methods to protect the foundation of adjacent buildings. This is particularly true where the buildings are found on material which has a decided tendency to flow. During the construction of a portion of the Copley Square station of the Boylston St. subway west of Dartmouth St., Boston, Mass., the work was made hazardous by the proximity of the high stone tower of the Old South Church, the foundations of which had, even before the subway construction was started, settled unequally in the soft silt and clay, causing the tower to lean an appreciable amount toward the street. The following article describes the methods used to prevent further displacement during subway construction, the data upon which the article is based being taken from the discussion of a paper, "Boston Foundation," by L. B. Manley, in the Journal of the Boston Society of Civil Engineers.

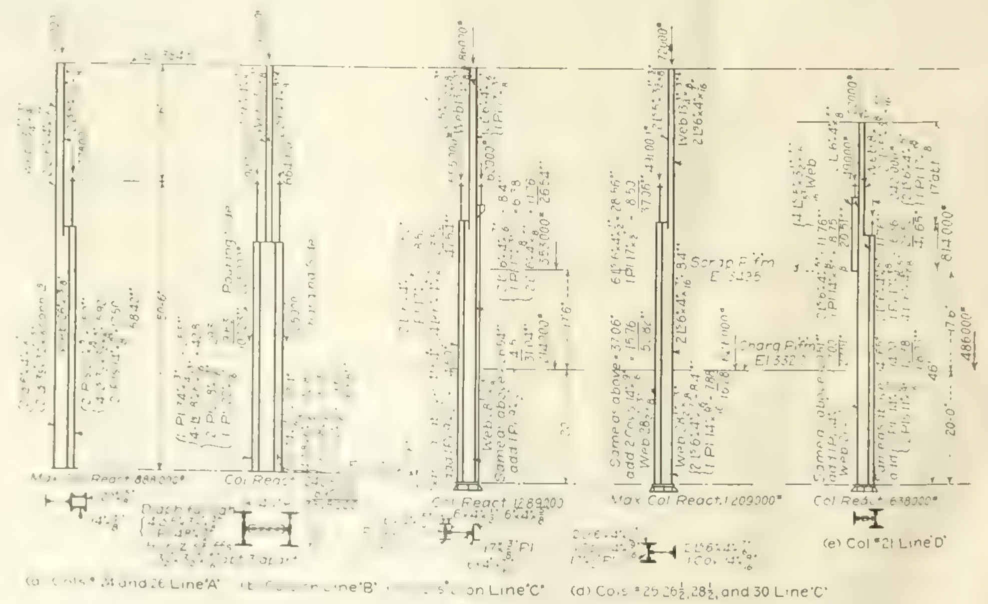

Fig. 6. Types of Columns, Loads and Sections in Open Hearth Building of Pennsylvania Steel Co.

DESCRIPTION OF TOWER.

The tower is a hollow stone shaft, 25x28 ft. in outside dimensions, with walls from 2.5 to 4 ft. thick, being connected on its north and east sides with the church building. At the belfry level, 120 ft. above the sidewalk, four corner turrets of brown sandstone are corbeled out and support the slate covered wooden roof whose peak is 231.6 ft. above the sidewalk, the peak being 246.5 ft. above the bottom of the foundations and 263 ft. above the bottom of the subway excavation. The footings are of granite, stepped out to 37x42.5 ft. They are laid in lime mortar, as is the remainder of the stone work, and rest directly on spruce piles, cut off at 3.5 ft. above mean low water.

The total weight of the tower and its foundations is about 5,000 tons, which, if uniformly distributed on its base, would produce a load of 3.18 tons per square foot. The effect of the inclination of the tower is to shift the center of loading toward the lowest corner of the base; and the inclination, combined with a wind pressure from the northeast of 30 lbs. per square foot, has been computed to give a maximum intensity of loading under the southwest corner of over 5 tons per square foot. The number and length of the supporting piles are not known, but if, as supposed, these piles are spaced 2.5 ft. on centers, this maximum loading would correspond to a load of 32.5 tons per pile.

The records show that shortly after the completion of the tower in 1875 a distinct inclination was observed which gradually increased until at the time the subway work was begun the tower leaned 2.5 ft. toward Boylston St. and 1.4 ft. toward Exeter St., the resultant inclination being about 3 ft. toward its southwest corner, or in the direction of the subway location. This movement of the peak of the tower corresponds quite closely with the inequalities of grade in the stone water table extending around the base of the tower and along the church vestry and parsonage toward the west, and it indicates an unequal settlement of the tower considerably more than that of the neighboring buildings.

CHARACTER OF SUBSOIL AND SETTLEMENTS OF SAME.

The character of the soil below the foundations of the tower is shown by the borings to be silt and clay for a depth of 115th ft. The filled material extends from 20 to 22 ft. below the sidewalk, or about 6.5 ft. below the tops of the piles. Below this gravel fill there are 16 ft. of black silt resting on pockets of sand and gravel, and below the sand there are 102 ft. of clay, hard at its surface but very soft for the remainder of the depth. Below the clay there is a layer about 4 ft. thick of hardpan, resting on boulders or rock.

The great depth of silt and clay has also caused a general subsidence of the adjacent territory. The northwest corner of the Boston Public Library, which is across Boylston St., has settled about 10 ins., and the curb between Clarendon St. and Fairfield St. has settled from 3 to 4 ins. since 1904. In Dartmouth St., where the depth of the silt layer is greatest, the curb at the southwest corner of Huntington Ave. has settled 0.46 ft. in 9 years; at Marlborough St. it has settled 1.54 ft. in 39 years; while a 6-ft. brick sewer in this street at Boylston St. was found to be 2 ft. lower than the grade at which it was set in 1870. It appears that the general subsidence in the vicinity of Copley Square has been progressing at the rate of nearly 5 ins. in 10 years.

SUBWAY CONSTRUCTION AND METHODS USED.

The Boylston St. subway, which is directly opposite the tower, consists of a two-track bore with a 20-ft. platform and a lobby for entrance stairways and ticket offices 30x55 ft. on the southerly side of the street (see Fig. 1). West of the lobby the two tracks and platform with side walls occupy a space 50 ft. wide. East of the tower the structure is widened by the addition of the north platform to 70 ft., the roof of which is supported by three interior rows of columns. The whole station is constructed of reinforced concrete and rests on a heavy invert designed to assure a uniform bearing over its entire area. The bottom of the subway structure is 30 ft. below the surface and is 27 ft. from the foundations of the tower at its nearest point.

To encourage bidding on this section of the subway the transit commission assumed the responsibility for the safety of the tower under certain conditions, and in the exercise of this responsibility its chief engineer, E. S. Davis, prescribed the methods of construc-

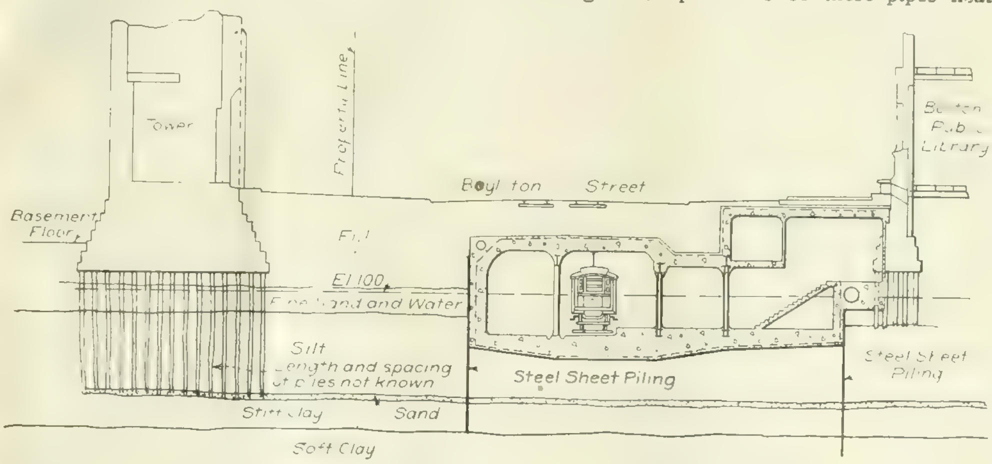

Fig. 1. Cross Section Showing Foundation of Tower of Old South Church, Boston, Mass., Adjacent Subway and Character of Subsoil at Site.

tion to be followed at this location, which methods have been successfully carried out by the contractor.

The commission finally decided not to attempt the difficult engineering feat of underpinning the heavy tower of the Old South Church, but in place of such underpinning to take the greatest care to prevent any movement of the earth which would tend to reduce the bearing power of the piles supporting the tower. The method of operation adopted was, in brief, to construct the station in short transverse slices between lines of steel sheet piling previously driven, each slice on account of the possible upward movement of the bottom of the trench being completed before the excavation for the next had reached a depth of more than 10 ft. To prevent any inward movement of the steel sheeting during the construction of the slices a self-sustainng back-wall of reinforced concrete was first constructed in a narrow trench, the timbering being so arranged as to permit the completion of the back-wall without the removal of any braces. After this wall was completed the remaining core of earth was removed, and the entire section of the subway slice was constructed in one operation. As an additional precaution the ground between the steel sheeting and the tower was filled with large quantities of neat cement grout, with the object of solidifying the sand pockets, closing underground water courses, and making the sheeting water-tight. These precautions were taken for a length of about 200 ft. on each side of the tower.

Throughout this length it was decided to rest the subway structure directly on the silt, without foundations, on account of the possible effect on the tower of the vibrations resulting from pile driving and of the risk of lowering the level of the ground water consequent upon sinking foundations to the hard clay. To prevent any lateral movement of the soil below the subway a line of steel cut-off sheeting was driven below the invert at Dartmouth St. between the rows of side-wall sheeting, and the soil inside the cofferdam thus formed was grouted with cement both before and after the construction of the subway structure.

The first operation was the driving of the steel sheeting around both sides of the subway and around the westerly side of the lobby in Dartmouth St., in a trench about 12 ft. wide and 10 ft. deep. The sheeting used was of the arched web "Lackawanna" type, having a length of 35 ft. and weighing 35 lbs. per square foot. This was driven to the top of the subway structure by means of a "Union Iron Works" No. 1 hammer, operated by compressed air, the bottom of the piling extending into the clay a distance of 15 ft. below the bottom of the subway.

The grouting operations were next begun through 2-in. pipes, spaced 10 ft. apart next to the sheeting (except in front of the church tower, where they were 5 ft. apart), the pipes being driven to the clay by aid of a water jet. Through the open ends of these pipes neat cement grout under 90 lbs. pressure was forced into the ground until it would take no more; the pipe was then drawn up 4 ft. and the grouting was continued in successive operations until the surface was reached. The pressure of 90 lbs. per square inch proved so great as to lift the walks and even to start one of the interior columns of the church, and it was afterward reduced to 50 lbs. per square inch with satisfactory results.

After the grouting was completed, the slice construction was begun simultaneously at the east and west ends of the sheet piling. The first operation consisted of constructing the self-sustaining back-wall in narrow trenches next to the steel sheeting. On the south side of the subway this back-wall carried a 36-in. sewer and was 4 ft. thick; on the north side the wall was about 12 ins. thick and was reinforced with steel rods laid horizontally. As each increment of this wall was carried about 5 ft. ahead of its corresponding slice it was supported against an inward movement by this amount of the earth core on its forward end and by the completed subway in the rear. As the earth between the back-walls was being excavated these walls were braced apart by a heavy trussed brace set at the top and forward ends to supplement the supporting power of the earth core. These supports were sufficient in most cases to sustain the back-wall during the excavation of the core and the placing of the main subway structure, but opposite the church tower ordinary timber braces were added for additional support, as the excavation was carried down.

After the excavation between the side walls was completed the concrete mat was immediately placed, and the floor and walls waterproofed. Then the invert was constructed; the side walls and structural steel were placed, and the roof was concreted and back-filled in the order named. This cycle of operations took about seven days for completion. As soon as the invert was concreted in slice No. 1 the excavation for the back-walls of slice No. 2 was begun, so that by the time the whole of the construction in slice No. 2 was completed the walls for the next slice were finished and the excavation of the core between them was begun.

The driving of sheet piling and the grouting of the soil were begun in the early summer of 1913, and the work of subway construction was started at both ends of the sheet piling during the last of June. After a short time spent in getting the organization to run smoothly, rapid progress was made, each slice, involving from 740 to 1,100 cu. yds. of excavation and from 178 to 200 cu. yds. of concrete, being completed in about seven days. After about 20 anxious weeks, during which time the tower was constantly under surveillance, the slices approached each other, and on Thanksgiving day, a day of special significance to all concerned in the work, the closing slice was substantially completed.

In consequence of this work the total settlement of the tower at its lowest point was about ½ in., and the increase of inclination at the belfry was slightly over 1 in. toward the street. During this time the use of the church was continued as usual, and no visible results of any magnitude are apparent in the building in consequence of the subway construction; in fact, on account of the solidification of the subsoil by means of the grouting it is supposed that the tower is now more stable than ever.

Topography of the Bed Rock Underlying Chicago.

Although architects, engineers and contractors usually have detailed information on the depth of bed rock under buildings with which they have been connected, their knowledge of the depth of rock under different parts of the city is generally very limited. As large buildings are being erected farther and farther from the loop district in Chicago there has arisen a demand for more definite data on the general trend of the rock topography. This demand caused Dr. Wallace W. Atwood to initiate, for the Chicago Academy of Sciences, an investigation of the depth of bed rock under the city of Chicago. Much of the information obtained is given in a paper by Roderick Peattie, presented before the Western Society of Engineers, from which the following data have been abstracted.

The investigation was started with the idea of cataloging the data from the various offices in some available form; of making a contour map of the surface of the Niagara limestone; and, if possible, of making a model for popular exhibition. The map was to be not merely a collection of the data, but it was hoped that, by means of it, it would be possible to anticipate the depth of rock in parts of the city where no caissons had been previously sunk. This was to be accomplished by the map maker through the interpolation of contours from known regions surrounding the district, and by a knowledge of the general behavior of contour lines.

The catalog of well borings and caisson borings has materialized in the form of a card index (a copy of which is in the library of the Western Society of Engineers), which contains about 2,000 records. These are, however, the averages of many records. Where a building had a number of caissons, all within a short distance of each other and not varying greatly in depth the record is merely an average of the lot. As the purpose was to discover the depth of rock under the adjacent piece of land only the general trend was of value. The records are catalogued by streets, as no other system is intelligible to the layman. The street farthest north had its data filed first; where there were several drillings on one street the drilling farthest west received first attention.

Figure 1 shows an outline map of Chicago on which have been plotted the elevations of bed rock at various points. These elevations were compiled from data on file with the Western Society of Engineers, the Bureau of Engineering and the Board of Local Improvements.

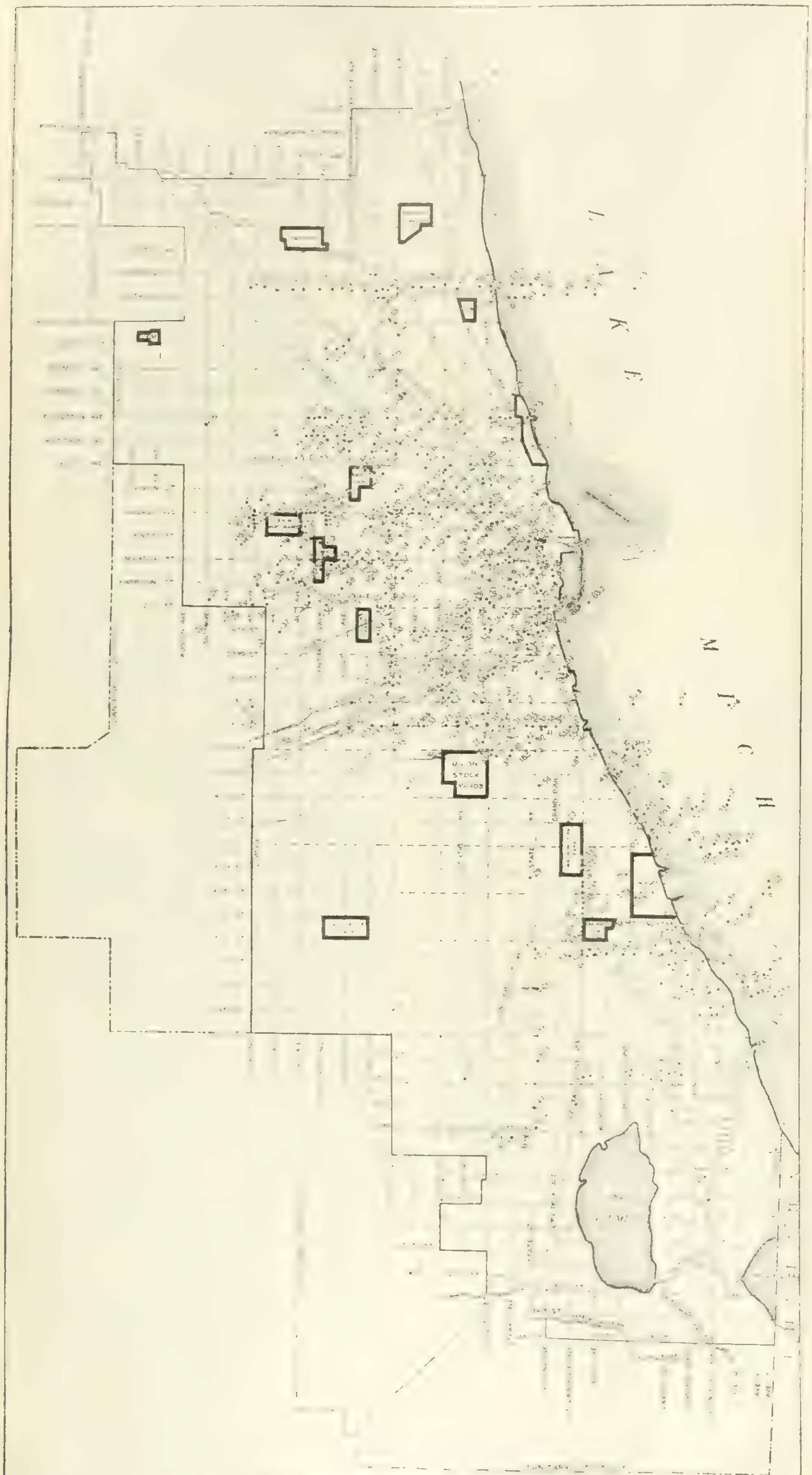

Fig. 1. Map Showing Elevation of Bed Rock at Various Points in Chicago—All Elevations Referred to City Datum.

The contour map (which is not yet available) will probably be of greater value than either of the other forms. This map when finished will have a contour interval of 10 ft. Data are not at present numerous enough to allow the use of a 5-ft. contour, and it is doubtful that this will ever be the case, with the exception of the loop district and perhaps some of the surrounding manufacturing districts. The map is accurate, so far as it goes, but there are details in the contours that are not recorded here. This is proven by the many irregularities in the contours of the loop section where more information was available. The writer can see no reason to expect that the other level lines will be more nearly regular, except those west of the divide, which would, of course, have fewer deviations owing to the milder slopes.

Nearly 70 firms were visited. About 40 of these firms were kind enough to furnish data, and some went to considerable trouble to obtain it. Some firms refused to furnish the information because they considered it their stock in trade, and others did not think it worth while to look up their records. They did not comprehend that to pool their data with other offices would be of mutual benefit, for the data would be not only a source of general information for their estimators, but would result in actual money returns to them. The map will perhaps never be finished, but it will be revised from time to time as more data are accumulated. The more detailed it becomes, the more valuable it will be, and it is only through co-operation that it can be developed.

Referring to the description of the surface of the rock the zero contour represents the city datum. (City datum corresponds to the low water level of 1847. It is about 14 ft. below the surface in the down-town district.) A divide starts at the north end of Lincoln Park at the height of about —20 ft. The col or pass between this and the next hill of the divide it as low as —65 ft. The peak of the next hill (Kinzie St. and Western Ave.), reaches the greatest height of any of the rock hills. The rock here is 25 ft. above the level of the lake. The divide then swings about to a smaller hill, whose apex at Cicero Ave. and 31st St. is 10 ft. above datum. The watershed then goes southeast until at 63d St. and Ashland Ave., at about 25 ft. in elevation, it turns and runs southwest to the corner of the area mapped. To the west of this divide there is a gently sloping area which, as far as can be discovered, never reaches a greater depth than 50 odd feet below datum. A great deal of this western region is mapped on insufficient data, as a few isolated factories and the drainage canal were the only sources of information. For the existence of such a contour as the 40-ft. line directly west of 55th St., the writer has no real justification, but from the shape and proximity of the other contours he is led to believe that the 40-ft. contour is there. Another section where data are lacking is the one at the extreme southwest corner of the map. Here the records show a canyon 60 ft. deep. From the narrowness of the canyon, its shape, and sudden termination (this canyon has been followed beyond the limits of the map), together with the uncertain accuracy of the data, its existence is doubtful.

The factories of South Chicago, and the soundings made by the U. S. War Department and the Illinois Steel Co., in making boat slips in Calumet Harbor, furnished the writer with ample material for making accurately the contours of the next hill north. The rock comes to the surface in one section here, which facilitated the mapping. The well-known outcrop at 79th St. and the lake, and the old quarry at 75th St. and Railroad Ave. three blocks back from the lake, mark the top of a hill. Four blocks north, at the water tunnel at 73d St., the rock dips until it is 52 ft. below datum. This tunnel, which continues west to State St. and thence south to 104th St., has furnished data where they would be otherwise wanting. The depression to the north continues to deepen until it reaches the —90-ft. contour. This contour has eaten back until it crosses Cottage Grove Ave. An arm of this valley, marked by the —70-ft. contour, reaches to the north as far as 63d St. at Stony Island Ave. To the north of this there is a pass into the next valley which is 25 ft. below the surface. To the east of it is a hill, the top of which is only 10 to 20 ft. below the surface of the lake. The present Field Columbian Musuem records itself as being on the side of a hill, for its well record reads 65 ft. to rock, and 1,000 ft. to the east the rock is only 15 ft. below the surface.

The next district of numerous data is the Stock Yards. From thence on, the data are excellent, both in quantity and in accuracy. The surface of the rock descends with various irregularities to the depth of 120 ft. below datum. The 120-ft. ancient stream bed comes under the city at about the river mouth and goes as far as the corner of Franklin and Randolph Sts. The greatest depth the writer has discovered for the rock was a well at Union and Randolph Sts. This was 132 ft. below the surface. The data in the bottom of this gorge, above which the loop district stands, are almost sufficient to permit the construction of a 5-ft. contour map.

ROADS AND STREETS

Traffic and the Methods and Cost of Road Maintenance in Massachusetts and a Comparison With English and French Conditions.

Maintenance methods have perhaps reached their highest development in this country in the state of Massachusetts. In a paper before the Fourth American Road Congress W. D. Sohier, chairman of the Massachusetts Highway Commission, summarized existing conditions and his paper is given here. It should be borne in mind that proper maintenance implies that the road is kept continually in as good condition as when it was first completed. The life of a properly maintained road is, generally speaking, unlimited, replacement or reconstruction of the wearing surface being an economic factor which is dependent upon annual maintenance costs.

If the road is to be properly maintained, it must in the first instance be properly constructed. The materials and methods used must be adequate to withstand the traffic that goes over the road without serious deterioration in a few months or even a year or two. This means that the road surface as much as the bridge and the road foundation as much as the bridge foundation must be able to stand without being destroyed the heaviest moving load that is going over it.

I am giving a number of tables showing the cost of maintenance in France and England and Massachusetts, with some tables on traffic that will illustrate this point. We can well learn something from the experience of other countries where road building has been a science for nearly 70 years in France and for the last 25 or 30 years in England.

MAINTENANCE OF FRENCH ROADS.

They established their French road system beginning in 1826, and constructed their total road system of 371,000 miles in about the next 25 years.

Their roads, practically uniform except as to width, have been built almost entirely of local macadam originally, 6 or 8 ins. deep on a proper foundation. In resurfacing some of the main roads in later years they have used a harder stone and Welsh or Belgium granite. The macadam surface of the road on the Routes Nationales is 24 ft.; Routes Departmentales. 18 ft., and the roads de grande communication and d'interest commun, 15 ft. The yearly cost of maintenance has been $273 a mile on the Routes Nationales, this maintenance cost varying to $78 a mile on the 184,000 miles of ordinary country road of only local interest.

The French engineers last spring estimated that some 8,000 miles of road ought to be resurfaced, using a tar macadam, because of the large increase in automobile travel around the cities, which travel was rapidly destroying their macadam roads, and they requested the government to furnish $60,000,000 for that purpose, about $7,500 a mile.

Their greatest problem for over 50 years has been maintenance. The roads are maintained practically by a central organization. The whole country is divided into 86 departments, and all of the county and rural roads within the department are managed by the prefect of that department, and the expenditures appropriated by the council.

Direct charge is in the hands of a centralized body of competent engineers, about half of whom are graduates of the National School.

Each department is divided into four or five political districts, each district being called an arrondissement, and the roads are in charge of a district engineer, who is under the direction of the chief engineer. Each arrondissement is again divided into districts or cantons, and an assistant road engineer under the direction of the district road engineer looks after all the county and rural roads within the canton. Then comes the final subdivision, where the roads are divided into sections of a few miles long, taken care of by patrolmen. All of these are under civil service and the men are promoted from time to time according to their ability.

Table I gives the approximate cost of maintenance on the French roads, annually.

Five or six patrolmen are under a foreman who is also a patrolman. When any resurfacing or reconstruction is to be done they use machinery, rollers, etc., belonging to the department and collect together enough of the patrolmen, with a section foreman, to reconstruct the road under the supervision of the district engineer. This provides them with men who are thoroughly familiar with the work.

TABLE I.—APPROXIMATE ANNUAL COST OF MAINTAINING FRENCH ROADS.

	Miles.	Total expense.	Per mile.
Routes Nationales	23,800	$6,500,000	$273
Routes Departmentales	8,000	1,500,000	185
Chemins Vicinaux 115,400			
De grande communcation	107,300	16,900,000	157
D'interest commun	47,500	6,000,000	126
Ordinaires	184,700	14,500,000	78

You will note that the French engineers state that on the main roads near the cities the $273 a mile, a year for maintenance is not keeping the roads up; that they need $60,000,000 to resurface with tar macadam 3,000 miles of these roads at a cost of about $7,500 a mile; that is, practically one-third of their main roads should be of tar macadam. If this expenditure is made in say a five-year period, or $12,000,000 a year, it will increase the cost of maintenance on their 23,800 miles of national roads about $500 a mile a year for resurfacing on the average, and will make their total maintenance charge on their main roads $770 a mile in place of the present cost of $273 a mile. Of course, if spread over a ten-year period it would only increase the cost half as much, the present roads would be practically impassable. This is in accordance with our Massachusetts experience on our state highways where our expenses per mile including resurfacing averaged over $850 in 1913.

MAINTENANCE OF ENGLISH ROADS.

Originally there was no road system in England. Everyone made his own road. Then came "statue labor" which was required by a general act passed in 1555. This practice was not superseded by "highway rates" until 1835. The roads were cared for by the parishes, and a little later several parishes were combined in a highway district.

In 1663 England began to pass "Turnpike Acts," authorizing the creation of a corporation with trustees, who were to build roads, maintain toll gates and charge tolls.

By 1833 Parliament had passed 3,800 Turnpike Acts, and created in England and Wales 1,116 Turnpike trusts, controlling 22,000 miles of road. They almost all failed and in the 22 years after 1864 the number of Turnpike trusts was reduced from 1,048 controlling 20,589 miles of road to 20 trusts with 700 miles of road.

In 1878 the cost of these main roads which had been disturnpiked was placed upon the counties. In order to even up the expense more or less the English government made appropriations to aid in the maintenance of these main roads, beginning in 1882 with an appropriation of about $800,000. In 1888 about $2,500,000 was appropriated, and now the Road Board has something over $6,000,000 annually which can be spent in improving the main roads.

They have now a combination of the county taking care of the urban and rural main roads, with the parish and local authorities taking care of the rural roads. The main roads are something over 20 per cent of all the mileage, leaving out London.

In almost all of the counties they have sections of road in charge of regular maintenance men. They have almost universally been obliged to give up the old system of patching the roads by putting in loose broken stone from the roadside because of the large number of automobiles that threw it out so rapidly, and they have substituted patches made of either tarred stone or tarred slag, or the patches are made by a painting method using stone chips and dust.

Most of the main roads in England outside of Metropolitan London and the other large cities, are maintained by the use of tar. Some 40,000 miles of road were tarred in England last year, and some 6,500 miles were built of tarred macadam. Their system of maintenance now is not only to keep the roads constantly patched, but every year or twice a year on their macadam roads they usually flush them and roll them filling the holes and depressions first and adding a small quantity of chips and stone dust. On the tarred roads a section man keeps them constantly patched. They usually require a fifth or a sixth of a gallon of tar per square yard which is sprayed on once a year, and this is covered with pea stone or gravel and kept covered so it won't pick up. The tar is usually sprayed on under pressure.

Where they have heavy traffic they are resurfacing their roads with tarred Welsh granite which is like our trap rock or with tarmac which is a tarred iron slag. They also build a road of three-inch stone, rolled hard, and grout it with a mixture of hot sand and hot tar, equal volume, poured into the road until it flushes it, and roll in 1½-in. stone and smaller stone with a surface coat of tar and sand. They find their macadam roads with a tarred surface require re-treatment every year. This costs about 2½ to 3 cts. a square yard a year. On the tar-mix roads they require a new coat of a fifth of a gallon of tar applied on the surface every two years.

In Table II is given the cost of maintenance on the various classes of road in England, in which you will see that the average cost is $1,100 a mile a year to maintain the county urban main roads, and the county rural roads $431 a mile a year, while the rural roads which are merely of local interest cost about $122 a mile a year to maintain.

CONTINUOUS MAINTENANCE.

In both France and England they keep the roads constantly patched. But in France near the large cities where there is a large amount of automobile travel, they have been unable to keep their water-bound macadam roads in good condition. Many of them are rough with a large number of pot-holes. Around Paris they have replaced the water-bound macadam with a tar macadam top 3 ins.

TABLE II.—ANNUAL COST OF ROAD MAINTENANCE IN ENGLAND.

	Miles.	Maintenance.	Authorities, county engineers.	Yearly maintenance per mile.
County councils:				
Urban main roads.	1,189	$4,601,790	61	$1,110
Rural main roads..	23,565	10,177,740	....	431
	27,754	$14,779,530	....	....
County boroughs...	9,366	$6,437,380	28	$ 685
London authorities.	2,192	3,691,355	2	1,680
	11,558	$10,128,735	30	....
			Road authorities.	
Urban roads.......	11,411	$4,848,020	1,733	$425
Urban roads.......	4,871	2,701,710	....	555
Rural roads........	95,077	11,562,920	....	122
	111,359	$19,112,650	....	....
Totals	150,671	$44,020,915	1,898	$290

The total expenditures per year, including improvements and interest, $75,990,000.

in depth on many of the roads that were undoubtedly worn out and rough and impassable five years ago.

In both countries on the main roads they keep section men constantly patching the roads. They add a little additional stone, flush them out and roll them every year. On the main roads where there is much automobile travel they are using tar in making their patches, as they found that the old method of putting on loose broken stone and letting the traffic roll it down is of no use because the automobiles throw the stones out over the road.

They are patching by two methods, more commonly using a tarred stone or tarred slag with tarred chips, cutting a square edge if there is a pot-hole and tamping the tarred material in. The largest stone used are at least three-quarters of the depth of the hole. In other places they are merely painting the little place where the tar fails and covering with pea stone or pea gravel.

In resurfacing in both countries they are using a large stone about the 3-in. size. In France where there is not a great deal of tarred macadam they are using a hard Belgium granite on the main roads, doing one-half the width at once.

In England they are using the same large size stone coated with tar or a tarmac (tar-coated iron slag). Most of their engineers use very little small stone with it. They use practically all large size with tarmac as it compacts under the roller. Some of the county engineers use 10 per cent of the fine material. They also do one-half the road at once. The engineers state that they have been forced to use this larger stone in resurfacing because of the motor trucks and the large number of traction engints with trailers. Our practice in Massachusetts is the same, except that we are very often on heavy traffic roads using asphalt or some asphaltic bitumen rather than tar because our experience leads us to believe that it is worth the additional cost for the material.

We are also in Massachusetts—somewhat experimentally as yet as we have only done three or four miles—resurfacing with the tar and sand mixture grouted into the three-inch stone described above.

In England, France and Massachusetts we have found it necessarv and economical with the change and increase in traffic to very much diminish the crown of our macadam roads.

Formerly with macadam surfaces 15 ft. in width we used a crown of ¾ in. to the foot. We now try to secure about ¼ in. to the foot on our bituminous macadam roads, and in resurfacing the old roads we are widening the macadam surface to 18 ft. in place of 15, as our experience shows that traffic otherwise will spread over the edge of the road and rapidly sheer down into the macadam and narrow the road up.

In England they have been forced to do the same; to wit, widen the road and diminish the crown, because they found that the traction engine with trailers, of which they use large numbers, would shear down into the macadam, thus rapidly destroying their older roads. They now universally use a crown that does not exceed one inch to the yard.

The result of this in England and Massachusetts has been that the traffic has spread all over the road, that no rut has developed and no horse track, a tremendous change from a few years ago when the center of the road as a horse track wore down quicker than the sides. In traveling over 2,000 miles of road in England this year and last, I didn't see a single rut and practically not a single pot-hole.

MAINTENANCE OF STATE HIGHWAYS IN MASSACHUSETTS.

Our commission began building roads in 1894. The earlier roads were almost entirely macadam with a few miles of gravel or graded road. The standard road with necessary foundation and proper drainage wherever necessary was 15 ft. in width, water-bound macadam either of trap or local stone, 6 ins. deep in the center and 4 ins. on the sides, with a 3-ft. gravel shoulder on each side and with a ¾-in. crown to the foot.

The ordinary cost of maintenance is given in Table III, but up to 1907 when some of the roads were 12 years old, the cost of ordinary maintenance was substantially $100 a mile a year. Ordinary maintenance with us consisted merely of keeping the gutters, catch basins, and drainage, open and clean, cutting out the grass and brush on the roadsides, keeping the shoulders in proper condition, spreading a little gravel or sand on the road surface from time to time and filling the few holes or ruts that might occur, with broken stone or gravel. Very few miles of road had been actually resurfaced prior to 1907.

In 1906 the automobiles began to come. Our roads were some of them 13 years old and only half the original depth of stone was left. We soon found that automobile travel, especially at high speed, disintegrated or tore up the macadam or gravel roads, especially on the curves, as soon as there were any considerable number, say 50 or more in a day. The traffic, of course, increased tremendously in the number of vehicles, because of the large mileage of the automobiles. What had been country roads developed, between that year and the present year, into main through routes carrying oftentimes away out in the country on a main route over 1,000 cars a day.

TRAFFIC AND THE COST OF MAINTENANCE.

In connection with maintenance traffic is of vital consideration. In Table IV is shown the average traffic on the Massachusetts state highways in 1909 and 1912.

TABLE III.—COST FOR CONSTRUCTION, REPAIR AND MAINTENANCE OF STATE HIGHWAYS, FROM 1894 TO 1913.

	Repair and maintenance.		State highways		
Year.	Cost.	Miles.	Av. cost per mile per year.	Miles laid out.	Cost of construction.
1894	....	......	......	39.88	....
1895	....	......	......	50.03	$ 637,847
1896	$ 4,727	89.10	$53.05	37.02	458,581
1897	13,267	126.01	105.28	53.25	482.076
1898	20,661	179.26	115.26	42.68	499,783
1899	24,538	221.94	110.56	44.56	407,309
1900	33,562	266.50	125.93	49.40	396,459
1901	31,061	315.90	98.32	61.68	453,826
1902	59,943	377.58	158.75	53.32	466,743
1903	55,083	430.90	127.83	74.17	443,972
1904	51,896	505.03	102.76	60.85	445,745
1905	57,456	565.88	101.53	56.55	509,007
1906	68,382	622.45	109.86	47.92	444,655
1907	106,189	670.37	158.40	39.33	467,944
1908	147,037	709.70	323.47	38.40	564,719
1908	92,628*	......	......	.....	....
1909	247,985	748.27	537.39	36.53	431,814
1909	154,131*	......	......	.....	....
1910	214,561	784.80	642.28	52.80	462,165
1910	289,498*	......	......	.....	....
1911	213,476	837.59	632.86	42.00	412,542
1911	316,603*	......	......	.....	....
1912	208,687	879.59	708.39	40.72	366,424
1912	414,407*	......	......	.....	....
1913	203,762	920.31	868.13	60.06	909,063
1913	595,183*	......	......	.....	....
....	....	......	......	980.88	$9,262,674

Average cost of repair and maintenance, 1895 to 1907, inclusive, $105 per mile per year.

Average cost of repair and maintenance, 1908 to 1913, inclusive, $619 per mile per year.

Average cost of repair and maintenance, 1895 to 1913, inclusive, $267 per mile per year.

*Motor vehicle fees fund.

ROAD MAINTENANCE.

Increases and Changes in Traffic from 1909 to 1912.—In Massachusetts the traffic using our roads is constantly increasing, but it is changing much more rapidly than it is increasing. This is conclusively shown by Table IV.

Table V shows the traffic on certain roads at night. We had a count made for 24 hours a day at a few points, and the result may be interesting.

I computed several night and day counts for the two years to get an average, and found that on the average the night traffic constituted about 18 per cent of the total traffic; consequently, one should add about 22 per cent to the 14-hour day count to ascertain the total number of vehicles.

Pleasure Traffic Around Boston.—The census near Boston in the parks may be in-

TABLE IV.—AVERAGE TRAFFIC ON MASSACHUSETTS ROADS IN 1909 AND 1912.

	1909 census, 238.5 stations.			1912 census, 156.5 stations.			
Kind of vehicle.	Av. total No. per day.	Av. No. per day per sta.	% of No. per sta.	Av. total No. per day.	Av. No. per day per sta.	% of No. per sta.	Increase or decrease, %.
Motors—							
Runabouts	4,958.5	20.8	8.5	5,819.0	37.2	11	+ 79
Touring cars	17,950.5	75.3	30.5	27,178.5	173.5	49	+130
Trucks	....	.....	....	1,800.0	11.5	3	.....
Total motors	22,909.0	96.1	39.0	34,797.5	222.2	63	+131
Horse-drawn vehicles—							
1 horse, light	17,033.0	71.5	29.0	8,380.0	53.5	15	— 25
1 horse, heavy	11,762.5	49.3	20.0	7,458.0	47.6	14	— 3
2 or more horses, light........	1,006.0	4.2	2.0	556.0	3.6	1	— 14
2 or more horses, heavy........	6,205.5	26.0	10.0	3,870.5	24.7	7	— 5
Total horse-drawn	36,007.0	151.0	61.0	20,264.5	129.4	37	— 14
Total of all kinds........	....	247.1	....	....	351.6	..	+ 42

TABLE V.—DAY AND NIGHT, 12 HOURS EACH—OCTOBER, 1912.

	Automobiles. Day.	Automobiles. Night.	Vehicles (all kinds). Day.	Vehicles (all kinds). Night.	Total vehicles.	Percentages at night.
Lexington	302	59	438	104	542	19
Watertown	273	72	671	141	812	17
Chelsea	103	10	358	53	411	13
Somerville	266	70	689	231	920	25
Boston	358	69	513	94	607	15

teresting, but it must be remembered that it is, in many instances, light pleasure traffic. The figures from a census taken in August, 1912, show that with a total number of vehicles varying from 1,109 to 3,009 from 60 to 97 per cent were motor vehicles, the average being about 90 per cent.

METROPOLITAN PARKS (MOSTLY PLEASURE VEHICLES).

Location.	Total of all vehicles.	Motor vehicles.	% motor to total traffic.
Lynn, Prescott Pl. and Shore Res.	1,530	1,411	92
Revere, Saugus River Bridge	1,872	1,808	97
Somerville, Alewife Bridge	491	474	97
Medford, Parkway and Main St.	515	492	95
*Somerville, Welling Bridge	2,526	2,174	86
*Milton, Mattapan Bridge	2,383	1,717	72
Medford, Malden River Bridge	1,884	1,848	98
BOSTON PARKS (ALL CLASSES OF VEHICLES.			
Prince St., Jamaica Plain	1,934	1,715	89
Commonwealth Ave., a city residential street	3,009	2,634	88
Washington St., a suburban city avenue	1,109	671	60

*All classes of vehicles.

Motor Vehicles in Massachusetts.—Table VI shows the number of motor cars registered in Massachusetts from 1906 to 1914, inclusive. You will note that the number of automobiles registered has increased from about 7,300 in 1906 to over 84,000 in 1914, and fully one-third of the traffic on our main roads consists of automobiles from other states. You will note also that there were less than 1,000 trucks registered in 1909 and five years later in 1914 there were 8,000—8 times as many.

TABLE VI.—STATEMENT SHOWING THE NUMBER OF MOTOR CARS REGISTERED AND LICENSES ISSUED, 1906 TO 1914.

	1906.	1907.	1908.	1909.	1910.	1911.	1912.	1913.	1914.
Autos (pleasure)	6,572	7,733	18,066	23,011	29,792	36,284	46,096	56,712	68,100
Dealers' autos	755	455	1,905	2,455	3,305	4,920	6,301	7,462	7,898
Trucks	.....	.....	.....	960	1,568	2,623	4,036	5,948	8,053
	7,327	8,188	19,971	26,426	34,665	43,827	56,433	70,122	84,051
Operator and chauf.	10,083	10,696	13,170	18,251	41,259	51,950	66,645	81,034	95,577
Total receipts	$33,085	$92,096	$121,488	$169,973	$374,789	$477,417	$616,245	$764,153	$914,119

Prior to 1907 certificates of registration did not expire annually. Prior to 1909 trucks were not classified.

Between 1903 and 1907 all automobile registration fees were $2.

In 1907 the automobile registration fee was increased from $2 to $5. In that year 9,006 cars, registered at $2, were re-registered in the same year at $5.

Beginning January 1, 1910, the automobile registration fees were based on the horse-powers of the vehicles, the fees varying from $5 to $25. The fee for registration of a truck, however, was $5 regardless of the horse-power.

Prior to 1910 operators' licenses did not expire annually, but continued in force indefinitely. Since 1910 all licenses have expired annually.

Increase in Maintenance Cost.—Starting in 1907 you will note that our cost for maintenance has risen by leaps and bounds from $158 a mile a year in 1907 to over $850 a mile a year in 1913, and it is still higher in 1914.

Our roads were rapidly going to pieces. We needed some money. We got the legislature to double its appropriation of $100,000 a year and make it $250,000 for one year and $200,000 a year since. We secured an increase in the automobile fees, and had four-fifths of that money available for the maintenance of state highways, and the other one-fifth for the improvement or maintenance of through routes in the towns.

We found that our old roads were being destroyed by the rapidly increasing amount of automobile travel. The traffic on the roads had increased from 10 to 40 times in volume.

Our roads, both gravel and macadam, were rapidly being torn up by the automobiles and deposited in dust over the country. They were rutted, pot-holes developed, and, as you see, we had to increase our maintenance cost. The question was, how to best preserve our old roads.

In 1907 we began to use a bituminous material as a surface coat. We used refined tar and a heavy cold asphaltic oil, applying about one-half gallon to the square yard.

Proper Methods for Economical Maintenance.—In passing let me say as a result of our experience in the use of bituminous binders on road surfaces, that we invariably true up the surface and patch the holes and ruts first. We invariably thoroughly brush and clean the road down to the stone or hard gravel. We invariably spread our bitumen, whether tar or asphaltic oil, evenly and under pressure. We invariably cover it and keep it covered so that it will not pick up, brushing the cover back from time to time when necessary. We cover it with pea-stone and dust, unless we can secure a good pea-stone in sand or gravel, or we coat it when the other method is too expensive and traffic is light, with a coarse sharp sand. Thereafter, constantly and eternally, we keep the road and road surface patched, using in patching substantially the same bituminous material that was used for the surface coat.

We used ½ gal. per square yard of heavy asphaltic oil that had to be heated to 250° F. spread upon the road. We used the same quantity of the heaviest oil that could be spread cold. Where we had only money enough for a dust layer, we used ⅕ to ¼ gal. of light asphaltic oil per square yard, this oil sometimes being called a 40 per cent oil. We used ½ gal. of hot refined tar, and we have used after the first application ¼ gal. of the same tar yearly. We have used water-gas tar and various proprietary materials of a bituminous nature known by various trade names. In every instance that has succeeded the road has always been properly cleaned and patched beforehand, and has always been covered and kept sufficiently covered to prevent its picking up. It has been constantly patched.

Surface Coatings of Asphaltic Oils, Tars, etc.—Today nine-tenths of all our state highways that have not been resurfaced have been coated and kept coated with some bituminous material and have been kept constantly patched. Where the heavy hot oil was used, it has lasted in some instances for five years, carrying a large amount of automobile travel but a small amount of heavy teaming. Under many heavy teams it has failed in a month or two. We have then used a light oil to lay the dust and prevent the automobiles from tearing the road up, and have left the stone to carry the travel until we could resurface the road.

Because of the automobile traffic on many main routes, we are now using in our resurfacing a bituminous top two to three inches deep. We have used refined tars by both the mixing and grouting method. We have used asphalts with the same methods, and we believe that use is economical and necessary on any roads that have more than 50 automobiles a day and where there are more than 50 loaded teams.

I am giving a table showing the class and character of surfaced roads that we believe will stand traffic of a certain class, kind, and character—we believe it will prove economical and satisfactory with the traffic.

Of course, in connection with this table it is absolutely essential that the drainage and foundation are sufficient and the material used is strong enough to carry the heaviest load which goes over the road without the road's being rapidly destroyed.

Materials That Have Not Failed.—Table VII has been somewhat changed as the result of our experience since 1912 when I first published a like table. It expresses the consensus of opinion of our chief engineer and four division engineers and my own best judgment.

The results have all been obtained on many miles of road where we have used a good grade of asphaltic oil, either hot or cold, heavy or light, or a good grade of refined tar.

We have had many failures on short sections of road where a non-asphaltic oil was used or a poor grade of oil or tar, and many proprietary so-called dust-layers have failed. Roads that failed have been resurfaced or retreated, but the results are not tabulated in the above table.

Table VII relates merely to the maintenance of gravel or water-bound macadam roads, not to bituminous macadam. It represents our average experience on many miles of road at over 150 observation points.

Certain exceptions should be noted. Army maneuvers, especially large bodies of cavalry and artillery, will rapidly destroy any bituminous blanket surface. A blanket coat of hot oil on macadam will carry a much larger number of teams if there is a ratio of two to three automobiles on pneumatic tires to each team to keep the bituminous surface constantly rolled down when the horses and teams pick it up. But note that a very few teams on narrow tires, or a few very heavy teams every day, will destroy the surface if the load is heavy enough to shear down entirely through the surface to the stone. If this process is repeated once or twice a day, a rut soon develops and the road becomes muddy and the bituminous surface rapidly disintegrated. Light oil or cold tar will then be more serviceable, laying the dust while the stone takes the wear.

We have maintained a few miles of road in reasonably satisfactory condition with annual applications of a cold tar or water gas tar. They have required ½ gal. per square yard annually, and the results have been about the same and certainly no better than where we have applied two ¼-gal. coats per square yard of light asphaltic oil the first year and ¼ gal. per square yard each succeeding year. The cost for the cold tar has been more.

Invariably we clean and patch the road first and cover the bituminous material sufficiently to prevent its picking up. We have sometimes tried dispensing with the cleaning and covering but shall not repeat that expensive experiment. We can usually have the light asphaltic oil sprayed onto the roads by motor trucks for 1.2 cts. to 1.5 cts. per square yard, using ⅕ to ¼ gal. per square yard. The cleaning, patching and covering costs about the same.

Maintenance Methods and Costs.—On gravel roads we have found it necessary to keep them constantly shaped and patched. We drag some of our roads once a week where there are 150 or more automobiles a day. We have found on the hills that have rutted with that traffic that it was more economical to use ¼ gal. per square yard, of light asphaltic oil, using the first year two applications and in subsequent years one, keeping it properly covered. Roads of this character have been extremely satisfactory with the traffic of a very large number of automobiles, certainly 500 a day, but they will not stand a large amount of heavy horse-drawn vehicles and probably will not stand many loaded trucks.

As you will note from Table III, our ordinary maintenance which has consisted of keeping the drainage open, shoulders cut back, and the road sanded, with slight patching, has averaged about $100 a mile a year. Where the road can be maintained with light oil, the additional cost is about $250 a mile a year, or $350 a mile a year in all. The cost of heavy oil or tar on the surface where it is suitable to use it because it will stand the traffic, is about twice the cost of the light oil in the first instance, and with the patching and all that is necessary it will cost about the same figure, or about $250 to $500 a mile a year, this being on a five-year basis.

In resurfacing our roads with 2 to 3-in. bituminous macadam top, the cost has varied from about 50 cts. a square yard for a 1¼-in. top no stone being larger than 1¼-in., and 1¾ gals. of tar sprayed into it, to $1.10 a square yard for a 3-in. top, made of 2½-in. stone, either mixed or grouted with a good

TABLE VII.—AVERAGE DAILY TRAFFIC LIMITS IN MASSACHUSETTS.

Table showing results of observations of traffic on different types of road surfaces in Massachusetts—Standard road, 15 ft. in width; gravel or waterbound macadam, 5 to 6 ins. in thickness, with adequate drainage and proper foundation, with 3-ft. gravel shoulder on each side.

	Light teams, carriages, wagons.	Heavy teams, 1-horse.	Heavy teams, 2 or more horses.	Automobiles.
A good gravel road will wear reasonably well and be economical with	50- 75	25- 30	10-15	50 to 75.
Needs to be oiled with	50- 75	25- 30	10-15	Over 75.
Oiled gravel, fairly good, heavy cold oil, ½ gal. to the square yard, applied annually with	75-100	30- 50	20	500 to 700 or more.
Waterbound macadam will stand with	175-200	175-200	60-80	Not over 50 at high speed.
Cold oil or tar will prove serviceable on such macadam with	175-200	175-200	60-80	50-500.
Macadam will then stand but the stone wears, of course, with	175-200	175-200	60-80	500 or more.
Waterbound macadam with hot asphaltic oil blanket will be economical with	100-150	50- 75	25-30	1,500 and more with fewer teams.
And stand at least	....	....	.....	50 trucks.
But will crumble and perhaps fail with over (On narrow tires, ice, farm and wood teams, etc.)	150	75	30	
Waterbound macadam with a good surface coating of tar (½ gal. to the sq. yd.) will stand with (But requires to be recoated annually with ¼ gal of tar per sq. yd.)	100-150	50- 75	25-30	1,500 or more.

It is assumed that all road surfaces are kept constantly patched, that before applying bitumen the road surface is cleaned and patched, and the bitumen covered with pea stone and sand or gravel and kept covered so that it never picks up.

grade of asphalt. The tar-sand grouted macadam has cost from 90 cts. to a dollar per square yard. We believe that the roads of this character will have a very small maintenance cost outside of the ordinary maintenance for a period of from eight to twelve years.

TRAFFIC AND MAINTENANCE.

I am inserting some tables showing the maintenance cost on certain roads compared with the maintenance cost in England on roads of the same character; to wit, water-bound macadam, giving the cost per mile and the cost per ton mile for each vehicle that is going over them. This figured upon the following formula, which is substantially the same in England and France, showing the assumed average weight of vehicles in tons.

Motor vehicles—	
Runabouts	1.43
Touring cars	2.23
Trucks	6.25
Horse-drawn vehicles—	
Light vehicles, 1 horse	0.36
Heavy vehicles, 1 horse	1.12
Light vehicles, 2 or more horses	0.54
Heavy vehicles, 2 or more horses	2.46

The cost in Massachusetts per vehicle per mile as shown by our maintenance cost and traffic census taken in three-year periods has been substantially one cent a vehicle per mile until we began to use bituminous binders. It now figures about 8/10 ct. a vehicle a mile.

It may be stated that traffic and maintenance must be considered together, i. e., quantity and character of traffic, economical maintenance and cost thereof, materials and methods to be adapted to the traffic that the road has to bear.

English Roads.—There is a very interesting and useful paper on construction and maintenance published in connection with the International Road Congress in 1913, this paper giving the experience of five of the most competent road engineers in England. Table VIII shows the weight in traffic in tons carried one mile for one cent of maintenance cost.

I have used the English long-ton but have changed the pennies to cents. This table shows that the cost on water-bound macadam roads to carry one ton one mile over the road, varies from about ¼ ct. to 1½ cts. for the maintenance. This illustrates very well, I think, the necessity of the traffic census showing the class and character of the vehicles which use the road rather than any formula which merely uses an assumed weight for each class of vehicle.

Massachusetts Experience.—We have found on our roads in Massachusetts that the largest cost of up-keep on macadam roads could come from two causes; first, automobile travel, which would disintegrate a plain gravel or macadam road in a month if there were many automobiles. This could be prevented by the use of some bituminous material on the surface, provided the team traffic was not so heavy that it would quickly wear the binder out. A large number of heavily-laden teams, of say three tons or more, would wear the binder out in a very short time. Also, a very large number of heavy loads carried on narrow tires would wear a macadam road out relatively quickly. Some of the roads that I am showing would be worn from ½-in. to 1-in. a year if of macadam, whereas when a bituminous macadam road with a 3-in. top

TABLE IX.—TABLE GIVING PARTICULARS OF EXPERIENCE OBTAINED IN LIVERPOOL WITH DIFFERENT CLASSES OF SURFACE PAVEMENT, INCLUDING LIFE-TONNAGE AND TON MILES PER YARD WIDTH PER CENT OF COST.

Pavement.	Tons per yard width per annum.	Life, years.	Life tonnage Per yard width.	Cost per sq. yd. of surface.	Annual cost, including proportion of capital and maintenance per sq. yd.	Ton miles per yd width %.	Cost in cents per traffic per mile.
6-in. Belgium block	524,000	18	9,432,000	$2.50	$0.17	17	$0.058
4-in. Belgium block	150,000	50	7,500,000	1.87	.07	12	.08
Hardwood	162,000	17	2,754,000	3,37	.25	3.7	.272
Softwood	204,000	18	3,672,000	2.12	.15	7.7	.128
4-in. pitch macadam	120,000	11	1,320,000	.75	.066	10.3	.096
7-in. waterbound macadam	120,000	1	120,000	.....	.18	3.8	.264
7-in. waterbound macadam tar sprayed.	120,000	2	240,000	.25	.12	5.7	.18

Tonnages on Roads Board basis, except on exceptionally heavy traffic, when it is based on estimated total actual weights.

English ton = 2,240 pounds.

was constructed amount of wear was very much reduced.

The necessity for knowing the kind of travel is well shown by the English tables. On one road in one of the counties it cost 1 4-10 cts. to carry a ton a mile, and on another macadam road in the same country it cost 12/100 ct. It appears from the English figures that when the cost of maintenance exceeded ⅔ ct. a ton a mile, it was more economical to use granite block on a concrete base. The cost for annual maintenance of a 6-in. block on a concrete base near the docks

TABLE VIII.—TONS CARRIED ONE MILE FOR ONE CENT OF MAINTENANCE COST.

County	Weight of traffic in tons.	Per annum.	Average cost. Per mile per annum.	Weight in tons carried 1 mile for 1 cent.	Cost of maintenance in cts. per traffic ton-mile.
Norfolk .	39	14,200	$ 206.25	.70	1.42
Norfolk .	96	35,000	80.00	4.50	22
Warwick.	185	67,500	430.00	1.60	62
Warwick.	239	87,200	635.00	1.45	68
Warwick.	242	88,300	430.00	2.15	46
*Kent ...	348	126,700	1,495.00	.85	118
Norfolk .	359	131,000	366.25	3.70	26
Norfolk .	385	140,500	281.25	5.20	192
Norfolk .	390	142,300	275.00	5.35	186
Warwick.	451	164,600	635.00	2.70	36
Norfolk .	504	184,000	251.25	7.55	132
Kent ...	528	192,100	1,740.00	1.10	90
Warwick.	609	222,000	1,345.00	1.70	58
Warwick.	734	268,000	2,095.00	1.35	74
Warwick.	736	268,600	670.00	3.95	254
*Kent ...	796	289,800	960.00	3.15	32
E. Sussex	984	359,000	1,665.00	2.20	44
Norfolk .	1,057	386,000	1,090.00	3.65	26
*Kent ...	3,030	1,102,810	10,500.00	1.05	94
Kent	3,030	1,102,810	8,960.00	1.25	80
*Kent ...	3,030	1,102,810	8,960.00	1.25	80
*Surrey .	5,694	2,078,300	5,160.00	4.20	24
Surrey ..	5,694	2,078,300	3,020.00	7.15	14
*Surrey .	5,694	2,078,300	3,020.00	7.15	14
Comparison With Two Massachusetts Roads.					
Beverly .	2,898	1,058,430	$3,257.00	6.50	37
Weston .	1,920	699,924	1,933.00	7.02	28

*Surface tarred.

Period of 14 years.

All waterbound macadam with or without surface tar.

in Liverpool has been 58/1000 ct. a ton a mile.

Table IX shows some of the costs of different kinds of street surfaces and pavements in Liverpool.

EXPLANATION OF TABLE X.

Table X shows actual facts in Massachusetts. It illustrates the misleadingness of statistics if read without adequate knowledge of actual conditions. The variation in the costs shown are due to various causes, traffic and weight, toughness of stone, whether road has been resurfaced or not, good and bad bituminous materials, and proper and improper use of materials. A study of each road will prove profitable.

Ashby, with high cost of maintenance, small traffic, can be compared with Hamilton, with more traffic and a small cost for maintenance, per ton. Ashby, built of local stone, comparatively soft, was resurfaced with the same local stone when the road was about 12 years old, 30 tons being used to each 100 ft. of road. It is a country road. It had an application of cold asphaltic oil in 1913, ¼ gal. being used to the square yard of road. Practically, the stone had worn down ¼ in. a year.

Hamilton was built of trap rock macadam and it is on a main through route. When the road was eight years old the stone had worn down about 3 ins., and the road needed resurfacing. Automobiles had arrived. In 1907, ½ gal. of the heaviest asphaltic oil that can be applied cold was spread upon the road and properly covered with pea stone and gravel.

This treatment was repeated for two years. Then ¼-gal. of heavy hot asphaltic oil was sprayed upon the road and properly covered; this treatment has been repeated once. The road has been constantly kept patched and sanded when necessary. It is in better condition today than in 1907.

Beverly, with a high cost of maintenance per ton mile, can be compared with Lynn, with a small cost per ton mile. Both roads are on main routes. Both were trap rock macadam. Beverly has a large number of heavy ice teams on 3-in. tires. It was resurfaced with trap rock when the road was eight or nine years old, 30 tons of stone being used to each 100 ft. of road. The trap rock had worn down ⅓ in. a year. In 1910 it was coated with ½ gal. of hot asphaltic oil per square yard, properly covered with pea stone and dust. This failed in one month under the heavy ice teaming, though the same material and methods were used on the next 20 miles of road on the same route, and the surface has stood ever since with constant patching and ¼ gal. per square yard of the same oil sprayed onto the center of the road, 8 ft. in width. For the next four years the Beverly road was maintained (except where it was resurfaced) by the use of ¼ gal. of cold oil per square yard, two applications being used the first year, one each year since.

One-third of the road was resurfaced in 1913 with an asphaltic macadam two inches in depth, at a cost of $1.20 per square yard, 2½ ins. stone being used because of the heavy teams and trucks.

Lynn, trap rock macadam, connects with Parkway where only pleasure vehicles are admitted, except on local business. In 1907, ½ gal. of hot refined tar was sprayed upon the road, and covered and kept covered with pea stone and dust. It was constantly patched, with tar and chips. It has been recoated twice with hot refined tar sprayed upon the road and covered as before. It is in excellent condition, but note—90 per cent of the travel is motor vehicles; it has few teams and fewer heavy teams.

Medford-Somerville, a trap rock macadam, built with the 2½-in. stone on top. A road 28 ft. in width, with heavy city teaming. A stone quarry on the side crushing 100 to 300 tons of stone a day. This road has never been in good condition since it was two years old. It always has some depressions, although it has been constantly patched and all depressions filled with trap rock. Constantly means daily. It has always been muddy. A part of it was resurfaced with asphalt macadam this year, the portion beyond the stone quarries. The whole road needs it. It has been treated with tar. A part of it has been coated with heavy tar. A portion had three coats of ¼-gal. each of hot refined tar, covered with pea stone, in one year. It failed, was never in good condition, and we are in doubt whether to reconstruct with granite block on a concrete base, with concrete, or to try an asphaltic macadam.

Milton is a road of the same character with heavy granite teams going over it. The cost has been high because the trap rock wore out so rapidly under the heavy concentrated loads on iron tires.

Saugus. Trap rock macadam on a through route with a great deal of heavy teaming, both teams and trucks. The stone wore out over ½-in. a year. It needed constant patching with additional stone, was never in excellent condition except when recently resurfaced. It had 2 to 3 ins. of new stone every four or five years. In 1910 it was resurfaced with 3 ins. of asphaltic macadam at a cost of about $1 per square yard. This has stood ever since but has needed some patching. A portion was built in the fall when it was cold and this portion failed. No bituminous work should be done in cold weather and a temperature of over 60 degrees is vastly better than one of under 40° F.

Shrewsbury. A through route—too much heavy hot oil was used on it before we understood how to use oil. One-half gallon per square yard was applied on two successive years. It rolled, rutted and was always in bad condition. It has cost a good deal to remove surplus oil, smooth off the bunches and rolls and keep it patched.

Thuro. A country road, with little traffic. Built 12 ft. wide of 4 ins. of broken stone on sand, the stone being bound with clay. An experiment but a failure. The road was constantly ravelling and needed more stone. It was widened to 16 ft. New stone was added and rolled in and it was coated with light oil in 1912 and 1913 and is now in good condition.

Weston. A macadam road on a through route. Refined tar applied in 1907—oiled with hot oil on surface in 1909 and 1910—maintained with patching until 1912 when a portion of the road was resurfaced with a 2-in. bituminous macadam. Trap rock stone 2½ ins. in size were rolled hard and about 1¾ to 2 gals. to the square yard of a good grade of asphalt being sprayed in under pressure. This was covered with the smaller stone, rolled, and on some portions of the road a surface application was sprayed of ⅓ to ½ gal. per square yard, properly covered with pea-stone and rolled. This cost from 90 cts. to $1 per square yard. The road is in most excellent condition and we expect to have it wear 10 to 15 years with practically no patching, although we may have to renew the surface coating by spraying every three to five years. We have one road of this kind six years old, that hasn't needed a single patch as yet.

TABLE X.—TRAFFIC AND COST OF MAINTENANCE ON MASSACHUSETTS STATE HIGHWAYS.

Town.	Amount of traffic.		Repairs and maintenance.			Character of traffic.			Number of vehicles per day—Horse-drawn vehicles			
									Single horse.		Two or more.	
	Total tonnage per day.	Total tonnage per year, 300 days.	Cost per mile per year.	Cost per ton miles per year, cents.	Period, years.	Runabouts.	Automobiles, touring cars and wagons.	Trucks.	Light.	Heavy.	Light.	Heavy.
Ashby	271	81,150	$ 266	0.38	16	14	65	4	70	16	5	14
Beverly (No. 1)	1,618	485,220	1,104	0.23	15	60	278	8	66	46	4	12
Hamilton	1,199	359,730	200	0.06	15	86	334	31	75	39	2	27
Lynn	3,468	1,040,430	1,081	0.10	9	194	1,365	13	28	19	1	14
Medford-Somerville	1,332	399,570	1,031	0.26	6	44	121	49	47	198	2	193
Milton	1,140	342,210	592	0.17	14	15	50	0	30	77	2	88
Saugus	1,022	306,660	1,334	0.44	14	15	58	78	25	190	3	65
Shrewsbury	1,305	391,500	510	0.13	17	76	407	17	64	60	4	36
Truro	186	55,770	143	0.25	17	7	63	1	15	14	1	3
Weston	1,918	575,280	1,040	0.18	15	115	533	30	167	98	5	59

*1909 Report used and results increased 70 per cent to correspond with 1912 Report. Also weight of double heavy teams increased from 2.46 to 5 tons.

MOTOR TRUCKS AND THE COST OF ROAD MAINTENANCE.

Mr. H. T. Wakeland, engineer of the county of Middlesex which is just out of London and has a very large amount of traffic over its roads, has given some very careful figures showing damage caused to roads by motor omnibuses weighing about six tons each when laden. He took certain roads which had heavy traffic and gave the cost of maintenance (not including watering or cleaning) for macadam roads for three years previous to the motor bus traffic, and the cost per square yard for the year 1912-13 as follows:

Road.	Average cost per Sq. yd. per annum for three years previous to motor omnibus traffic, cents.	Cost per sq. yd. for 1912-13 since the advent of motor omnibuses, cents.
A	[illegible]	[illegible]
B	[illegible]	[illegible]
C	[illegible]	[illegible]
D	[illegible]	[illegible]
E	9.1	15.4
F	8.7	15.1
G	5.9	16.8
H	5.1	11.1
I	[illegible]	[illegible]
J	16.9	42.9
Average	12.3	25.6

This shows that the average cost of maintenance for three years before the motor bus came in was about 12 cts. a square yard a year. Since the motor bus was put on the cost has increased to over 25 cts. a square yard a year. The maintenance cost to carry one ton one mile in 1911-12 was 1.2 cts. When the motor bus was put on the maintenance cost was raised to 1.8 cts. per ton per mile. Mr. Wakeland's opinion is that this increase was practically all due to the motor bus. The increased cost of the road upkeep has been found to be about four cts. per car per mile, or two-thirds of a cent per ton per mile in the case of a motor bus on rubber tires. In many cases the macadam surface has been practically destroyed by motor bus traffic on hard rubber tires. These were macadam roads in good standard condition prior to the inauguration of the motor bus traffic and more than sufficient to carry the ordinary traffic. The road authorities should be authorized to direct which roads shall and which roads shall not be used by motor vehicles and motor buses, and Mr. Wakeland states, as do the other county engineers in England, that a license fee of $50 a year for motor trucks is entirely insufficient to pay for the increased cost of maintenance caused by the use of the trucks on the roads.

CONCLUSIONS.

A road means a highway that can be traveled over with reasonable convenience and with reasonable effort by ordinary vehicles. Two ruts and a horse track with 6 ins. of mud and large potholes scattered frequently over its surface do not constitute a road.

A road to be a road worthy of the name must be constructed and maintained so that it will at all times satisfactorily and economically bear the traffic which passes over it. Good drainage, foundation whenever necessary, and a top surface always maintained so that it will shed water, are necessary prerequisites. If any one of them is missing you do not have a good road. Constant maintenance is eternally necessary; drainage must be always open, and road surfaces must always be maintained. The most economical way to maintain a road is by constant maintenance.

With modern Massachusetts motor vehicle travel, a newly-built macadam road without any bituminous materials having been used on it, will easily be destroyed in one month, so far as its surface is concerned, and will be damaged so that it will require 15 to 25 cts. a square yard to put it back into condition so that some bituminous material can be used.

Roads and road surfaces must bear the traffic that goes over them. The tables show conclusively how expensive it is if materials are not used and methods are not used with those materials, so that the road can withstand the traffic which goes over it.

I think by a study of the tables and our experience, you can gather, knowing the traffic on your road, what will probably be the most economical way to construct or reconstruct your road. With a certain volume and weight of traffic, granite block on a concrete base will perhaps be the most economical, especially for heavy concentrated loads. Thirty-ton loads on four wheels are very common near the docks in Liverpool, where 6-in. granite block on a concrete base has worn 18 years.

In figuring costs you must figure what the road has to bear in tons or otherwise. Speaking in round numbers, you will find on figuring the English roads and American roads that it costs about a cent a vehicle a mile per year to maintain the roads. It would not surprise me if the same were true on dirt roads. Dirt roads, of course, will not stand very much heavy teaming, no matter if they are kept constantly patched. This money you must have and it must be properly expended if your original cost of construction is not to be lost and you are not to find yourself saddled with a debt on which you are constantly paying interest years after the road has passed away in dust.

But you must go further than that and figure what are the proper methods, proper materials, and the best and most economical surfaces. As I have shown, for merely automobile travel, cars on pneumatic tires, you can economically and satisfactorily maintain a gravel or macadam road in the summer season by the proper use of a [illegible] bitumen and constant maintenance and patching. You can maintain a water-bound macadam road, if the traffic is not too heavy and by too heavy I mean too much teaming, by the same methods.

You can maintain a bituminous macadam road more economically than either of the above roads when the traffic in teams or trucks is so heavy that neither of the other roads are economical. With heavy concentrated roads that will crush or wear a trap-rock macadam say at the rate of from ½-in. to an inch a year, water-bound macadam is not economical, either with or without a bituminous surface. Bituminous macadam will be much cheaper in the end. That is, of course, provided bituminous macadam can withstand the traffic that goes over it. With a certain volume of traffic it is more economical to construct a pavement probably on a concrete base or possibly in some places a concrete road.

You will note that the increased cost on many water-bound macadam roads in England when the motor bus service was put on was substantially 4 cts. a mile for every mile that the buses, which weighed 6 tons including their load, went over the road. Proportionately a small runabout would probably cost about ¼ ct.; a large touring car from 1 to 2 cts., and probably a heavy horse-drawn vehicle with a 3-ton load would cost about the same.

TABLE I.—COST OF MAINTENANCE WORK ON VARIOUS ROADS DURING 1914.

Section.	Miles.	Dragging.	Repair.	Permanent grading.	Tools.	Misc.	Total.	Average per mile.
1	13	$245.00	$583.75	$600.00	$ 36.74	$ 35.20	$1,500.69	$115.50
2	9	164.47	384.39	......	31.20	13.95	594.01	66.00
3	11	221.65	153.34	......	31.20	9.76	415.95	37.80
4	17	136.55	1,404.30	......	33.00	37.37	1,611.22	94.80
5	11½	151.00	514.14	......	4.55	16.08	685.77	59.60
6	9½	225.90	123.75	......	9.30	8.63	367.58	38.60
7	9½	96.75	311.76	......	46.05	10.90	465.46	49.00
8	15	......	425.50	......	3.45	10.29	439.24	29.25
9	9½	81.45	757.74	......	1.00	20.18	860.37	90.75
10	14	172.98	844.08	......	7.89	24.62	1,049.57	74.80
11	13	141.05	242.00	145.90	8.86	12.92	550.73	42.85
12	16	......	120.00	......	5.04	3.02	128.06	8.00
13	9	121.45	100.00	663.70	30.44	21.99	937.58	104.17
14	10	79.65	357.00	......	......	10.50	447.15	44.72
15	12½	163.35	842.78	......	......	24.18	1,030.31	82.50
16	15	251.82	332.18	......	......	14.03	598.03	39.85
17	6½	69.65	381.32	207.00	......	15.80	673.77	103.50
18	3	18.00	76.00	......	......	2.27	96.67	32.22
19	3	32.00	44.75	......	......	1.84	78.59	26.20
20	5	63.55	126.14	......	......	4.55	194.25	38.85
21	4½	27.67	39.92	......	......	1.63	69.22	15.38
22	6	48.05	142.02	......	......	4.57	194.64	32.44
Total	222½	$2,511.99	$8,307.26	$1,616.60	$248.72	$304.28	$12,988.85	$55.76

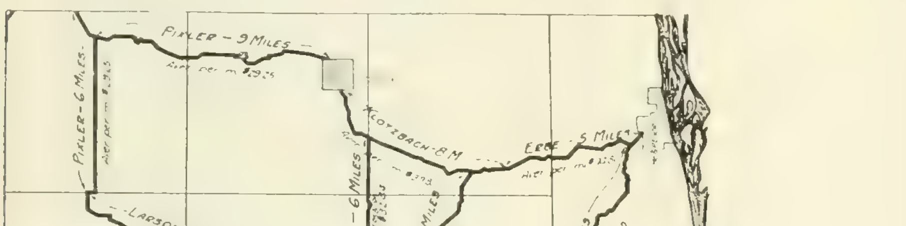

Fig. 1. Map of Clayton County, Iowa, Showing Roads Under Patrol Maintenance.

Methods of Cost of Maintaining Earth Roads by the Patrol System in Clayton County, Iowa.

The patrol system of earth road maintenance inaugurated by E. B. Tourtelot, county engineer of Clayton county, Iowa, is described by him in a recent issue of the service bulletin of the Iowa Highway Commission.

The greatest difficulty in the past with which this county and no doubt most of the counties in the state, has had to contend, has been the failure to secure at the time most needed, labor to drag or repair the road. Contracts, of course, had been made for dragging, but in very few cases had a contract been made which required a man to give his undivided attention to county road work.

The new system, as planned by the writer, tries to do away with this feature. There are twenty-two patrolmen who cover the entire county road system, all of whom have contracts with the county to work at such a time and at such a place and according to such methods as the board of supervisors or engineer may see fit to adopt in order to maintain continuously and in the best condition practicable, the entire length of the road. Nearly all of the patrolmen have given a $500 bond for the faithful performance of all the duties imposed upon them, which includes the exercising of all reasonable diligence and care in the preservation of all road machinery and tools in their possession.

All of the maintenance, repair work and most of the permanent work on the county system roads is taken care of by the patrolmen. Each man is supplied by the county with all road machinery and tools necessary for the carrying on of the work. He is also allowed to hire additional help at the same rate he is paid, to assist in dragging or other work, providing an order is given by a member of the board or engineer. The road machinery given each man was as follows: 1 wheel scraper, 1 slip scraper, 1 road plow, 1 Kelly road drag, 1 Glide, 2-horse grader. Necessary small road tools. such as picks, shovels, stone hammers, drills, etc.

The patrolman furnished his own team and wagon and paid for the upkeep of the team, as well as his own expenses. The county paid 45 cts. per hour for man and team, and 25 cts. per hour for labor. When it was necessary to use more than two horses for dragging, the county paid 7 cts. per hour for each additional horse. The county road system has been well dragged and repaired this past season. Some patrolmen have proven far superior to others. The county will use the patrol system next year. Incompetent men will be weeded out, and new ones secured.

Table I gives the cost of maintenance work on various roads. Figure 1 shows the location of the roads and average costs of maintenance.