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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

SAFE STRESSES IN STEEL COLUMNS.

By J. R. Worcester, M. Am. Soc. C. E.

To be Presented February 19th, 1908.

The subject of a proper allowance for stresses in columns has been treated so often by theorists that it may seem as though no more could be said on the subject without danger of exhausting the patience of the engineering profession, but, in spite of all the theories, the practice of steel designers, as shown by the specifications in general use to-day, may well bear further consideration.

The reason for this is that all "rational" column formulas, based on the elastic properties of the steel, are founded on considerations which are applicable only to ratios of length to radius of gyration far beyond those allowed in actual construction. It is known, in a general way, that steel in compression should not be strained as high as in tension, and there is a popular impression that the only reason for this is that when the ratio of l to r increases above 0, or, at most, above a value very little above 0, the strength becomes lessened rapidly on this account; but there has been a growing tendency to neglect the fact that, even in very short columns, there is not the same unit strength manifested against compressive and tensile stresses.

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The reason for this difference is manifest from a moment's consideration. It may be admitted that, within the elastic limit, the modulus of elasticity is practically the same, whichever way the metal is strained—in tension or compression. One may even go further and admit that the elastic limit is practically the same for both stresses; but, what happens after the elastic limit is passed? In tension, the member merely straightens out—if it is not straight to start with—and stretches, while with every increase in length comes an increase in resisting strength until the ultimate strength is reached, the final strength being nearly twice as great as it was at the elastic limit.

In compression, however, soon after the elastic limit is passed, the column will cripple, and the more it cripples the weaker it becomes. It is not necessary to consider the ideal conditions of exact equilibrium in the resisting power of a section, when crippling would not take place, because the equilibrium—unstable at the best—is not attainable in practice. On the other hand, it must be admitted that the ductility, which is of such great advantage in tension, is not present to an appreciable amount in compression, and that the ratio between working stresses and destructive stresses in all structures depends on the compression members, and not the tension, when anything like equal working units are allowed in the two.

An examination of the results of tests of full-sized columns made by Tetmajer, Marshall, Christie, Bouscaren, Strobel, Lanza and the Watertown Arsenal, shows strengths of wrought-iron columns, in which the $l \div r$ does not exceed 120, of from 16 000 to 43 000 lb. per sq. in., and for mild steel, from 18 000 to 46 000 lb. per sq. in. By far the larger part of these range between 22 000 and 34 000 lb. for iron, and between 22 000 and 46 000 lb. for steel. It is very noticeable, also, that one finds results of more than 28 000 lb. with the longest length and less than this amount with values of $l \div r$ as small as 30. While the axis drawn through the central portion of the group of these experiments, when plotted, shows some inclination toward lower values for increased length, the center of the group lies at about 30 000 lb. when $l \div r = 90$, and there is very little increase in strength manifested in the tests with a lesser length than this. It is apparent, therefore, that if the compression is allowed to run as high as 16 000, the factor between working stress and ultimate will not exceed 2. In tension members, on the other hand, the corresponding factor is nearly

4. The answer to this argument is, of course, that nobody cares what the factor between working strain and ultimate may be, as one is really interested only in the elastic limit, which it is never intended to reach. Is it not time to call a halt on this line of reasoning? Have not engineers been overconfident in their ability to design structures so that all possible contingencies are taken into account? One would not willingly make use of a material in tension which had no stretch beyond the elastic limit, yet it would be in no way more hazardous than to neglect the fact that such is the case with compression members, unless a greater factor below the elastic limit were allowed in these.

The history of the development of the column formulas used in bridge specifications may shed a little light on the way in which unit strains have crept up.

The adaptation of the Gordon formula to wrought-iron columns by Rankine had for a numerator 36 000 lb. That is, Rankine recommended that this value be assumed for the ultimate strength of wrought iron in compression of short columns. The earlier specifications for railroad bridges, in which Rankine's formula was used, recommended 7 500 or 8 000 in the numerator when 10 000 was used for tension, and this difference between the numerator of the compression formula and the tensile unit has been retained to a large extent in specifications until recently.

When the straight-line formula was first introduced, it was recommended for the reason that a straight line could be drawn that would coincide very well with the plotted results of experiments for ratios of l to r between 90 and 150, and that, in giving less values than experiments warranted above this point, it erred on the side of safety. The straight line, thus drawn, when prolonged the other way, reached the $l \div r = 0$ line at about the tensile value of the steel, making the formula take the form, $A = B - C \times \frac{l}{r}$, in which $A =$ the allowable compressive stress, and $B =$ the allowable tensile stress. This simple form appealed strongly to engineers, and was readily accepted by many, but the fact was not recognized by all that the line when plotted goes far above the experiments for values of $l \div r$ less than 90. The tables by C. L. Strobel, M. Am. Soc. C. E., for the strength of **Z**-bar columns were based on a straight-line formula, but this is a notable instance of recognition of the error of the formula for short lengths, because he limited his stresses to 12 000 when the straight line went higher than this amount.

At this point, may be noted what seems to have been an unwarranted change in specifications, due to the reprehensible practice of copying from one to another with slight changes. There have always been many engineers who liked the form of the Rankine formula and refused to give it up. Many appear to have been struck with the simplicity of the straight-line formula in having the unreduced compression unit the same as the tension, and, wishing to take advantage of this feature, but still adhering to the Rankine form, they adopted the tension unit for the numerator of the formula. This throws the curve entirely above the field of tests, and, apparently, cannot be defended by any reasoning.

A later development, of the specifications which are based on the form of the Rankine formula and still retain the tension unit in the numerator, is to adopt a lower constant in the denominator. This, by some, is made 20 000, and by others, 8 000. The former brings the curve within the outer limits of the group of tests, while the latter passes well through the middle of the group for values of $l \div r$ greater than 50, but is above the group for lower values.

Perhaps enough has been said to show that the formulas in general use to-day need to be sawed off at the end toward low values of $l \div r$. It may also be said that they all need to be amputated at the other end. Mr. Schneider, years ago, suggested that values of $l \div r$ greater than 100 should not be allowed in main members, and this limitation, with slight variations, has been generally accepted since that time as an essential of good practice.

If, then, the Rankine formula be used, with the numerator value equal to the tension, and the compression stress be limited to, say, 75% of the tension, and the value of $l \div r$ to 100, or thereabouts, one obtains for a diagram a horizontal line running to a cusp, then a concave curve running to another cusp, then another straight line. Could anything be more irrational? The straight-line formula is little better; the only difference being, that, in the middle portion, there is a straight line instead of the curve. How much better it would be to use a continuous curve throughout, embodying its own limitations at each end!

The late J. B. Johnson, M. Am. Soc. C. E., suggested this same thought in his book on Modern Framed Structures, and proposed a parabola. This is safe and simple, though, if the vertex is kept down to a safe value of stress for short lengths, and the limitation of $l \div r$ is made not higher than 120, the central portion of the curve does not reach as high as tests would warrant.

An elliptical curve fits the case much better, the ellipse being drawn with its center at the zero value for both stress and $l \div r$, and having for one semi-diameter the limiting value of $l \div r$, and for the other the limiting stress for zero lengths. The form of this equation is:

$$A = B \sqrt{1 - \frac{l^2}{(C\,r)^2}}$$

in which A = the allowable stress, B = the maximum stress at $l \div r = 0$, and C = the maximum value of $l \div r$ allowed. This curve is easy enough to plot as an ellipse, but, if a diagram be only arranged so that on the scale of ordinates B is of the same length as C on the scale of abscissas, the curve becomes the quadrant of a circle.

The diagram, Fig. 1, illustrates graphically a number of curves of well-known specifications, together with the results of tests, by the experimenters previously referred to, reduced so as to allow for a proper factor of safety. This reduction is made so that the experimental results can be compared with the formulas in their usual forms. The reduction applied is proportional to the ratio between the tension unit and the ultimate tensile strength of the metal. That is, for wrought iron, the test values are multiplied by $\frac{16\,000}{50\,000}$, and for steel the multiplier is $\frac{16\,000}{60\,000}$.

The proposed formula, as plotted, is based on limiting values of compressive stress of 12 000, and of $l \div r$ of 120, which appear to be warranted by experiments and by good practice, and, as the scales are arranged, the curve is circular.

The writer puts forward a new formula with great diffidence, knowing well that custom is a very difficult thing with which to contend, and how cold a reception new compression formulas have met in the past; but, considering how poorly the formulas now in use fulfill the requirements, and realizing that the public is fully awakened at the present time to their insufficiency, the time seems to be opportune for at least suggesting the possibility of an improvement.

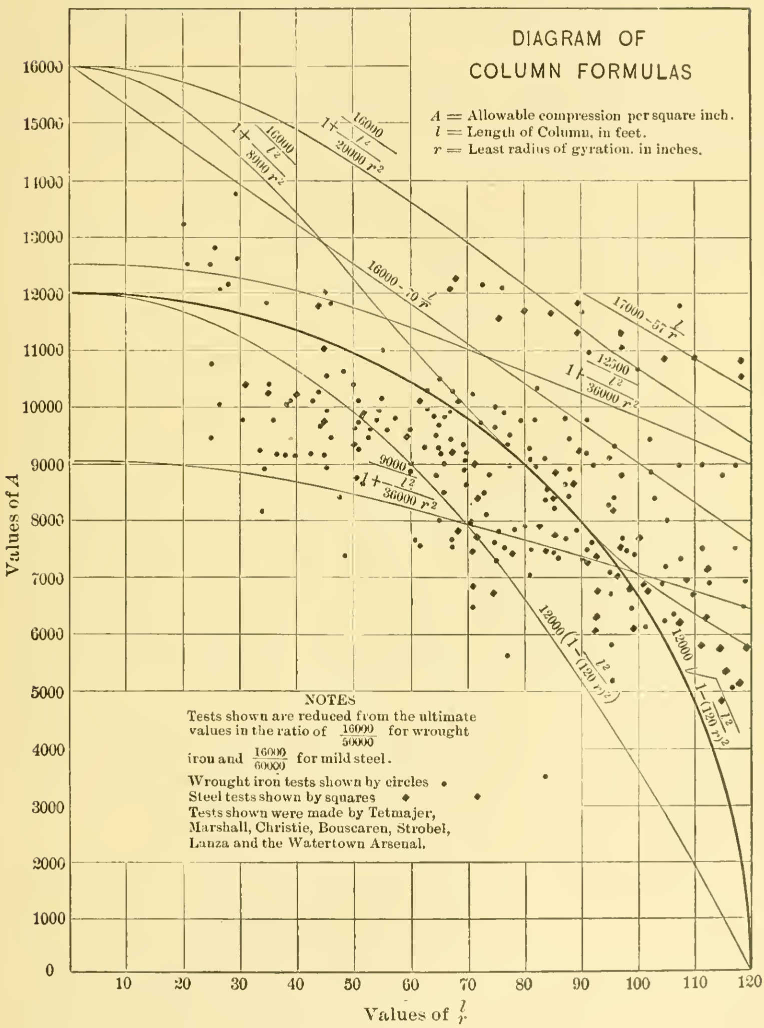
DIAGRAM OF
COLUMN FORMULAS
A = Allowable compression per square inch.
l = Length of Column, in feet.
r = Least radius of gyration, in inches.
NOTES
Tests shown are reduced from the ultimate values in the ratio of 16000/50000 for wrought iron and 16000/60000 for mild steel.
Wrought iron tests shown by circles •
Steel tests shown by squares ◆
Tests shown were made by Tetmajer, Marshall, Christie, Bouscaren, Strobel, Lanza and the Watertown Arsenal.
Values of A
Values of l/r

Fig. 1.

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PAPERS AND DISCUSSIONS.

EFFECT OF EARTHQUAKE SHOCK ON HIGH BUILDINGS.

By R. S. Chew, Assoc. M. Am. Soc. C. E.

To be Presented March 4th, 1908.

In submitting this paper, the writer is well aware that he is dealing with a force that can be measured only by the resistance encountered; and it was simply with a view of determining the nature of the stresses induced in structures by a shock, such as that in San Francisco on April 18th, 1906, that the following was undertaken.

All realize that, with a possible exception, the steel-framed structures in San Francisco stood this shock. This fact has promoted confidence, and has satisfied architects and owners that such is the safe type of building for the Pacific Coast. The engineer, however, cannot be satisfied until he ascertains just how a disturbance of this nature affects high buildings.

The effect of an earthquake is to produce a complex movement in the crust of the earth. This movement is a wave motion accompanied by more or less twisting. This twisting or torsional effect is small, and affects only the first tier of columns. The length of the wave is very long, so that the vertical movement is small, and, for a structure with a well-designed foundation, may be neglected. The effect of the shock, then, is from the horizontal motion, which is a rapid oscillation.

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From mechanics, it is known that:

$$\text{Force} = \text{Mass acceleration, or } F = M\,a = \frac{W\,a}{g} \quad \text{..........(1)}$$

or, the force of the earthquake on any structure is the mass of the structure into the acceleration produced.

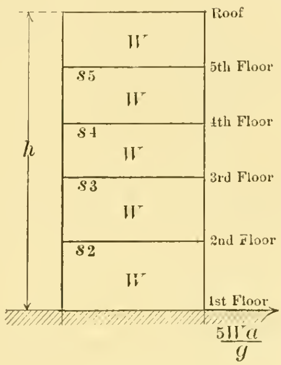

FIG. 1.

Imagine the structure represented by Fig. 1 to be built of a perfectly rigid material, and that W is the weight of each story; then, as the foundation takes up the movement of the earth, it endeavors to set the structure in motion. The inertia of the building resists, and calls into play the force, $F = \frac{5\,W\,a}{g}$. The shearing stresses at each floor are:

$$S_2 = 4\,\frac{W\,a}{g}$$

$$S_3 = 3\,\frac{W\,a}{g}$$

$$S_4 = 2\,\frac{W\,a}{g} \text{ etc.}$$

The maximum bending moment $= \frac{5\,W\,a}{g} \times \frac{h}{2}$. If these shears and bending moments could be developed, the building would follow the movement as a whole. There are, however, no perfectly rigid materials, so that, under the action of a force, there would be deformation which, as will be seen later, is different in different types of buildings.

Consider first a structure which has no wind bracing. By reference to Fig. 2 it will be seen that each story weighs W, and that, therefore, the resistance that each story would offer to having set up in an acceleration, a, would be $\frac{W a}{g}$.

The building, under this action of forces, is a beam cantilever under a uniform load so that:

Moment of inertia at section $1 - 1 = I_1$
" " " " " $2 - 2 = I_5$

assuming $I = \frac{I_1 + I_5}{2}$ then, approximately,

$$\Delta = \frac{5\ W a}{g} \times \frac{h^3}{8\ E\ I} \dots\dots\dots\dots\dots\dots\dots\dots (2)$$

It will be noticed that Δ, in Equation 2, does not represent the displacement due to the shock, and may be greater or less than this.

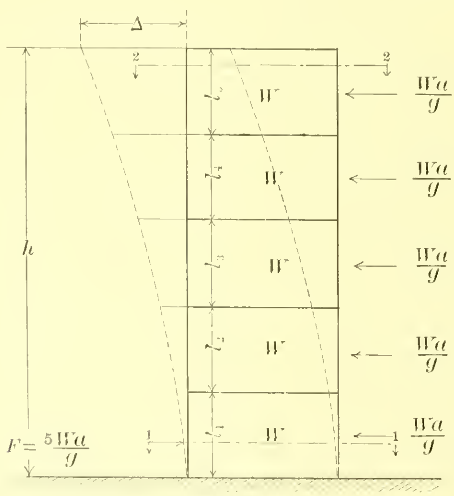

FIG. 2.

If the building be represented as a single line it can be seen that, as Δ varies directly as a, h^3, and W, and inversely as E and I, that either of the three curves may be attained, dependent on these variables.

If now, E and I be large, and W and L small, and the oscillation rapid, then the building would endeavor to follow the movement closely, in which case the curves produced by the oscillation would produce a wave motion in the structure as shown approximately by Fig. 4.

The writer believes this to be the effect of the late shock as felt on the majority of low structures. He identifies it as the effect produced on the three-story apartment house which he was in on that memorable morning. It was quite different from the effect felt in the fifth story of a six-story steel-frame structure during a later shock. In the latter case, there was a decided swinging movement of the building.

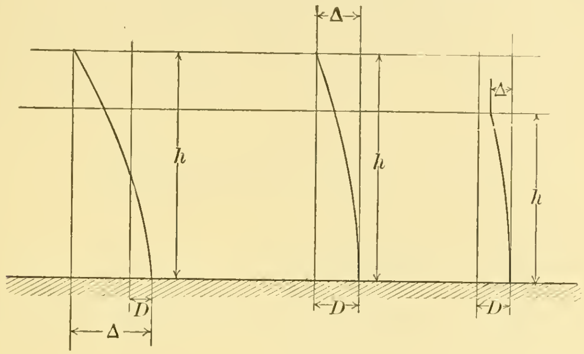

FIG. 3.

Referring to Fig. 1, one may note the high shearing stresses produced. These are, of course, the same in Fig. 2. The bending moment is a maximum at the foot of the column, and equals:

$$\frac{5\,W\,a}{g}\,\frac{h}{2} + 5\,W\,\frac{\Delta}{2} = \frac{5\,W}{2}\left(\frac{a\,h}{g} + \Delta\right) \ldots\ldots\ldots\ldots(3)$$

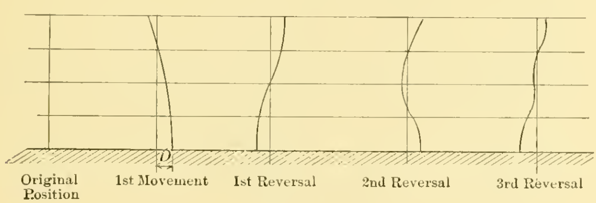

FIG. 4.'

It can be seen at a glance that this type of construction is not adapted to resist any shocks except those of very small displacement; and, even in these, the buildings will fail in detail. For instance: It was noted, after the shock of April 18th, that in a number of cases the connection of beams to columns had failed by the rivets shearing off. Reference to Fig. 5 will explain the condition. By referring to the curves, it will be seen that during the vibration the column bent, throwing the

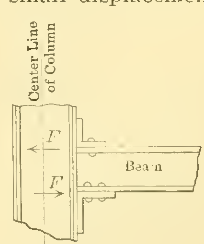

FIG. 5.

couple, F F, into action, and the rivets, not being sufficient to stand this, failed. The writer has also noted several buildings of this type in which there is a decided crack in the brickwork following the column, which tends to substantiate this theory.

A wind-braced building will act a little differently from the foregoing, due to the fact that the point of contraflexure in the columns is fixed by the bracing, so that the building in part will follow the movement.

Let D = the horizontal displacement,

t = the time for said horizontal displacement,

a = the acceleration $= \dfrac{D}{t^2}$,

g = the effect of gravity;

then, the force exerted on the building $= F = \dfrac{W\,a}{g} = \dfrac{W\,D}{g\,t^2}$, where W equals the weight of the building.

If, in Fig. 6, it be assumed that the horizontal girders are stiff enough to fix the columns at the knees, then the effect on the building by the movement, D, is as shown.

$D' = \lambda_1 + \lambda_2 \ldots\ldots + \lambda_5 = 2\ (\varDelta_1 + \varDelta_2 \ldots\ldots + \varDelta_5)$ = the deformation in building.

W_5 = the weight of the building above and including the first floor.

W_4 = the weight of the building above and including the second floor.

W_3 = the weight of the building above and including the third floor.

From mechanics, it is known that:

$$\varDelta = \frac{F\,l^3}{3\,E\,I} = \frac{W\,D}{g\,t^2}\,\frac{l^3}{3\,E\,I} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(4)$$

$$\varDelta_5 = \frac{W_5\,l^3_5\,D}{3\,E\,I_5\,g\,t^2}$$

$$\varDelta_4 = \frac{W_4\,l^3_4\,D}{3\,E\,I_4\,g\,t^2}, \text{ etc.}$$

$$\text{Therefore } D' = \frac{2\,D}{3\,E\,g\,t^2}\left(\frac{W_5\,l_5^{3}}{I_5} + \frac{W_4\,l_4^{3}}{I_4} + \frac{W_3\,l_3^{3}}{I_3}, \text{etc.}\right) \ldots\ldots(5)$$

or, if $D = D'$,

$$\text{Therefore } 1 = \frac{2}{3\,E\,g\,t^2}\left(\frac{W_5\,l_5^{3}}{I_5} + \frac{W_4\,l_4^{3}}{I_4}, \text{etc.}\right) \ldots\ldots\ldots\ldots(6)$$

The bending moment is a maximum at the base, and

$$= F\, l_5 + W_5\, \Delta_5$$

$$= \frac{W_5\, D\, l_5}{g\, t^2} + \frac{W_5^2\, l_5\, D}{3\, E\, I_5\, g\, t^2}$$

$$= \frac{W_5\, D\, l_5}{g\, t^2}\left(1 + \frac{W_5\, l_5^2}{3\, E\, I}\right) \qquad \text{.................(7)}$$

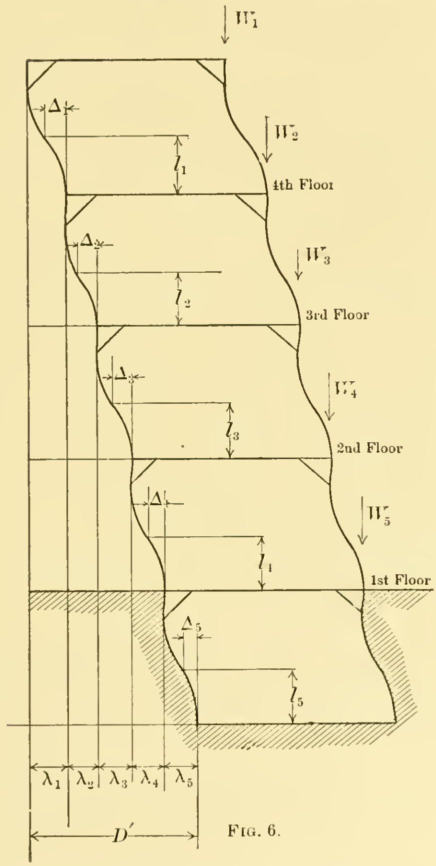

Fig. 6.

These conditions are reached for the movement in one direction. As this movement is back and forth, it gives the approximate curves shown by Fig. 7.

It can readily be seen that while the curve, $O\ A$, shows the curve in the building for the movement $1 — A$, that, before the return movement throws the reverse curve $O — B$ into the structure, a portion of the frame at the top will endeavor to straighten, or that the point, O, will move to x, and that the curve on the beginning of the return movement will be $x\ y\ z\ A$. This tendency is aggravated on each reverse, and produces the whip action at the top.

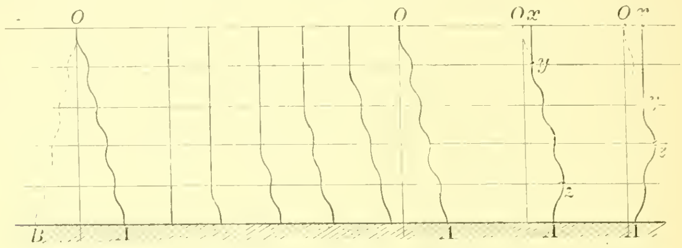

FIG. 7.

By Equation 5 it may be seen that should D be very small and l_5 on the first story large, nearly all the bending would occur in the first-story columns, the building above receiving a very small force. In this case, the first-story columns vibrate back and forth, and the building above is practically stationary. Of course, this would be productive of a high bending movement in the first-story columns, and a shock of any magnitude would wreck the building.

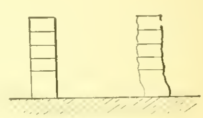

FIG. 8.

If the foregoing analysis is correct, the following may be noted:

1st.—That the stresses produced are similar to those caused by wind;

2d.—That, on account of quick reversal, the stresses are increased;

3d.—That, in a wind-braced structure, the total effect is distributed throughout the structure;

4th.—That, as this effect is a direct function of the weight, the wall and floor construction should be as light as consistent with strength;

5th.—That, as this effect is inversely as the coefficient of elasticity, the frame should be of a highly elastic material;

6th.—That, as the effect in buildings that are not wind-braced varies as the cube of the height, these structures should be limited in height;

7th.—That a monolithic foundation is preferable to one having isolated footings.

These conclusions all point to a steel frame with reinforced walls and floors as the type of construction for the vicinity of San Francisco.

With respect to reinforced concrete, the writer, although he believes it to be a valuable combination, thinks it unsuited to resist the forces that an earthquake shock would produce in a high building, for the following reasons:

1st.—This type of construction is not adapted to resisting reversed stresses;

2d.—It cannot take shock;

3d.—The construction is heavy, which conflicts with Conclusion 4;

4th.—The coefficient of elasticity is low, which does not agree with Conclusion 5;

5th.—The high bending moments produced in columns and girders would make their designs uneconomical;

6th.—Added to these, when it is considered that any failure in detail will necessitate the renewal of several entire members, the disadvantages of this construction will be seen.

Finally, the writer would recommend:

I.—The building to have lattice girders of the Warren type, as deep as the spandrel section will allow, running entirely around the structure at every floor. The advantage of this construction is obvious: Being in the center of the wall, the brickwork or concrete can be built around the members, the wall thereby being reinforced, the girder can be designed economically for the different floors. Being deep, it forms the lintel over the windows and at the same time decreases the length of the column.

II.—Monolithic foundations.

III.—Reinforced concrete walls, the reinforcement to run horizontally and vertically.

IV.—Floor slab to be of rock concrete at least $3\frac{1}{2}$ in. thick.

V.—Floors cut up by a large number of openings to be braced so that they can transmit all horizontal shear to the columns.

VI.—Wind bracing connections to be designed to develop the main member.

VII.—Wind bracing to be carried to the ground.

VIII.—Columns to be calculated for an extreme fiber stress of 12 000 lb.

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PAPERS AND DISCUSSIONS.

INVAR (NICKEL-STEEL) TAPES ON THE MEASUREMENT OF SIX PRIMARY BASE LINES.

Discussion.*

By Messrs. J. A. Ockerson, Horace Andrews, and Noah Cummings.

Mr. Ockerson. J. A. Ockerson, M. Am. Soc. C. E. (by letter).—Mr. French's account of the progress made in the use of steel tapes in geodetic work is interesting and valuable. The old-time methods of using bars or rods for the measurement of base lines were both laborious and expensive, and, as a consequence, the intervals between such lines in a system of triangulation were entirely too long.

Professor Woodward's paper on long steel tapes, and the discussion thereon,† gave an account of the use of steel tapes in connection with the triangulation work of the Coast Survey.

The official reports of the Mississippi and Missouri River Commissions, of earlier date by several years, gave accounts of the steel-tape work on their respective surveys, which included high-grade triangulation, where the length of the triangle sides and the closure required were such as to compare favorably with so-called primary work.

In August, 1880, the writer made use of a steel tape in the measurement of a base line 6 663 ft. long, opposite Grafton, Ill., in connection with the triangulation in that vicinity, and, although the equipment was deficient in many respects, the results obtained were

* This discussion (of the paper by Owen B. French, M. Am. Soc. C. E., printed in *Proceedings* for October, 1907), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† *Transactions*, Am. Soc. C. E., Vol. XXX, pp. 81-107 and 638-652.

FIG. 1.—MEASURING FORT SNELLING BASE LINE WITH 300-FT. STEEL TAPE. REAR END OF TAPE.

FIG. 2.—FRONT END OF TAPE, FORT SNELLING BASE LINE, SHOWING TENSION DEVICE.

Mr. Ockerson.

such as to establish practically the use of the steel tape in the extensive triangulation work which followed, from the mouth of the Ohio to the headwaters of the Mississippi.

The writer believes this to be the first use of the steel tape in refined geodetic work, at least in the United States.

The methods of manipulation in the field were modified and improved, in the interest of both economy and accuracy, as experience developed the defects.

Mr. O. B. Wheeler, of the Missouri River Commission, is entitled to the credit of introducing an accurate tension adjuster, which is described in the Annual Report of that Commission for 1886.

The use of metal strips on which to mark the graduated extremities of the successive tape lengths was also developed in the river surveys. This was a very important step, as it virtually permitted the graphical results of each tape length to be transferred from the field to the office, where the discussion could be taken up at leisure.

Mr. Marshall, in connection with the survey of the Red River, made a number of improvements, among which was the use of two tapes of different metals at one and the same time.

The greatest source of error in the use of the steel tape lies in failure to secure its temperature, as it is much more sensitive to changes than the thermometer used in connection therewith. The writer had in mind a method of diminishing the difference between the two, by the construction of a thermometer with an elongated bulb of the same material as the tape, and perhaps extending the metal along the back of the glass scale tube. The invar tape, apparently, eliminates much of the objection to the steel tape incident to changes of temperature.

In the Mississippi River triangulation, the measurement of base lines became so easy that the general practice called for a base line at intervals of about 12 triangles. That is to say, the instrumental errors of centering both instrument and target, and errors of pointing, were larger than the errors of base measurement, hence such errors were largely localized by the use of frequent bases.

In the writer's opinion, each tape should be standardized by measuring a primary base the length of which has been determined by a Repsold or other refined base-measuring apparatus. The measuring should be done under conditions and by methods identical with those to be used in the measurement of a new base, in preference to relying on a laboratory determination of the length of the tape.

Table 9 gives some results of base-line measurements with steel tapes on the Mississippi River Triangulation. The measurements were generally made in the morning, before sunrise, when changes of temperature were not very rapid. The lengths are given in round numbers, omitting the decimals.

Mr. Ockerson

TABLE 9.—SOME RESULTS OF BASE-LINE MEASUREMENTS WITH STEEL TAPES ON THE MISSISSIPPI RIVER TRIANGULATION.

Location.	Length of base line, in feet.	Discrepancy between successive measurements.
New Boston	18 066	1 in 759 000
Rapid City	5 624	" 594 917
East Dubuque	7 105	" 346 965
Cassville	7 001	" 266 412
Prairie du Chien	5 312	" 265 000
De Soto	6 486	" 929 300
Trempeleau	5 223	" 2 396 000
Wabasha	6 783	" 740 000
Red Wing	5 379	" 8 400 000
Fort Snelling	5 400	" 517 000
Monticello	5 401	" 1 475 000
Rice	5 700	" 2 969 000
Brainard	5 400	" 438 400
Aitkin	4 798	" 1 304 000

The method of handling the tape is shown by the photographs on Plate I. Single measurements of lines 1 mile long have been made in 28 min. In the lines cited, no effort was made to secure a very high degree of accuracy, but simply to keep within the limit of discrepancy between two measurements, 1 in 250 000, as prescribed.

Mr. Andrews.

HORACE ANDREWS, M. AM. SOC. C. E. (by letter).—The engineering profession is indebted to Mr. French for his clear and useful exposition of the practical adaptability of invar to field use.

There would seem to be little left to be desired in base-measuring apparatus, now that temperature corrections are so well eliminated. The history of base measurement has been one of constant struggle against the uncertainty of temperature corrections. At present, the use of iced bars, steel tapes, and night work, enables high precision to be attained, together with speed and economy passing all former experiences. A further and most important advance, from the economical standpoint, is now assured through the use of this wonderful alloy.

Previous to the six base lines referred to by Mr. French, some 35 base lines of primary importance had been measured in the United States. The three earliest, one in 1834 and two in 1844, measured with the Hassler apparatus, had an average probable error of $\frac{1}{214\,000}$. Between 1847 and 1873, seven bases were measured by the Coast Survey, with the Bache-Würdemann apparatus, an average probable error of $\frac{1}{437\,000}$ being indicated. Similar apparatus used by the United States Lake Survey, from 1870 to 1875, gave the average probable error of five bases as $\frac{1}{730\,000}$. The Repsold apparatus then came into use on the Lake Survey, three bases being measured with an average probable error of only $\frac{1}{1\ 075\ 000}$. The United States Coast and Geodetic Survey, after 1873, measured eight bases with various apparatus. Two of these, measured in 1891-92 with the iced bar and steel tape, showed excellent results. The average probable error of these eight bases was $\frac{1}{857\ 000}$. Then came the phenomenal achievement of 1900, when the United States Coast and Geodetic Survey, having commissioned one field party to measure nine bases in a working season, the aim being to secure a precision of about $\frac{1}{500\ 000}$, obtained, not only unprecedented economy of time and money, but an average probable error of only $\frac{1}{1\ 150\ 000}$. This success was due to the use of the iced bar and steel tapes, as mentioned by Mr. French; the advantages of invar are those pointed out by him, and are irrespective of the higher precision which was incidentally obtained. Mr. Andrews.

Obviously, invar will be an admirable material for precision leveling-rods. Its use for pendulum rods was one of the first suggested. A pendulum, supported by an iron rod, will change its rate about 1 min. a week, if subjected to a change of temperature of 30° fahr., but, with invar having the coefficient given by the author, the change of rate would be only 2 sec.

It would be interesting to know the exact proportions of nickel and steel entering into the composition of the invar tapes. In view of the fact that the coefficient of expansion given in Table 2, is only one-half that found in Guillaume's 36% of nickel alloy, it would seem that some change must have been made in the proportions, and if the name, "invar," is to be adopted, it would be well to have it apply to a definite nickel-steel alloy; at present, there are two "invar" alloys, one of which has only half the invariability of the other.

Engineering measurements in general must be made under all conditions of temperature, and it will be of great advantage to be independent of temperature corrections. The writer has found it very advisable to keep temperature notes for important steel-tape measurements, and correct, under the rule of 0.01 ft. in 100 ft., for each 15° fahr. In the surveys for the Boston Back Bay tract, it has been stated that brass tapes were used, and with the correction of 0.01 ft. in 100 ft. for 10° fahr. change of temperature. With invar, the correction would appear to be 0.01 ft. in 100 ft. for each 44° fahr., so that temperature corrections would in general be negligible.

Noah Cummings, Assoc. M. Am. Soc. C. E.—The low coefficient of expansion of invar makes it a most desirable material for a tape, Mr. Cummings.

Mr. Cummings. and the results given in the paper show that it has proved very satisfactory for base-line measurements.

It could be used to advantage for measuring base lines for city or bridge triangulations, or wherever great accuracy is required in base-line work. However, there seem to be serious objections to the use of the invar tape for ordinary city surveying, even though its low coefficient of expansion would practically eliminate temperature corrections. It is more easily bent and less elastic than steel, and, according to the makers, requires a reel 16 in. in diameter. Thus great care must be taken in handling it, and it would need to be retested every time someone happened to run into it while a measurement was being made or in case it was stepped on, either of which may easily happen in ordinary street surveying.

It is a question whether surveying needs to be done more accurately than the holding of the points established by the survey. When street corners are marked by stone monuments placed near the surface of the sidewalk, there is a possible movement due to frosts and to excavations for building and street construction purposes. If the invar tape should be developed so as to be more easily handled and then come into use for city surveying, a greater degree of accuracy would be obtained; but, to secure the benefits of this, the points established would have to be marked on stone or concrete monuments either below or extending below the frost line.

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PAPERS AND DISCUSSIONS.

MUNICIPAL REFUSE DISPOSAL: AN INVESTIGATION.

Discussion.*

By Messrs. W. M. Venable, Albert A. Cary, E. H. Foster, B. F. Welton, C. Herschel Koyl, Louis L. Tribus, and H. Norman Leask.

W. M. Venable, M. Am. Soc. C. E. (by letter).—In this valuable paper the author proves that the refuse of the Borough of Richmond, New York City, contains sufficient calorific material to enable it to be burned without offense, and without using auxiliary fuel. He also presents data regarding forty incinerating plants in Great Britain, with the object of determining the best features of design for use in a proposed plant. The investigation which led to the conclusion that it is desirable to burn all the refuse in one incinerator, if that is found practicable, is not given in the paper, nor is there any investigation of the merits or demerits of incinerators of American design. The writer is of the opinion that, in the United States, it is seldom desirable to reburn all refuse, including the ashes from private houses and other buildings, and would like the author to present in detail the data upon which this determination, which preceded the investigation reported in the paper, was based. Mr. Venable.

Whether or not the method of destroying all wastes in one set of furnaces will be found the best for municipalities generally, engineers are indebted to Mr. Fetherston for his thorough work in ascertaining the quantities of garbage, ashes and rubbish, and their calorific value, in what may be taken as a representative district. It is remarkable,

* This discussion (of the paper by J. T. Fetherston, Assoc. M. Am. Soc. C. E., printed in *Proceedings* for November, 1907), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Venable. also, that the best summary of British practice in refuse disposal is found in this paper, by an American engineer, for use in America. Too much praise can hardly be given for the judgment shown in the preparation of the various tables, although Table 1 was prepared so long ago as to make it necessary for the reader to guard himself against the error of assuming that it contains all the data now available on the subject with which it deals. The work of Messrs. H. de B. Parsons, Rudolph Hering, W. F. Morse, and others has been published since 1904.

From this paper and other available data, it is safe to assume that, in almost any municipality, if all the household refuse is collected and brought to one place, the mixture will contain sufficient calorific energy to make it practicable to burn it without admixture of other fuel, and to permit the generation of some steam for power purposes from the heat in the gases of combustion. It does not follow from this, however, that such collection and disposal is the most advisable. It can hardly be granted, as a general proposition in cities, that it is impossible to collect ashes in such condition that sanitary disposal of them without reburning is impracticable. If such is granted, as appears to have been done in the present case, there would still be reasons for considering separate collection and burning in separate parts of an incinerator, keeping ashes separate from garbage and refuse, both for sanitary reasons and for convenience and economy in actual burning.

If the reburning of ashes is to be decided from considerations of economy only, it should be regarded entirely apart from the disposal of other wastes, for the introduction of ashes into the garbage makes their disposal much more costly than otherwise, even if it is necessary to furnish a considerable quantity of coal to assist in destroying the garbage.

While ashes from household fires contain much combustible material, they do not contain enough, as a rule, to make up for the cost of stoking them through a crematory, not including plant charges; and, unless a very great reduction in weight is secured by reburning, there will be no saving in total haul by the burning process. Generally, the weight of ashes passed through a crematory is not very greatly reduced, although the weight of rubbish and garbage is very much decreased by burning. There may be cases, however, where a furnace can be located at the center of a district, and the haul to the dump is very much longer than that to the furnace, in which cases the saving in haul will more than counterbalance the cost of dumping, stoking, interest and depreciation on plant, and reloading for haul to the dump.*

* This matter is discussed in the writer's book, "Garbage Crematories in America," John Wiley & Sons, 1906. In this book will also be found descriptions of every type of crematory installed in the United States, reference to every United States patent of interest in this field, and a list of the more important and representative plants installed by each builder of such works in the United States.

In the United States it has been customary to dispose of ashes separately from garbage, from motives of economy, and furnaces for the disposal of garbage or refuse, or of both combined, have been designed with the expectation that ashes would be excluded. It is practicable to burn these materials properly without forced draft, and several builders of crematories have accomplished this successfully, at prices of disposal per ton quite as low as those obtaining in England for the mixed refuse; but crematories operating on natural draft can be abused more readily than those using forced draft, and, consequently, when handled by the ignorant persons who are so often placed in charge, the furnaces have received the blame that ought to have been charged against the persons in authority. Of course, very many crematories of poor design, and crematories attempting to burn materials for which they have not been fitted, have been installed, and the blame has not always been with the operator. Crematories of the so-called American design are much cheaper to build than those of the British type, as they require no boiler plant, or power auxiliaries. They will consume successfully garbage and rubbish of a character which cannot be burned in those of the British type, and are very economical in the use of labor in stoking. Therefore they ought not to be condemned, or left out of consideration in selecting a method of disposal, but should be installed where economy shows that they will be most economical in the long run; and proper precautions should be taken that they are operated so as not to produce a nuisance. Mr. Venable.

On the other hand, it is practically impossible to burn ash-bin refuse with natural draft. The reason is, not that a strong enough natural draft cannot be obtained, but that the constant opening of doors for stoking, on account of the large proportion of ashes to actual fuel in the mixed refuse, admits to the furnace too much air for proper combustion. This reduces the draft and also causes the production of foul odors in the chimney gases. It requires so much more head to create a proper draft through a mixture of ashes than through a mixture of rubbish that it is possible to burn the rubbish without offense, on natural draft, even with doors frequently open, although it is not possible if a large proportion of ashes is introduced into the mixture to be burned. Thus, practically all British incinerator builders have been compelled to adopt forced draft because they reburn ashes, and to install boilers in order to develop power to obtain it. When forced draft is used, the stack should be designed merely to carry off the gases of combustion, not to produce any portion of the head across the grates. Thus, when the stoking doors are opened, there is no tendency to draw the air in the stoking room into the furnace; but, on the other hand, there may be a tendency for the heated gases within to come out through the open doors, as observed by Mr. Fetherston in several British installations. Forced draft, subject to close

Mr. Venable. regulation, is preferable in any installation, and is a very great safeguard against the admission of too much air into the furnace, above the fires; but it is not the only way in which this can be safeguarded, and, in many plants, especially in the smaller towns, the advantages to be derived from the installation of a boiler are not as great as the disadvantages.

These remarks may be considered as not properly applying as a discussion of Mr. Fetherston's valuable paper, one of the premises of which is that the ashes are to be burned. While fully recognizing this, as a condition precedent to his inquiry, and having no quarrel with it, in the case of the Borough of Richmond, the writer has ventured these remarks as perhaps of some interest to others, for conditions which may differ from those stated in this paper.

Mr. Cary. ALBERT A. CARY, ESQ.—This paper, viewed from the standpoint of a furnace and fuel specialist, is of great interest to the speaker, who, having had considerable experience in burning various fuels of low calorific value and also fuels carrying large percentages of moisture, such as spent tan bark, wet refuse wood-pulp shavings, spent licorice root, bagasse, etc., can well appreciate the difficulties encountered in burning wet municipal wastes; and burning them so as to obtain sufficient heat for steam-making, which heat is in excess of that required to evaporate the moisture contained in the fuel, and to dissociate the fuel (thereby liberating the volatile gases it contains, which action is necessary before these fuel constituents can burn).

If any fuel be dried and an analysis be made of its chemical composition, then, by the use of a modification of the well-known Dulong formula, a determination of the calorific value of the sample analyzed can be made. This may or may not be of use in furnace determinations, depending on the nature of the fuel and the value of the sample as a fair representative of the entire mass of fuel consumed.

It is no easy matter to obtain a representative sample of the entire fuel consumed during a test, even when the fuel is fairly uniform in quality, but when its quality is of a very variable nature, such as in refuse-burning plants, the difficulties in obtaining a small sample of fair average value become almost insurmountable. Aside from this difficulty, after making calculations from the chemical analysis of a dried sample, one does not obtain a true calorific value of the fuel, as this process of determination assumes that the combustion is wholly an exothermic process, that is, one producing heat with no heat absorption occurring for internal or external reactions. Such endothermic actions always take place in the process of combustion, as it requires heat energy to break up solid masses of fuel and liberate and split up the hydro-carbons, to say nothing of the energy required to evaporate the moisture, both on the surface and contained hydroscopically.

Mr. Cary.

For this reason, when careful tests are made, the calculated fuel values are found to be higher than those obtained by using the oxygen fuel calorimeter; the difference between these two values indicates the heat energy absorbed by endothermic reactions.

Coming now to the fuel calorimeter, reference will be made only to the work done in the Mahler bomb. By a proper manipulation of this apparatus, there is no difficulty in determining the true value of the sample tested, and the results obtained will require no corrections for chemical endothermic actions; but here, also, there is difficulty in obtaining correct samples, representing a fair average of the whole fuel mass, and it must be remembered that the quantity of fuel tested weighs only 1 g. (that is, less than 0.04 lb.).

The great difficulty in obtaining the true calorific values of the refuse by either of these methods, therefore, can be well appreciated, and the question naturally is: How can this most important value be determined?

The answer is prompted by a somewhat extended experience in making furnace investigations leading to accurate heat balances. Heat balances are usually obtained by calculations made from the analysis of the fuel, the quantity of fuel used, the analysis of the products of combustion, and a proper consideration of the various furnace losses.

By a somewhat reversed method of calculation, made from a series of observations, the chemical composition of the fuel may be obtained, and from it the unknown quantity to be determined, then, with a very fair degree of accuracy, its value may be found.

The accuracy of such a determination is, unquestionably, far greater than may be obtained by any system of sampling when such a mixed fuel as municipal wastes is used.

This method was used in the work of Mr. C. E. Stromeyer, referred to on page 957,* but he did not carry his work far enough to obtain sufficiently satisfactory results.

The furnace gas analysis becomes a most important matter in such test work, and the mere finding of the percentages of CO_2, O, CO and N, by difference, by the use of the ordinary Orsat apparatus, will not give sufficient information, as experience has taught the speaker that in such work it is necessary to determine the free hydrogen and hydro-carbons as well.

Mr. Stromeyer also relates, in his report, the failure of his high-temperature measuring apparatus, which furnishes most important information. The speaker is continually using such apparatus, without trouble, in furnace tests where much higher temperatures exist.

The speaker does not wish to be understood as criticizing Mr. Stromeyer in this work, on the contrary, he regards it as very much

*Proceedings, Am. Soc. C. E., for November, 1907.

Mr. Cary. in advance of any testing work previously done in refuse crematories. He merely wishes to indicate that this is the most reliable way of obtaining this much sought for information, when proper testing is done.

To obtain data needed to make such fuel determinations, one does not require any other apparatus than that used in making complete and exhaustive furnace tests, but careful refinements must not be neglected, both in applying and using the apparatus and in having them all carefully calibrated.

The work done by Mr. Fetherston, as shown in this paper, to obtain such information, by fuel sampling methods, is certainly highly creditable, and the amount of work involved appeals to the speaker strongly, as he knows by experience what it means.

Concerning the large percentages of moisture held in fuels, the speaker has profitably passed very wet fuel between a pair of large cast-iron rolls, with rough faces, one roll being of a little greater diameter than the other. These rolls, both running at the same number of revolutions per minute, were held together by large springs which allowed them to separate when solid chunks reached them.

In this way a large quantity of the contained moisture was squeezed out, and higher temperatures were obtained in the furnace, as well as better combustion, for the furnace is the poorest place in the world to evaporate water.

Mr. Fetherston's statement of the requirements necessary for burning wet fuel, on page 972* which, as he states, originated with Professor Thurston, may be found in the *Journal* of the Franklin Institute for 1874, where it will be found to refer especially to spent tan and wet saw-dust.

For the combustion of moist fuel, the highest furnace temperatures possible are most essential, and that requirement is one of the weakest features in general garbage incinerating plants. It is firmly believed that much profitable development is possible in this direction.

The disposition of the highly heated surrounding surfaces mentioned is a matter of much importance with such fuel, and combustion chambers must be proportioned to the amount of gaseous matter and moisture given off by the fuel.

The speaker can hardly admit the statement indicating that combustion should be retarded and limited by spots of dry fuel forming on the grate and burning to expose wet fuel, thereby stopping combustion. Such conditions should never exist as they indicate bad design.

To obtain the most desirable results, the combustion of the fuel should constantly be accelerated.

Pre-heated air, introduced under some pressure, to offset its dilated

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condition, will assist in producing such results, as is noted by Mr. Fetherston. Mr. Cary.

Steam jets should certainly be avoided as much as possible, as there is altogether too much steam given off from the fuel in the furnace, and steam has a cooling effect on the fire-bed. To obtain the best results, the steam used to disintegrate the clinker should be superheated.

The speaker cannot agree with Mr. Fetherston when he places the minimum desirable furnace temperature as low as 1 250° fahr., which is dangerously near the lowest temperature at which some of the gases found in the furnace will ignite. Such a temperature will surely be followed by most imperfect combustion.

A furnace should not fall below 1 800° fahr., as experience proves that, under lower temperatures, both furnace and boiler efficiencies drop. Further, 2 000° fahr. is too low for a maximum temperature, as the speaker's best furnace results have always been obtained with temperatures of 2 500° fahr., or greater.

If a furnace is properly designed and built, there is no reason why it should not be durable under a temperature of 2 500° fahr., and with destructor furnace conditions.

The speaker's experience, of many years in furnace work, has taught him that proper provision for great expansion and contraction is frequently neglected, and, also, that high-grade refractory materials are not used as much as they should be, and that high-grade furnace masons are not employed, but, where all these requirements are met, the durability of furnaces is greatly increased.

On page 970,* it is noted that Mr. Fetherston assumes a combined furnace and boiler efficiency of 50 per cent. By the system of testing referred to in this discussion, the exact efficiency of the furnace can be obtained. The information thus obtained will also point out definitely the exact causes of inefficiency, and thereby lead to a rapid, rational, and scientific development and improvement of the system of garbage incineration, and the time is certainly favorable for work of this nature, as shown by Mr. Fetherston's earnest work and careful investigation of existing conditions.

E. H. Foster, M. Am. Soc. C. E. (by letter).—The valuable data which Mr. Fetherston has presented in this paper will certainly be appreciated by engineers who have occasion to explore this comparatively obscure field, and it is certain that the paper will prove an important addition to the Society's *Transactions*. Mr. Foster,

Attention is called to the quotation on page 972* from Professor Thurston, giving the requirements for success in burning wet fuel: To insure that "the rapidity of combustion may be precisely equal to

**Proceedings*, Am. Soc. C. E., for November, 1907.

Mr. Foster. and never exceed the rapidity of desiccation" offers a condition which is to be steadfastly striven for, but which, unfortunately, can only be obtained under the most ideal conditions, and one can only hope to fulfil it when the combined collection of the city's waste is to be burned. So long as furnaces are required to burn garbage only, or garbage and rubbish, special provision must be made for carrying out the above requirements. When garbage alone is burned, fuel must be added to support combustion. Professor Thurston's remarks show why coal should be used, and not oil or natural gas, since it is the heated mass in the coal, and not the volatile matter, which accomplishes the drying process. The quality of coal need not be high; in fact, coals of the poorer quality, containing high percentages of ash, are really more suitable for this purpose. When garbage or rubbish have to be dealt with, without ashes, some other means must be adopted for preparing the garbage for burning, and, whatever method is used, it must be carried out inside the furnace, thus it becomes a part of the detail of the design.

It must not be considered that the chief desideratum is to obtain the highest possible temperature in any part of the furnace. Such an impression would be entirely wrong. The temperature, on the contrary, must be maintained between certain moderate limits, preferably between 1 800 and 2 000° fahr., but with a minimum never less than 1 250° fahr. The disadvantages of too high a temperature may be stated as:

Excessive cost of repairs;

Melting of the dust and clinker, causing it to stick to the fire-brick linings inside, and loss of time and labor in cutting out, periodically, by hammer and chisel, the slag-like accumulations;

Discomfort to the operators in removing the clinker from the furnace.

The limit of low temperature is reached at the point where the gases of combustion cease to be dissociated and oxidized. It is necessary, then, to maintain a temperature well above this, thus rendering them thoroughly innocuous, and without which no process of destruction may be termed sanitary or without nuisance.

In the study of designs of various furnaces intended for destroying refuse, attention has been drawn to the conservation of the heat by various recuperating devices, such as an air heater for extracting the heat from the flue gases and transferring it to the air which is being fed to the furnace, and the method of cooling the clinkers by taking up their heat in the same air going to the furnaces. All these devices, which resemble the conventional economizers and air heaters used in connection with steam-power plants for securing higher efficiency and economy in fuel consumption, serve an entirely different purpose in the case of destructor furnaces. They are rendered necessary in order to insure a high minimum temperature which must at all times be maintained, the character of the material fed to the grates being of such a nature that these precautions are necessary. Mr. Foster.

Whereas the steam generated from the plant represents a valuable asset, and, in some instances, can be made to do useful work, doubtless there will be many cases where, in the absence of a suitable method of utilizing this power, the steam must be blown off and wasted. As it is the condition in the furnace which is of most importance, even if the steam generated is wasted, the devices for recovering the heat must be used.

On page 978* Mr. Fetherston suggests a further improvement which might be made in the Westmount plant, namely, to utilize the heat contained in the hot clinker for raising the temperature of the air for combustion. This idea is being carried out in the city plant now under construction.

It is a mistake to rely on the recommendation that the conversion of the power of the destructor plant into electrical energy is the most suitable outlet for that power. Mr. Fetherston mentions the pumping of water or sewage as an appropriate use. A still more appropriate use would seem to be the manufacture of ice, for which such a plant is strikingly well adapted. For instance, with an absorption ice machine, 9 lb. of ice may be readily procured by the burning of 1 lb. of coal under the grates of the boiler, whereas, in a destructor plant of 50 tons capacity, 1 lb. of steam may be readily evaporated per pound of mixed refuse destroyed. A 50-ton destructor plant would serve a community with a population of approximately 40 000. By comparing these figures it will be seen that a 50-ton refuse destructor will produce 50 tons of ice per day, or an allowance of 2½ lb. of ice per capita, which would be a liberal amount.

An important feature in the design of furnaces is the avoidance of smoke; this can only be accomplished by isolating completely the furnace and combustion chamber from the water-heating surface connected with the boiler, as the chilling effect produced by contact of the partially cooled consumed gases against the cold surface of the tube containing water will suppress complete combustion and result in a smoky chimney.

B. F. Welton, Assoc. M. Am. Soc. C. E.—The speaker has followed with great interest the development of the work done by Mr. Fetherston, the results of which are so admirably presented in his valuable paper. Mr. Welton.

The speaker's relation to this work, as stated by the author, has been in connection with the determination of the calorific values incorporated in the text of the paper. The purpose of this discussion,

* *Proceedings*, Am. Soc. C. E., for November, 1907.

Mr. Welton. therefore, is to describe in some detail the methods used in making the calorific tests and proximate analyses, in order that the reader may be enabled to form his own estimate of the relative value of those results as compared with similar tests of other and more homogeneous materials used as fuel.

It is also desired to record the results of a series of chemical analyses of the component parts of the refuse made in the same laboratories by Professor Stephen F. Peckham, Member of the American Chemical Society, whose assistance has been highly appreciated.

The primary purpose of the experiments was to provide fundamental data from which could be determined the feasibility of the sanitary disposal of the wastes of the Borough of Richmond by self-combustion, in a refuse destructor of the same general type as used in Great Britain.

Inasmuch as the matter of heat utilization and power production was to be taken up ultimately, in connection with the disposal of the refuse, the results of the experiments were also to be considered as possibly affecting the design of the destructor. If the results of the calorific tests should show that the material was not suitable for self-incineration, it was hoped that the chemical analyses might provide the necessary information for determining some alternative method of sanitary disposal. On the other hand, if the material should be found suitable, the chemical analyses might furnish additional data for the study of means for the prevention of possible nuisance by the escape of the products of combustion, or for the recovery of commercially valuable material.

After the conclusion of trial calorific tests on two sets of samples to ascertain what methods of handling the material in the laboratory would secure the desired uniformity of results, a consultation was held between the author and the speaker to define the scope of the experiments.

It was decided:

First.—That, if the experiments were to be conclusive, they should be extended over a period of at least a full year, thus showing the entire seasonal variation in the character of the collections, which variation, it was thought, might be sufficient to interfere, perhaps, to a serious degree, with the successful operation of a destructor;

Second.—That the samples should be taken with sufficient frequency and in such manner as to be truly representative of the collections, both as regards the character of the material and the period covered.

It was finally settled that the samples submitted to the laboratory should represent the daily collections of the Bureau of Street Cleaning for a period of about two weeks, or a half month.

The primary sampling from the actual collections, as well as the initial preparation of the half-monthly samples, was to be made under the direction of Mr. Fetherston. Mr. Welton.

The sampling, as described in detail by the author, consisted in the selection of representative material which was subsequently separated, by sieves and hand-picking, into six general classes as follows:

1.—Garbage,
2.—Coal and cinders,
3.—Rubbish,
4.—Fine ash,
5.—Clinker,
6.—Incombustible material.

The garbage consisted of vegetable and animal matter, etc., such as ordinarily collected from dwellings.

The coal and cinders was the better portion of the stove and furnace wastes of the district.

The rubbish consisted of a variety of materials, such as paper, excelsior, rags, fibrous material, etc.

The fine ash was the material from the general collections which would pass through a screen of $\frac{3}{8}$-in. mesh, and consisted principally of the finer residue from domestic fires.

The clinker was that contained in the residue from domestic fires, and those of schools, churches, etc.

The incombustible material was largely glass, metal, stone, bricks, etc.

The initial preparation of the samples comprised the reduction of large quantities of material of the several classes by quartering, the evaporation of nearly all the moisture, and the rough pulverizing of all samples to effect a uniformity which would serve to make the samples submitted to the laboratory truly representative.

The weight of these samples was approximately:

1 lb. of garbage (dry);
2 lb. each of coal and cinders, clinker and fine ash;
$\frac{1}{4}$ lb. of rubbish.

The condition of the samples, as they arrived at the laboratory, after going through this preliminary process, was about as follows: The garbage, in the majority of cases, was fairly dry, but soft and greasy; most of it would pass a sieve of $\frac{1}{2}$-in. mesh, and, while the odor was decidedly in evidence it was not offensively so. Nearly all the coal and cinders would pass a sieve of $\frac{1}{8}$-in. mesh, and showed a large proportion of unburned coal.

The fine ash and clinker were in about the same condition as the coal and cinders, except that the difference in the quantity of carbon present was plainly evident from the color and general appearance.

Mr. Welton. The rubbish presented the appearance of shredded rags, paper, etc. No incombustible material was tested, for obvious reasons.

Upon arrival at the laboratory, each sample was immediately placed in a wide-mouthed glass jar with a ground-glass stopper, and as soon as convenient thereafter a careful determination was made of the contained amount of moisture. This operation was conducted using about 10 g. of garbage and about 5 g. each of the other samples.

The whole of each sample of garbage and rubbish was then made to pass a sieve of No. 20 mesh by repeated grinding in a small pulverizer of the coffee-mill type. The coal and cinders sample was pulverized in a laboratory ball mill until it would all pass a No. 40 sieve. The clinker and fine ash were treated in the same manner.

The samples were then replaced in their respective glass jars and thoroughly mixed by agitation.

Proximate analyses were next made, determining again the moisture, and, in addition, the volatile matter, fixed carbon, and ash. For these determinations, the following weights of material were used:

Garbage	about	1.5 g.
Coal and cinders	"	2.0 "
Clinker	"	2.0 "
Fine ash	"	2.0 "
Rubbish	"	0.5 "

These quantities of the several materials were taken at random directly from the jar containing the whole sample, since it was found that practical duplication of results could readily be obtained without further reduction in size or quantity of the sample.

All determinations of moisture were made by using an electric oven kept at a constant temperature of about 180° fahr. The coarse samples were allowed to remain in the oven for about 18 hours, but only about 1½ hours were necessary when the samples were in the pulverized condition. The volatile matter was determined by placing the dried material in a small, covered, porcelain crucible, over a three-flame Bunsen burner, care being taken that all the carbon deposited during the combustion of the volatile hydro-carbon was afterward consumed by the Bunsen flame. (Platinum crucibles were first used for this work, but some constituent of the coal and cinders, which was later discovered to be tin, probably from tin cans, re-acted with the platinum, ruining the crucibles, and their use was abandoned).

The fixed carbon then remaining in the crucible was next reduced to ash by open burning over the same Bunsen flame until no loss in weight occurred.

The percentages of the various determinations were reduced by calculation to the basis of the condition of samples as they were received at the laboratory, and the garbage analyses were still further modified to represent the conditions in the original sample before evaporation of any of its moisture, a record of the evaporative tests being sent to the laboratory with each sample. Mr. Welton.

The calorific values were determined by the Mahler bomb calorimeter, which provides for the combustion of the material in the presence of oxygen at a pressure of 25 atmospheres.

The tests were made using the following weights of material:

Garbage	0.80 to 1.00 g.
Coal and cinders	0.50 " 0.75 "
Rubbish	0.35 " 0.50 "
Fine ash	0.50 " 0.75 "
Clinker	0.50 " 0.75 "

These values, as obtained by the actual tests, were reduced to values per pound of dry sample, original sample, and combustible, in that order, using the corrected proximate analyses as a basis. These are the figures that appear in Tables 6 and 7.

There was no difficulty in securing satisfactory combustion, except in the tests of "fine ash" and "clinker," in which the percentage of inert matter was so high that it prevented ignition of the combustible portion of the sample by the ordinary means. In these cases, therefore, a small amount of naphthaline was introduced with the sample to start the combustion, and a deduction, representing the calorific value of the naphthaline used, was made subsequently.

The residue from the combustion of the garbage was hard, vitreous, and invariably in the form of small globules of a brownish black color. That of the coal and cinders was naturally about the same in appearance as the ash of anthracite coal, while the rubbish left little more that could be seen with the eye than a stain on the combustion tray.

At the beginning of the experiments, tests were made in duplicate on all samples until it became evident that the differences in results, as shown by the duplications, were well within the variation that might easily occur in the primary selection of representative samples. In amount, these differences were generally less than 1% of the calorific value of the dry material. By this time, also, the uniformity in the character of each class of material, as shown by the calorific value per pound of combustible, began to be noticeable, and it was observed that this value would serve to dctect errors in manipulation and computation as well as to indicate the occasions when duplication was required. As a consequence, tests on the same sample were rarely repeated thereafter, unless the value per pound of combustible was at some variance with the average of the other tests already made.

To those who are not familiar with the calorific values of the staple fuels, such as anthracite and bituminous coals, it may appear that no great confidence should be placed in the results of these tests

Mr. Welton. on material which would naturally be expected to vary widely in character. As a matter of fact, the experiments have shown a uniformity of character in the material which is all the more remarkable in that it was not anticipated. Indeed, now, when all the data are at hand, the conclusion might easily be drawn that in the instances where the largest variations in calorific values per pound of combustible occur, this variation is more likely to be due to the difficulty of obtaining representative samples from the collections than from actual differences in character.

Moreover, few who have had no occasion to study the matter of analyses and calorific tests of coal are aware of the variation in fuel value of its combustible portion or what is known as "pure coal."

In this respect the figures in Table 12 are of interest. These are deduced from:

First.—The report of the coal-testing plant of the United States Geological Survey at St. Louis, in 1904;

Second.—From records of the Department of Water Supply, Gas and Electricity, at Mount Prospect Laboratory, New York City.

The chemical analyses made by Professor Peckham consisted of organic analyses of composite samples representing the collections of the entire period, and inorganic analyses of the residue from burning the same over a Bunsen flame. They will not be described in detail here, but the results of both series of analyses have been combined in Table 13.

These results would have been included in Mr. Fetherston's paper, except for the fact that their completion was delayed by pressure of more important matters in the laboratory, and they have only very recently become available.

The same reason also accounts for the relatively large percentage of undetermined constituents in the garbage sample.

Table 13 also shows a comparison between the calorific value of the samples, as calculated from the chemical analyses, and as determined by the calorimeter. The correspondence is believed to be sufficiently close to serve as a general verification of the entire work. The percentages in Table 13 are all computed on the weight of dry samples as a basis. The presence, in considerable amounts, of volatile hydrocarbons in the garbage and rubbish samples may be noted if the carbon, as shown by the chemical analyses, be compared with the fixed carbon of the proximate analyses reduced to a dry-sample basis.

In the determination of the moisture in the garbage samples for the proximate analyses, it is extremely probable that some of the lighter and more easily volatile hydro-carbons were driven off and computed as moisture. This is undoubtedly the reason why there is

Mr. Welton

TABLE 12.—RANGE OF CALORIFIC VALUES, SHOWN BY TESTS, AND THE PROPORTION SUCH RANGE BEARS TO THE AVERAGE VALUES.

Basis.	ANTHRACITE COAL.		BITUMINOUS COAL.				MUNICIPAL REFUSE (1906).					
	Department of Water Supply, Gas and Electricity, New York City.		U. S. Geological Survey.				Laboratory, Commissioners of Accounts.					
	From Pennsylvania, (56 samples).		From West Virginia only, (12 samples).		From West Virginia, Pennsylvania, Illinois, and Iowa, (23 samples).		Coal and cinders, from Borough of Richmond, (24 samples).		Garbage, from Borough of Richmond, (23 samples).		Refuse, from Borough of Richmond.	
	Range, in British thermal units.	Percentage of average value.	Range, in British thermal units.	Percentage of average value.	Range, in British thermal units.	Percentage of average value.	Range, in British thermal units.	Percentage of average value.	Range, in British thermal units.	Percentage of average value.	Range, in British thermal units.	Percentage of average value.
On the basis of the calorific value per pound of combustible..............	1 080	7.5%	879	5.7%	3 458	23.8%	1 960	13.7%	2 220	21.5%	1 640	19.0%
On the basis of the calorific value per pound of dry sample...............	6 379	50.0%	1 464	10.0%	5 342	42.0%	5 155	60%	4 265	51%	3 053	42%

Mr. Welton. not a closer agreement between the calorific values of dry samples of garbage, as calculated from the analyses, and as determined in the calorimeter. The oxygen is determined by difference, and the ash is the average of the proximate analyses reduced to the dry-sample basis.

TABLE 13.—CHEMICAL ANALYSES OF DRY COMPOSITE SAMPLES OF COAL AND CINDERS, GARBAGE, AND RUBBISH, REPRESENTING COLLECTIONS FOR THE YEAR 1905-06.

Constituents.	Coal and cinders.	Garbage.	Rubbish.
Percentage by weight of:			
Carbon	55.77	43.10	42.39
Hydrogen	0.75	6.24	5.96
Nitrogen	0.64	3.70	3.41
Oxygen	2.37	27.74	33.52
Silica	30.01	7.56	6.49
Iron oxide and alumina	8.98	0.41	2.03
Lime	1.21	4.26	2.26
Magnesia	Trace.	0.28	0.57
Phosphoric acid	None.	1.47	0.10
Carbonic acid	None.	0.59	1.49
Lead	Trace.	Sulphides, 0.20	0.52
Tin	Trace.		Trace.
Alkalies and undetermined	0.27	4.45	1.21
	100.00	100.00	100.00

CALORIFIC VALUES, IN BRITISH THERMAL UNITS.

	Coal and cinders.	Garbage.	Rubbish.
Calculated from above analyses	8 382	7 970	7 250
Average of calorimeter determinations	8 510	8 351	7 251

The calculations from the chemical analyses are made as follows:

$$62\ 100 \left(H - \frac{O}{8}\right) + 14\ 500\ C. = \text{British thermal units.}$$

Mr. Koyl. C. HERSCHEL KOYL, ESQ. (by letter).—This paper presents in admirable form the results of a very careful, systematic, and thorough study of the possibility of destroying by fire the mixed wastes of the Borough of Richmond in a manner innocuous, inoffensive, and not too costly.

The need of such an investigation was pressing, and its value not merely local, because the number of small communities in America, in which this problem is of first importance, is considerable and growing.

The technical question is whether the mixed waste contains enough combustible to be self-burning, at a temperature sufficiently high to destroy and not merely distil the volatile organic matter. Records show that, in England and on the Continent, a satisfactory disposal of mixed municipal refuse is made in this way, but it is also known that abroad there is less waste of edible matter than in the United States; and, therefore, before risking $60 000 of municipal money, it was the part of wisdom to determine the theoretical fuel efficiency of the waste of Staten Island. From the character of the examination and its completeness, Mr. Fetherston's results may be accepted with confidence, and also his conclusion that a destructor of the English type will burn the mixed waste of the Borough of Richmond effectively. True, the expense will not be small; but if the destruction of organic matter is complete and inoffensive to the neighborhood, a cost of from $1.00 to $1.50 per ton should not be prohibitive, in view of the fact that any other method of disposition would be extremely difficult in Staten Island. Mr. Koyl.

Regarding the limits of usefulness of these waste destructors: It has been proven by eight months' operation in Westmount, Montreal, that an average of 20 tons of mixed waste per day can be destroyed at a working cost of 31 cents per ton, and a total cost of 80 cents per ton; therefore, a population of 13 000 people is not too small to have the economical service of a destructor.

An upper limit, however, is reached when considering a city from which there is enough garbage to make profitable a modern reduction plant to separate the organic matter into grease and fertilizer. For instance, the City of New York makes satisfactory disposal of approximately 3 000 000 tons of mixed waste per year (70% ashes, 12% street sweepings, 12% garbage and 6% light refuse, by weight) at an average cost of 40 cents per ton. It would be folly to talk of putting all this material through destructors at a cost of 75 cents per ton.

Note should be made of a fact not mentioned by Mr. Fetherston, that in the "coal and cinders" which makes 27% of his total collection, or about 35% of his ash collection, more than half is not only burnable coal, but salable coal. This arises from the fact that most of the coal which gets into the ash-pit is undersized for the grate and falls through unburned and indeed unmarked by the fire. The writer has taken from many sample tons of Manhattan ashes an average of 20% of salable coal, from furnace size down, of which about half, after being washed, was indistinguishable from coal fresh from the mine. This means nothing in the Borough of Richmond, but it will be the determining factor in settling the method of final disposal where anthracite is used in cities which, at the same time, are large enough to make profitable the mechanical separation of the coal from the clinker and ashes. The process is not more difficult than the concentration of ore, and there is an average profit of about $2 per ton of recovered coal.

In the United States the advocates of reduction "utilize" garbage by separating it into water, grease, and fertilizer. The advocates of incineration "utilize" dry refuse by picking out its 30% of salable

Mr. Koyl. paper, rubber, etc., before they burn the remaining 70% of rubbish. And "utilization" is the keynote to successful policy in any large city.

It now costs New York $1 250 000 for the final disposition of its municipal wastes. It would cost $2 250 000 to put all the waste through destructors. It would cost about $200 000 to do it scientifically and save what ought not to be burned or buried, as follows:

				Cost.	Profit from utilization of rubbish and coal.
Garbage	360 000	tons	(contract)	$200 000	
Street sweepings....	360 000	"	at 40 cents	144 000	
Ash and clinker....	1 680 000	"	at 40 cents	672 000	
Rubbish	180 000	"	burned at profit.....	......	$40 000
Coal..............	420 000	"	recovered at profit..	......	840 000
				$1 016 000	$880 000
Less......................				880 000	
				$136 000	

or, the Department of Final Disposition would be almost self-supporting.

Mr. Tribus. Louis L. Tribus, M. Am. Soc. C. E.—This paper gives evidence of a great deal of work, and the speaker can say from personal knowledge that, in the Borough of Richmond, and along the lines described, there has been a vast amount of work which does not appear in the paper, yet its results will certainly secure great advancement in the art of refuse disposal.

Prior to the inauguration of the Greater City of New York, Staten Island (then becoming the Borough of Richmond) was occupied by a number of corporate villages and a great many small hamlets, the latter controlled by the usual "township" and "county" system of government, the incorporated portions by "village" form, with more or less intelligent management, as politics determined.

During the first four years following consolidation, little was done, other than to get accustomed to being a part of the great city.

On January 1st, 1902, under the revised charter, considerable home rule, and a borough president of character and ability, the first advance toward real progress was made.

Street cleaning and refuse disposal had been cared for, to a very limited extent, during the preceding four years, with a small force of men, but supervised by a man trained under Colonel Waring, who had the welfare of his subject at heart. The speaker was called upon by the borough president early in 1902 to act in both professional and executive capacities, and take charge of the public works and maintenance bureaus of the borough. He was given very free rein in securing betterments in plan and operation, but, on taking charge, insisted that he was to be free from politics in the work itself. Richard T. Fox, formerly in charge of the work in Richmond, as noted before, for the Department of Street Cleaning for the whole city, was placed in charge of the newly created "Bureau of Street Cleaning," which at this time came under the President of the Borough through the Commissioner of Public Works. Fairly liberal appropriations were made, so that, after careful plans had been laid, improvement became the order of the day. The first step noted was the banishment of garbage cans and refuse receptacles from the sidewalks and streets as far as possible, all such being removed by collectors from behind the buildings, the empty cans being then returned to their places. The next step toward efficiency and the establishment of *esprit de corps* among the men, was made by placing them all in uniform; the third step was to employ the men continuously throughout the year, so as to render service daily instead of spasmodically. This, of course, applied more to street cleaning, pure and simple, than to refuse collection which, formerly also, had to be more or less regular throughout the year. Mr. Tribus.

After some two years' service, Mr. Fox accepted a call to the City of Chicago, to show there what scientific and business methods could accomplish in the way of street cleaning and refuse disposal.

Mr. Fetherston, as a member of the borough engineer corps, had been assigned to specific work in connection with local scientific tests in refuse disposal, in which work he gave so good an account of himself that when Mr. Fox resigned he was selected to take the place of "Superintendent."

The paper describes very clearly the course taken, which has led to the recommendation and the actual construction of the first refuse destructor of this type in the United States. It is confidently hoped that in a few years this paper will be supplemented by one describing the destructor and telling of its efficient operation. That, however, will depend largely on the intelligence exhibited in its management, for the best piece of machinery may give poor results unless well handled.

In studying the refuse disposal question in the Borough of Richmond, it has been necessary to estimate very carefully the probable development of this specific locality, as its conditions are changing very rapidly year by year. It is not improbable that, within the life of the present generation, the whole island will be practically built up with residences, factories, stores, and valuable water-front improvements. This would mean that it would be impossible to find places for the burial of garbage, for maintaining ash dumps, and for any

Mr. Tribus. of the nuisance-producing plants for the destruction of garbage by low-temperature cooking. All experiments, therefore, in the past six years have been directed toward finding a process that would convert refuse, without nuisance, into some useful or innocuous material. The investigations which have been made so carefully by Mr. Fetherston and others assigned to the work from time to time, therefore, have been directed specially to this system; as, by process of elimination, all other systems were dismissed as not suitable for the probable local conditions of development, hence the conclusion that mixed refuse destruction promised more to the Borough of Richmond than did any other process; though it should be clearly understood that other systems might be more advantageous in other communities under different conditions, and time may prove that even in Richmond some different method may be evolved. In view of these explanations, the special studies, almost exclusively, have been directed toward acquiring information about and perfecting plans for mixed-refuse destructors, with the primary object of collecting materials at the lowest expense and converting them into an innocuous product without causing nuisance in the process, and it seems probable that the high-temperature system planned will accomplish the object desired. Up to the present time, theory and experiment indicate that, not only will the material collected have sufficient fuel value in itself to convert it into inoffensive slag, but that, in addition, there will be developed a liberal amount of heat units.

In the installation under construction, there is provided a bqiler which, it is expected, can be operated by the otherwise wasted heat units, so as to furnish ultimately all the power and light needed at the plant. If the results justify the expectations, it is probable, also, that the slag from the destructor will eventually be ground up, and, with an admixture of cement, be converted into paving blocks, which would have value for gutters and pavement in places where traffic is not specially heavy. That feature, however, is only being considered for the future, the present epoch being confined to what might be called the self-destruction of the refuse collected.

If success attends the operation of the new plant, it is expected that, ultimately, six or seven similar plants will be installed in other portions of the borough, as near as possible to the centers of collection; with one concession, however, to public sentiment, placing the destructors in manufacturing districts, near a railroad, or at the water front, rather than in residence localities.

Mr. Fetherston's paper covers the general phase of the refuse destructor, from the standpoint of economy in gathering garbage and other refuse and disposing of it finally. He has not gone particularly into the reasons why prompt final disposal of refuse is desirable. There are, perhaps, three reasons why every community should take care of this feature of urban life: First, that which appeals most popularly to citizens, the removal of refuse materials because they are obnoxious to the senses of smell and sight; second, because the keeping of decaying organic matters near habitations is generally supposed to breed disease; and, third, a reason which should have more consideration, though it has not been taken up extensively, that, during the heat of the summer, when the house fly develops and feeds and thrives on refuse, it is a very prolific distributor of disease. It is, to begin with, not a cleanly insect, and feeds on decaying matter; then, as likely as not, it proceeds to the nearest receptacle containing milk, for a drink, and not infrequently a bath; the combination is often too much for the fly, and it remains for a few moments or several hours floating around in the milk, leaving in it very often the germs of disease, which in turn thrive very readily in the milk and are taken into the human system. During the summer, the human system, particularly in infancy and childhood, is in excellent condition for the growth of disease germs in the intestines, and the various so-called summer complaints ensue. While probably no one as yet will claim that all intestinal diseases are caused by flies; by the process of elimination, in records that have been kept in certain places by intelligent observers, the fly can very fairly be charged with a great deal of the trouble. The mosquito has borne its share of public contumely as a dispenser of yellow fever; why should not the ordinary house fly be given credit for the work which undoubtedly it can do, and which many are beginning to believe it does do? If this is the case, the community that promptly removes and disposes of its decaying organic matters should, first, enjoy the presence of a lessened number of flies, and, second, a lessened number of cases of intestinal disease. This subject is only mentioned here as one worthy of fuller investigation, rather than as a conclusion based upon observation. Mr. Tribus.

This whole topic of refuse collection and disposal is one of very great interest, and is a field as yet almost untouched in the United States, and, prior to the publication of this paper, very little, of much real value and based on facts, has been printed. The speaker hopes that additional information will be gathered, not only in the Borough of Richmond, but in other places, and be put at the disposal of this Society, to aid in this most important work.

H. NORMAN LEASK, ESQ.—The speaker, being conversant with the literature on this subject, and having had long experience in designing and operating destructor plants, ventures the opinion that, for engineers, this is one of the most valuable papers which has ever been presented. It is the more valuable as it enters a new field and presents data from which a contracting engineer can design plants and guarantee results without risk of failure to either of the contracting parties. Mr. Leask.

It is pleasing to note that the author has commenced with first

Mr. Leask. principles, and not at some place in the middle of the subject, which, unfortunately, is often done. The exact figures in the paper, however, do not apply generally, and must be used with the discrimination born of experience, due allowance being made for losses, after the manner set forth in the problematic balance sheet, Table 9.

The information in the paper has not been available heretofore in such extended form; and, as far as the speaker is aware, it has not covered such a long period, or such a great weight of refuse. At the same time, it should be remembered that an inspection of material is desirable, in order to note any peculiarities in its character, without which immediate success is not likely to result.

When the author took up the question, the conditions existing in the Borough of Richmond were practically the same as those which have induced most cities to resort to destruction by fire. In many cities abroad other methods have been tried, such as reduction, making fertilizer, and gasification for lighting and power purposes, all of which have failed signally to deal completely and in a satisfactory manner with the final disposition of refuse of all classes, which is the chief desideratum. The only system of final disposal which is growing in use, and is now quite general, is the destruction or cleansing of refuse by fire, thus rendering it innocuous.

That an examination of existing garbage crematories in the United States should offer no hope of meeting the requirements satisfactorily, is not surprising, for such crematories can hardly be termed engineering propositions, and one doubts very much whether American engineers have had anything to do with their design. The principal faults which one recognizes in garbage crematories of the present type are:

1.—That the process of destructive distillation, rather than oxidization, has been resorted to.

2.—That apparently no lower-limit temperature has been regarded as a standard by the builders of such apparatus, although it is absolutely necessary to maintain a temperature of more than 1 250° fahr., in order to insure the combustion of the hydro-carbons and the dissociation and oxidization of objectionable chemical compounds.

3.—That the usual method of feeding and stoking precludes the possibility of obtaining anything like a regular temperature in the furnace, the temperature rising and falling with an amplitude of probably 800°, and sometimes falling as low as atmospheric temperature.

4.—That the high temperature is usually at the wrong end of the furnace, namely, that farthest from the outlet, and as long as this remains there can be no hope of dealing successfully with the material in a sanitary manner. Attempts to overcome this difficulty have been made by following the ideas of Mr. Charles Jones, of London, who, in 1885, introduced a fume cremator between the furnace and the chimney. This was a palliative rather than a cure, and while it succeeded in reducing the nuisance to some extent, it only went half way. Mr. Leask.

5.—Another error, in certain types of garbage cremators, is made in the environment of the burning mass. Water-jacketed furnaces are absolutely unsuitable for burning garbage or other material high in hydro-carbons. In such a furnace, flame is no sooner generated than it is extinguished by absorption, due to contact with cold surfaces, or by radiation. No one would think of hatching eggs out in an ice box.

6.—Finally, restrictions as to the amount of organic matter remaining in the ash after cremation do not seem to be imposed upon the builders of such apparatus, nor have such apparatus succeeded satisfactorily in eliminating the organic matter from the ash.

Table 1 is likely to give a very erroneous impression as to the quality of the refuse collected in Great Britain. That the author does not rely on the figures given in this table, is quite apparent. His estimates of the character of the refuse in various cities, as given in Table 10, prove that the conditions at the plants he visited did not correspond with the figures in this table. Hutton's figures as to the percentage of coal, coke, breeze, and cinder are much too high, even for mid-winter. Mr. Codrington's figures appear to be much nearer to the actual conditions, while those of Mr. Russell, giving 64.53% for coal, coke, breeze and cinder, are not justified by the results which he has obtained at the Shoreditch plant, the operation of which he directs. The figures for Torquay appear to be inverted, and, if inverted, would more fairly represent the conditions existing in that town and in similar towns along the south coast of England and in suburban districts.

That there is a marked similarity between the refuse collected in Great Britain and that collected in New York is undoubted, and the difference relates more to character than to calorific value. The speaker agrees with the author that there is probably more moisture in refuse, as collected in Great Britain, than in refuse collected in the Borough of Richmond, but it is in a different form. In the Borough of Richmond the moisture is principally contained in the garbage, while in Great Britain the ash, as a rule, contains quite a large percentage of water, and it must be remembered that, in this form, it is more easily attacked than when carried in the structure of material such as garbage.

Mr. Leask. There is a similarity, also, between the refuse collected in many cities on the Continent of Europe and that collected in the United States. It has been stated that the refuse collected in Berlin has a calorific value of about 2 000 B. t. u., while at Frankfort it has a calorific value of 4 350 B. t. u.; the refuse collected in Vienna is stated to contain about 3 000 B. t. u., and that at Kiel somewhat less. The refuse collected in Paris has been analyzed frequently, and has been variously stated to contain from 3 600 to 5 400 calories.

It should be noted that the chemical analysis of the refuse at Kings Norton was made of refuse collected in winter. In the spring, three years later, another analysis was made, and the refuse was found to contain 4 300 B. t. u. In summer, however, the calorific value could not be much more than 3 000 B. t. u.

It might be interesting to give the calorific value applied to various classes of refuse by German scientists, in order that a comparison may be made between that part of Table 1 devoted to that subject and the values ascertained by Mr. Welton:

Vegetable matter ...	2 165	B. t. u.
Bones	540	"
Paper	3 950	"
Sawdust	5 750	"
Wood	6 280	"
Straw	5 400	"
Coal, coke	9 380	"
Hair	1 620	"
Rags	3 600	"

It will be seen that the calorific value of vegetable matter corresponds very closely with Mr. Welton's figures, while that for coal, coke, etc., is somewhat higher, and that for rubbish (composed of paper, wood, rags, etc.) is appreciably lower.

In order to make a comparison of the refuse collected in the Borough of Richmond and that collected in the London residential district, containing some stores, and a suburb of one of the large provincial towns, the speaker's firm, by the courtesy of the city engineers, made a number of analyses of the refuse as collected in the Metropolitan Borough of Stoke Newington and Kings Norton, near Birmingham. This refuse was sorted by hand into four classes:

1.—Garbage;

2.—Coal, coke, cinders and fine dust, including fine inseparable vegetable matter;

3.—Rubbish;

4.—Large incombustible matter, such as tin, bottles, etc.

These analyses were made at a period corresponding to the critical month in Richmond, that is, September and early October. The volume of the refuse at Stoke Newington worked out to about 4 cu. yd. per ton of 2 240 lb., and it contained on an average: 34.43% of garbage, 42.92% of coal, fine dust, etc., 15.4% of rubbish, and 7.35% of glass, metals, etc. The refuse had averaged 4 cu. yd. to the ton for about five months, and presented somewhat similar characteristics during this period. At Kings Norton the refuse had a volume of about 3.75 cu. yd. per ton, and carried 39.5% garbage, 45.4% coal, fine dust, etc., 9.3% rubbish, and 5.2% glass, metals, etc. Taking September alone, there was 49.37% garbage, 38.80% coal, coke, etc., 7.73% rubbish, 4.28% glass, etc., from which it can be seen that the percentage of garbage is even higher than that collected in the Borough of Richmond for that month. It is not suggested that the foregoing figures are absolutely accurate, but merely the result of an honest endeavor to ascertain the make-up of the refuse. The results obtained with the refuse at Kings Norton agree very well with the balance sheet, as shown in Table 9, as to evaporation and combustion-chamber temperature. In September the evaporation was somewhat higher than that calculated in the balance sheet, that is, more than 1.25, actual, in ordinary work, and the average temperature in the combustion chamber about 150° higher. The speaker's make-up of a balance sheet would differ slightly in that the radiation loss would not be as high, while there would be a somewhat lower percentage of unburned carbon in the clinker and ashes, but probably a greater loss in moisture in chimney gases. As the average combustion-chamber temperature and evaporation in winter and spring are considerably higher than the foregoing, it can be seen that, in British refuse, as well as that under discussion, there is a considerable seasonable variation. Mr. Leask.

The practical tests, as given in Table 8, demonstrate clearly that the material is burnable, and the results obtained are such as might be expected when burning material in such a crude furnace, and preclude all doubt of obtaining satisfactory temperatures with a properly regulated and heated air supply.

Based on the results obtained with summer refuse in England, Messrs. Heenan and Froude, Ltd., of Manchester, who have been entrusted with the plant for New York City, specially designed and erected a plant in Vancouver, B. C. This plant has certain departures from their standard type. It is now in successful operation, and, of the refuse collected in that city, it is destroying more than 50 tons per 24 hours, at suitable temperatures, without the aid of supplementary fuel, and with an excellent residual. The refuse burned is very poor in quality, due to the presence of considerable quantities of wood-ash and moisture. The difficulties of obtaining high temperatures with this material have been overcome by checking the quantity and increasing the temperature of the air supply. Figures in detail as to the results are not yet at hand, therefore they cannot

Mr. Leask. be given here. The cost of destruction is 36.1 cents per ton, including the salaries of two engineers who look after the pump, boiler, fan, etc. During the first week the plant was put in operation, it surpassed the guaranty, which is unusual—the men being untrained—and better results may be expected in the course of time.

The utilization of the steam and clinker resulting from the destruction of refuse is by no means the only offset to the cost of burning. The most important offset is the reduction of the cost of collection, for a modern plant may be placed in the center of a city or residential quarter without fear of nuisance to the neighborhood. This means a great reduction in the cost of collection and transport. Numerous instances can be cited supporting this: in the Metropolitan Borough of Stoke Newington, previously mentioned, two 45-ton units, each with 200-h.p. boiler and the appurtenances thereto, a clinker-crushing and screening plant, has been erected in the middle of the Borough, at the rear of the Town Hall, and surrounded on all sides by three and four-story dwellings of a good class. Notwithstanding the fact that the interest and sinking fund on the capital outlay, the repairs, maintenance, and labor charges have to be added to the cost of disposal, the cost of collection, transport, and final disposal is now lower than it was prior to the erection of the plant, and this in spite of the fact that, as yet, no use has been made of the steam generated, which is equivalent to about 175 k.w. per hour from one unit; also exclusive of the sale of clinker, which has been contracted for on a profitable basis. The same conditions apply to Woodgreen, London, where the plant is also critically located, and at Rathmines, Dublin, where a saving in coal of $2 000 per year (in addition to the saving in cost of collection and transport) has been made, and where, in the past, the power has been utilized without relying on storage batteries. Now that storage batteries have been installed, the saving in coal, as shown by the working during the past few months, will be more than $5 000 per anum. This is the more remarkable as the quantity of refuse to be handled is less than 35 tons per day.

Table 9 gives the average temperature which may be expected in the combustion chamber at various seasons; it gives no idea, however, of the lowest temperature which may occur. When burning September refuse on the standard British furnace with air heated to, say, 250° fahr., the lowest temperature would be near, if not actually below, the limit of 1 250°, momentarily. To insure the temperature always being above this lower figure, it is absolutely necessary to increase the temperature of the preheated air, to control the air supply very carefully, and, further, to increase temporarily the temperature of the air entering the furnace immediately after a fresh charge. Fortunately, this can be effected by taking the heat out of the clinker, just withdrawn from the grate, prior to charging. All these points have been given special attention in the case of the plant for New York City. Mr. Leask.

British destructors have been designed in accordance with the principles mentioned by the author, but it has been by progressive steps, and after many failures. The various steps in the conception and improvement of refuse furnaces, as made in England, may be traced as follows:

1.—The attempt to burn refuse in or under shell boilers;
2.—The building of a fire-brick lined furnace, or Dutch oven, operated by natural draft;
3.—The introduction of the fume cremator;
4.—The abandonment of the fume cremator and the introduction of forced draft;
5.—The preheating of the air supply;
6.—The use of a continuous furnace chamber, containing a number of grates with divided ash-pits;
7.—Suitable ventilation of the building;
8.—Methods of handling the clinker and recovering the heat contained in it.

With regard to the cost of operation, it is possible, with a large plant, to reduce the labor charge, part of the work being effected by mechanical means. It must be remembered, however, that the feeding of the furnace is only one of three operations necessary in the working of the plant: First, there is the introduction of material into the furnace, which may be done mechanically; the other two, which, however, do not appear to be susceptible to mechanical operation, are the stoking and spreading and the final cleaning out of the mineral residual from the grate.

The speaker's firm has attempted to solve this problem, and after many failures has at last succeeded in devising a machine which will handle all classes of refuse and will feed the refuse in any desired quantity. Extended trials of such an apparatus have been made at one plant, and it will soon be installed in some city. The system to be adopted for charging depends on the specific gravity of the material to be dealt with and the size of the plant.

The speaker has had the opportunity of examining the refuse in a number of large cities in the United States, and is strongly of the opinion that the combined refuse of most cities can be destroyed by fire at suitable temperatures, without the aid of supplementary fuel.

This brings up another phase of the question which has been mentioned by Mr. Fetherston, namely, the collection of the refuse. The adoption of a system of a single collection of refuse, combining the ashes, rubbish, and garbage, cannot be urged too strongly. It is impossible to get complete or satisfactory separation. It is a question

Mr. Leask. of public health, rather than profit. The single collection costs less to make. The mixing of the refuse retards decomposition, the ashes acting as a deodorant by absorption, and it provides a wherewithal in calories to cleanse the mass of its impurities, and, when burned, leaves a marketable residual in the form of clinker. If advantage be taken of the heat generated by combustion (and here it should be noted that, whatever the material may be, when it is burned at suitable temperatures there is always utilizable heat), there is placed at the disposal of the authorities another valuable residual in the form of steam, the best uses for which are those giving a large load factor, such as pumping—sewage or water—or electric traction. Lighting alone is not a satisfactory outlet. The question of collection and disposal of refuse in the United States to-day appears to be in the state that it was in older countries some years ago, that is, in the hands of contractors. It has been abundantly demonstrated that the only satisfactory method is for the municipalities themselves to undertake it. Where this has been done, and where refuse destructors form a part of the scheme, it has been followed by a noticeable decrease in the death rate. Why should not one of the richest countries in the world forsake the problematical gain arising from reduction, and regard the question from a purely public health standpoint, as has been done in older countries, even in backward Russia?

The whole question is purely one of combustion, and, generally speaking—provided the moisture contained in the material is not excessive—refuse containing 2 000 B. t. u. per lb. will cleanse itself.

The great difference between the combustion problem as applied to coal and refuse is this: When dealing with coal one has a material which is comparatively low in ash and requires about 20 to 24 lb. of air per lb. to burn it in a practical and satisfactory manner, whereas refuse is high in ash, and the air required is only from 4 to 5 lb. per lb. The difficulty, therefore, is to find a small quantity of carbon in so large a bulk, with the minimum quantity of air. To effect this one must look to the distribution and the temperature of the air supplied, the intensity of the draft, and the environment of the burning mass. It is, therefore, wholly an engineering proposition.

The speaker must again congratulate the author on the service he has rendered to engineering science in general and contracting engineers in particular.

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PAPERS AND DISCUSSIONS.

THE REINFORCED CONCRETE WORK OF THE McGRAW BUILDING.

Discussion.*

By Messrs. Guy B. Waite and E. P. Goodrich.

Guy B. Waite, M. Am. Soc. C. E.—Had the McGraw Building been designed ten years ago, it would not have been built of reinforced concrete. Few engineers at that time had sufficient confidence to undertake the experiment, and the local building laws did not permit it. Even more recently than this, nearly every important architect in the vicinity would have refused to listen to an argument for using concrete in a building in which there was to be heavy vibration, as it was only considered fit for light, cheap buildings. Mr. Waite.

At that time, even engineers were afraid to speak in favor of concrete for general building purposes, for fear of becoming unpopular. One who dared engage in concrete building construction had to endure the humiliation of seeing some of his former friends pass quietly to the opposite side of the street when they saw him, in order to avoid meeting one who was engaged in an immoral business, and who was just escaping the meshes of the law. The change in public opinion, with regard to reinforced concrete, has been brought about purely by the merits of the construction.

About ten years ago the Building Department of New York City inaugurated standard tests for concrete constructions to be used for fire-proof floors. All constructions for floors had to be submitted to a 4-hour fire test, at an average temperature of 1 700° fahr. They were to be loaded with 150 lb. per sq. ft. on a full-sized floor not less

* Continued from December, 1907, *Proceedings*.

Mr. Waite. than 14 ft. long. Immediately following the fire—the material being still red-hot—a regulation stream of cold water was to be thrown on the construction, with a pressure of 60 lb. per sq. in., for 10 min. The construction was then to withstand a distributed load of 600 lb. per sq. ft.

During the five years following the inauguration of this test, about twenty-five concrete constructions withstood it successfully. By this time some of the public had been convinced that concrete possessed merits, as a fire-proof material, but did not dare to speak out; while others feared that it had some merits, and set out to kill it. Quite successful obstacles were placed in its way, by the Board of Insurance Underwriters, who fined it; by codes of law, which practically ruled it out; by labor unions, who dictated by whom and how it should be made; and by politicians, whose interests were generally in other directions.

Interest was finally awakened by the favorable showing made by concrete in some of the great fires on which a few honest reports were made by eminent engineers, and since that time concrete constructions have gained rapidly in popularity.

It was only about four years ago that the speaker secured, from the Department of Buildings of New York City, the first permit ever granted in the Borough of Manhattan for a concrete building, including concrete wall construction. All who are interested in the advancement of reinforced concrete must feel indebted to Professor Burr for giving his name and influence to this cause.

The McGraw Building was undoubtedly made in reinforced concrete because it offered the best construction to withstand the peculiarly heavy work of a printing house; and because it gave the safest fire risk. The National Board of Fire Underwriters, some two years ago, recommended a minimum rate of insurance on similar constructions, and, from recent inquiry of the Local Board of Fire Underwriters, it is learned that even this august body has at last reached a point where a similar action is under consideration.

As a fire risk, concrete structures offer the best possible investment, either for individuals to carry their own risks, or for insurance associations to make a specialty of these constructions. Such undertakings would be the safest and most profitable kind of insurance ventures.

Concrete is superior to burned clay, not because it is more fireproof, but on account of its superior elasticity under the stress due to fire and water. Some years ago the speaker constructed, entirely of cinder concrete, a test house, 14 by 14 ft. and about 12 ft. high, with walls 12 in. thick, and with a ceiling only 1½ in. thick. In various tests conducted by the Department of Buildings in that house the ceiling was submitted to four separate series of fire tests.

Fig. 1.—Cinder Concrete Test House, After Fourth Fire

Fig 2.—Stone Concrete House, with Test Load, After Four-Hour Fire Test.

When this structure was torn down, to make way for dock improvements, this 1½-in. ceiling was examined by an engineer from the Department of Buildings of New York City, and by Professor Woolson of Columbia University, and was found to be in good condition, the fire having affected scarcely ½ in. on its under side, and this was due to the first fire. Mr. Waite.

While reinforced concrete has been demonstrated to be superior in many respects to other forms of fire-proof construction, it has forced its way to the front principally on account of the economy it has effected.

The parts of a building in which it is best adapted must be determined largely by the engineer's experience. Sometimes this experience is paid for very dearly, and forms a secret chapter in his biography.

Local conditions often alter circumstances to such an extent that a kind of construction which might be erected in one locality at a profit would become a loss in another locality a short distance away.

As the relative prices of built steelwork and concrete per unit section are about as 65 to 1, and the relative working capacities in compression (16 000 to 500) are about as 30 to 1, it is evident that, other things being the same, the more concrete is substituted for steel in compression, the greater is the economy.

Where heavy loads are to be carried, concrete will be found highly advantageous; conversely, where small loads are carried, it will have little advantage. Keeping this fact in view, one would expect to find the greatest economy in using concrete for column supports and floor constructions—the heavier the construction, the greater the economy.

The great barrier to using concrete for columns is the impractical size necessary when more than a few stories are required. Leaving to others the discussion of the rationality of the combination of concrete and steel in the columns of the McGraw Building, the speaker considers this form of construction superior to anything heretofore done in reinforced columns.

The column is reduced to a reasonable size, and is made safe against accidents. The positive dead loads from the building are carried by positive steel supports, while the doubtful superimposed floor loads are adequately provided for by the more questionable form of concrete reinforcement.

The certainties are balanced one against the other, and the uncertainties are also brought to face each other. Although columns of this form are not directly the cheapest, the speaker believes them to be the most economical, all things considered. Columns of this form are adapted to much speedier erection than the cheaper reinforced concrete, and are absolutely safe during erection; they also allow as good a monolithic connection of the column with the floor as that

Mr. Waite. obtained in other reinforced forms. The column forms, with the steel as a guide, cost less than where they are made as independent structures.

As pointed out by the author, the construction of the forms is probably the greatest problem in the practical construction of reinforced concrete. Almost every beginner in this field has arrogant confidence in his ability to eclipse everything previously done in the way of perfect centering. It is only necessary to watch such an one and see a second scheme in his second job, a third scheme in his third job, and so on, until he becomes a meek plodder along the tow-path of experience.

A discrimination should be made between centers or forms designed for a building, and a building designed for the forms. Forms made to fit a special building may cost several times as much as those with which a standard building might be made. The cost of centerings may be reduced about in proportion to the standardization of the building. Many useful schemes for systematizing the general construction of centerings have been invented, and many of these have simplified the problem so that the main cost is in taking down and putting up the forms.

Most of these centering schemes are used for rough concrete work, where the surface is to be plastered afterward; but when finished surfaces are to be produced, the cost of centers is more than doubled. If the mechanic trained to do finished-center work be told to make rough standard centers, he will spend nearly 7½ hours carefully getting ready to do work which would take the other man ½ hour; conversely, the rapid standard-center man if put on perfect-center work, would do in ½ hour what would take the perfect-center man about 7½ hours to undo and do over again. False conceptions and misrepresentations on the part of competitors in concrete work have led them into bitter warfare by presenting owners with what they term "finished surface" free of cost. Is it not possible to conceive a method of trying uniformly rough surfaces instead of uniformly smooth surfaces, as a help toward solving the problem of centers?

The indirect method may be used to cheapen centers, that is, constructions may be used which do not require the expensive centering necessary in ordinary forms of reinforced concrete. If such constructions do not advance the total cost by increases in other directions, there will be a net saving. As is evident, the forms or centers for ordinary reinforced concrete must be sufficiently heavy to maintain a perfectly independent structure under the tendency to warp and deflect, due to the fact that they are alternately wet and dry, and also on account of the heavy load of the concrete. Where a steel skeleton (such as the columns in the McGraw Building) maintains the construction lines, one may use for centers material which is much lighter, and

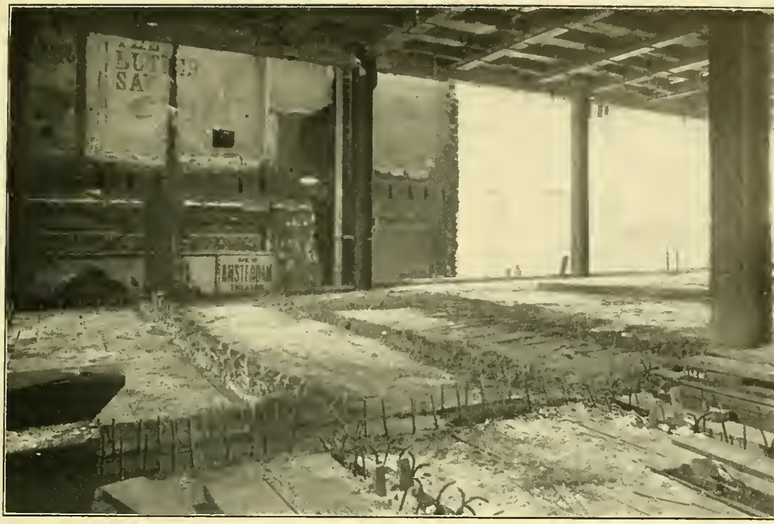


FIG. 1.—THE BONWIT-TELLER BUILDING.

FIG. 2.—THE SALVATION ARMY WAREHOUSE.

more easily worked and handled than with ordinary reinforced concrete. Mr. Waite.

The author does not describe the wall construction of this building, but the speaker believes it to be of some form of reinforced concrete. In wall construction, the conditions are very different from either column or floor construction. In both columns and floors, concrete makes a saving in steel, but, in wall construction, this element of saving does not enter. Further, in monolithic wall construction two forms must be kept plumb, but in floors only one form is required, and gravity helps to hold this in place. The speaker, with the very best of assistants, after finishing several buildings having complete concrete wall construction averaging 8 in. thick, concluded that the cost of the concrete in such wall construction was of small consequence and could be safely neglected in totaling up the entire cost of the wall. In factory construction, where there are practically no walls except panels under windows, no such difficulties are encountered as in dead wall construction.

Nothing is stated definitely by the author concerning the character of the steel used in the floor construction, other than that round rods were used. The kind of bars and the character of the steel in them seem to command a great deal of attention at present. The only logical conclusions that can be drawn from the claims of the big grist of deformers (with increased capacities for each new deformation in their rods) is that they are developing the art toward a state where (according to claims) practically nothing but bond and grip will be required, and steel for tension, etc., as now designed, will become of little consequence.

In referring to some recent constructions which the speaker has executed, the only excuse he has to offer for so doing is that such improvements have come after quite a lengthy experience in general steel and reinforced concrete construction, and, being a product of natural evolution, they belong to the general scheme of development toward something higher. The speaker's experiences have been unlike those of many engaged in reinforced concrete construction, because, in most cases, he has had to contend with the conditions existing in crowded parts of large cities, where space for storing materials and performing work is extremely limited, and where great rapidity of erection is necessary; and, on account of the extra height of buildings, safe construction must be considered.

Having been fundamentally trained in steel construction, followed by the fire-proofing of the steel; and subsequently having pursued general reinforced concrete construction, the speaker was forced to consider the merits and demerits of the combinations of these three factors in building construction for the conditions found in large cities.

Mr. Waite. The safety and the speed of steel construction were apparent, and the advantages in the use of concrete for the protection and fire-proofing of steel were well demonstrated. Then followed the combination of steel with concrete (formerly used for fire-proofing), and this developed into a system which possessed all the merits of the steel skeleton construction and the advantages of reinforced concrete. In this system (known as System "M," in its order with other systems) there is required only from 35 to 40% of the steel necessary for the conditions in which the steel does all the work. The concrete—which must be used for fire-proofing—is made to do the remainder. The light steel frame is run up ahead of the concrete, in the usual manner for steel frames, and is made strong enough to take all tensional and shearing stresses in the subsequently reinforced construction formed by the steel and concrete. The combination forms a truly reinforced structure.

Work can be done on several stories simultaneously, as in other steel construction. The necessary forms are simplified, as compared with those required in most reinforced concrete constructions, on account of the assistance given by the steel frame. Within the last two years, some twenty buildings in the vicinity of New York City have been constructed by this system.

From January 1st to April, 1907, while the McGraw Building was being erected, three buildings in that vicinity were constructed in which the floors were of this form of construction, namely: The Bonwit Teller Building,* at 15 and 17 West Thirty-fourth Street, shown by Fig. 1, Plate III; the Salvation Army Warehouse,† at 533 and 537 West Forty-eighth Street, shown by Fig. 2, Plate III; and the Strack Building, at 214 and 220 West Twenty-third Street.

In these buildings steel columns carry the entire loads, but, where conditions permit, a light steel frame, similar to the construction used in the McGraw Building, carries the dead floor loads.

The general form of the steel in this combined column construction is shown by Fig. 1. It is made of channels or similar sections disposed centrally with respect to floor beams and girders, and the separate steel members are connected at the corners. The speaker has found this to give a very simple and effective steel skeleton which, he believes, affords ample means for the proper combination of the steel and concrete.

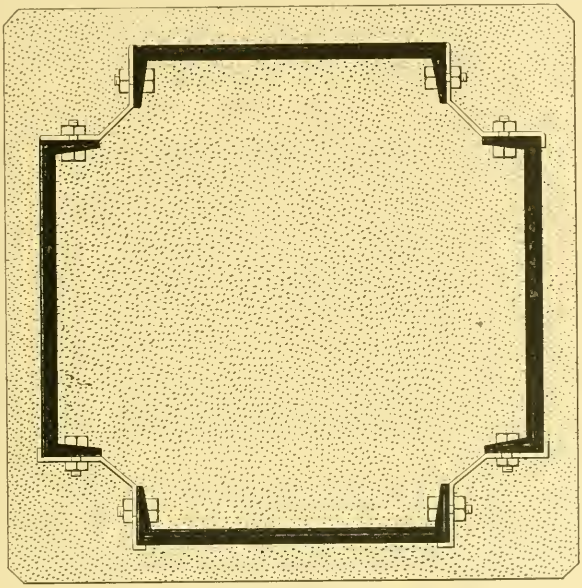
Some months ago, in building a garage for the use of his family, at Whitestone, Long Island, the speaker concluded to make the entire building of reinforced concrete in order to demonstrate the economy of a new form of construction suitable for a small number of laborers. The building is 40 ft. long and 20 ft. wide, and has two stories, and

* *Engineering News*, April 25th, 1907.

† *The Engineering Record*, June 22d, 1907.

Reinforced Concrete Garage at Whitestone, Long Island

an attic. The walls consist of a series of 12 by 8-in. reinforced pilasters, spaced 5 ft. apart between centers. Between these buttresses, and erected simultaneously with them, there are concrete blocks, 3 in. thick and 12 in. wide. In the lower story the blocks are flush with the outside of the pilasters, and in the second story they are kept back from the front to give the effect shown in the photograph, Plate IV. Mr. Waite.



SYSTEM "M" COLUMN

Fig. 1.

The floors were reinforced with 4 by 7½-in. steel beams, resting on the pilasters, and having shear bars extending up from holes in the webs of the beams. One handy man and two laborers constructed the foundations and all the walls and floors in about 8 weeks. The walls were run up several feet above the second floor in order to make a full story of the attic.

E. P. Goodrich, M. Am. Soc. C. E.—The speaker's connection with the McGraw Building, in a supervisory capacity, during the major portion of its design and construction makes Professor Burr's description of special interest to him. In a few particulars, that description may be somewhat amplified, for the sake of noting additional points of interest. Mr. Goodrich.

The power plant for the building is located in a sub-basement situated in the southwest corner. Consequently, the columns in that portion of the building are somewhat longer than the others and exceed the dimensions given in the paper by 12½ ft., making the length

Mr. Goodrich. of the longest column 172 ft. It may also be of interest to note that the "reinforcement" in the first length of one of these columns weighed 14 050 lb.

The windows on the sides and rear of the building are of wire-glass in metal frames, so that practically the only possible additional device which could be added to give security against fire, would be a complete automatic sprinkler system. In consequence, the McGraw Building is one of the best in the city, as far as insurance conditions are involved, and carries a very low rate for both the building and the contents.

The column spacing was determined primarily by the dimensions of modern printing presses, nearly half a score of which are now in operation on several of the upper floors of the building. This fact brought about the use of rectangular floor bays, while a more nearly square arrangement would have been slightly more economical, had it been possible to design the building in that way.

Of the several new features in the building, of course, the column design is the most unusual. While the whole arrangement, as finally worked out, proved highly satisfactory, from a construction point of view, it may be open to some adverse criticism, from a solely economic standpoint. As shown by Mr. Douglas, a design for purely structural columns would have cost less money, and Mr. Stern suggests that, even when fire-proofed, such columns would have been smaller than those used. Plenty of evidence has been adduced from the San Francisco conflagration to show that no comparison can be made between structural columns, however well "fire-proofed" in the usual manner, and such columns as are used in the McGraw Building. No comparison is fair unless this superiority to resist fire is capitalized. On the other hand, columns of the Considère type likewise possess this good quality, their principal drawback, under such conditions, being their size. It may be of interest to state that, early in the history of the design of this building, the speaker caused to be prepared a typical column of the Considère type, based on the accepted stress requirements of the New York City, Manhattan Borough, Building Regulations at that time. The columns, of course, were circular in section, with a diameter in no case greater than the diagonal of the corresponding square column of the Burr type, finally used. The estimated cost of the Considère column, on a conservative basis, showed an apparent saving in its favor of approximately $10 000 for the whole building.

Two other small objections to the Burr column were also discovered, which were almost entirely obviated during the progress of the work, and could be entirely remedied in future designs. The wide faces of the angles in the lower stories, and the wider expanses of some of the splice-plates, made necessary a special wrapping of wire or wire lath to hold the fire-proofing concrete in place; and the extreme rigidity Mr. Goodrich. of the column steel, made necessary a much more careful adjustment of the forms than is usually required for reinforced concrete buildings. In most cases, the less rigid reinforcing rods are given slight eccentricities, which do not affect their efficiency seriously, and are thus made to accommodate themselves to small variations in the spacing of the forms, and thereby save some labor cost. This latter possible defect of the rigid reinforcement may even be considered a real virtue, in the eyes of some people.

The speaker is aware of really very few reliable tests of reinforced concrete columns, and of none which possessed anything like the percentage of longitudinal steel found in those of the McGraw Building. Some time ago, the speaker arranged for a series of tests on specimens designed after the Burr type, and of practically full size, but, unfortunately, the results have not yet been secured. In this, it is well to note a fact to which Professor Morsch, of Zurich, calls attention, in his "Eisenbetonbau," that the efficiency of longitudinal rod reinforcement decreases with the increase of the percentage used, at least up to 4%, and that no one knows how larger amounts will act. It is thus incumbent upon designers to exercise great care in selecting working stresses for concrete columns possessing considerable longitudinal steel, as the field is absolutely unknown at the present time, and some serious trouble may result for inexperienced designers who follow rules blindly.

Another point to be noted is the fact that most experimenters on concrete columns have concluded that the concrete appears to carry much the larger percentage of the load until it has reached a stress far above the usual allowable working one, when the steel comes into more pronounced action. Of course, this conclusion is based on computations involving an assumed modulus of elasticity of the steel and the observed stresses and strains of the column. The distribution of stress, above described, is probably due to the fact that the stress-strain curve for concrete is not a straight line, thus demonstrating the existence of a variable modulus of elasticity. From these facts, it might seem to be more rational to reverse the condition as to the dead and live load carrying capacities of the steel and concrete in the Burr column, and require the concrete, at say 750 lb. unit stress, to carry all the dead load and then add enough steel in structural form, if so desired, to carry the total or reduced live loads.

Another item of design in the McGraw Building to which special attention was paid, was the connection between the reinforcing rods and the column steel. This was worked out so effectively that the steel erectors of the columns often attached, to any convenient point of the beam reinforcement, one end of the turnbuckle which they used for plumbing the column sections. In no other reinforced concrete building within the speaker's knowledge could this be done.

Mr. Goodrich. With such rigidity of column steel and its firm connection with the beam rods, the best method of beam design would seem to be that of cantilevers or continuous beams throughout, instead of simply supported members. Such an arrangement of the steel in a concrete beam has the following advantages:

Maximum shears occur at points where maximum moments are found, and, in consequence, where most steel is placed.

Not as much steel is found near the bottoms of beams, where it would be exposed to the most trying effects of fire.

Such a method of design obviates the tendency to sharp deflections near the supports, with the resulting probability of the occurrence of cracks at points where the shear is the greatest.

Such design gives most resistance against the type of failure observed in impact experiments.

There is also less likelihood of the displacement of reinforcement, because it is in view during the greater part of the process of concreting.

All beams and girders throughout the McGraw Building were designed as fully continuous, or restrained, even where supported in the outside columns and walls. The drawings show twice as much steel over the supports as in the centers of the spans, and, since the factor used in connection with the moment at the latter point is $\frac{1}{10}$, according to the requirements of the Building Code, the factor for the supports is only $\frac{1}{5}$. Thus it is seen that, when compared with $\frac{1}{24}$, and $\frac{1}{12}$, the theoretically correct values, more than twice as much beam and girder reinforcement was used as theory would dictate. This extra material was used in a literal compliance with the anomalous wording of the New York Building Code. A comparison of this building with numerous others has led the speaker to the conclusion that the requirements therein contained are rarely complied with literally, and that this faulty requirement of the code has been the real cause of much poor work.

To the speaker, no reason is apparent for using, over points of support, more steel than enough to satisfy the theoretical moment formula, with a coefficient of $\frac{1}{12}$. When that amount is used in that way, only half as much, of course, is theoretically necessary in the lower part of a beam at its center, while the building requirements specify as much as would be indicated by a coefficient of $\frac{1}{10}$, which is even more than is needed in the upper part of the beam over a support.

Some slight argument may be advanced for using as much steel below as above, in the two locations, from the fact that eccentrically placed partial loads on continuous members resting on perfectly movable supports, subject the members to maximum positive and negative moments which are much larger than those produced by a continuous load, as usually considered. This fact has often led the speaker to Mr. Goodrich. recommend a partial concession to the older ideas of design, and to use equal amounts of steel over the supports and at the centers of spans, determining this quantity by the coefficient, $\frac{1}{12}$. This distribution allows of an economical design for, and method of handling, the rods; it meets practically all the requirements of partial loads, and, at the center of the spans, uses within 20% of the quantity of steel required by the New York Building Code, with 100% better distribution, as far as prevention of cracks is concerned.

The reinforced concrete beams and girders of a monolithic concrete building are not beams and girders at all, in the sense of the wooden and steel ones in the older types of structures, which simply rest on brackets and have ample opportunity for motion in each joint. Until cracks have formed, the concrete beams are really extended brackets on the columns and other members, and should be designed as such. The early workers with reinforced concrete were influenced too largely by the old type of structure, and few designers have even yet grown into the true spirit of the newer material.

Thus it has transpired that the McGraw Building has a floor construction which is rated far below its true safe carrying capacity. Were any floor loaded to failure, the latter would probably take place by shear, or rather diagonal tension. In relation to this, however, it must be stated that the speaker never has understood why those in actual charge of the design of the reinforcement (other than the author of the paper), invariably used an odd number of rods to resist tension. By so doing it is impossible to bend upward the same number of rods at each end of a beam to assist in resisting shear, so-called. Thus, five rods might be used in a given case for tension reinforcement. Three could be bent upward at one end, but then it is practicable to bend up only two at the opposite end of the adjoining beam without causing a congestion of steel over the support. If the two bent rods were just sufficient to assist the shear, the three rods at the other end would give 50% better efficiency at that end; and 20% more resistance would have been secured at the weaker end by using six rods of smaller individual (but aggregating the same total) area, and bending up three rods at each end.

The floor forms were designed with especial care. They were collapsible in type, and were erected in an exceptionally substantial manner, so as to be capable of serving as platforms from which to erect the structural work of the columns. The determination, afterward made, to use the central tower for erection purposes, rendered this special reason for heavy forms unnecessary, but their value was repeatedly shown for other reasons, and the speaker is decidedly of the opinion that a little extra material in excess of that sometimes seen, is of real economic advantage. The forms were designed by Mr. J. G.

Mr. Goodrich. Ellendt, and were all built in a special shop, hauled to the building site on trucks, and the whole truck-load hoisted by the central derrick in a single operation and set practically in place. The speed actually attained in erecting the building, shows how well the form work was prepared and carried on, because that work is the crucial part of the erection of all concrete work. Matched and dressed material was used throughout, always well coated with oil, so as to obviate the necessity of special surface finish if possible. However, the rapid and repeated use of this material during the winter soon disclosed the fact, the truth of which has always been held by the speaker, that it would be necessary to plaster the building, if it was to be given a character on a par with the average office structure.

No plans of the forms are included in the paper, although reference to them appears at one point.

In the speaker's opinion, the tower used in the erection of this building was really a factor of large economy. For instance, all concrete was hoisted to each floor in buckets dropped through the elevator shafts to the mixers, which were in the basement and placed so as to dump directly into the buckets as they rested. The booms swung the buckets so that they could be dumped exactly at the desired points, thus obviating the use of other hoists, hoppers, wheel-barrows, runways, etc. This method proved so effective that very often the cost of all labor on concrete for considerable quantities would not exceed 40 cents per cu. yd.

The speaker certainly would repeat the use of that special contrivance on a similar operation, except that he would stiffen the structure to a somewhat greater extent, and would use 12 by 12-in. timbers for corner posts instead of the 10 by 10-in. posts used in this instance. The tower structure also served as a storage space, and was of almost inestimable value in this respect, because of the congested portion of the city in which the building stands.

During cold weather, besides making use of the salamanders, as described by the author, the concrete was mixed with hot water and all aggregates were heated so as to prevent frozen lumps from getting into the work. On one operation with which the speaker was connected he once removed a lump of frozen sand from a column in which it would have occupied about 15% of the total area. The necessity of heating the aggregate is obvious, since, even when boiling water is thrown into the mixer, it has such speed of operation that not enough time elapses to thaw frozen masses and get them properly distributed, before the mixing process is complete.

With the methods used on the McGraw Building, even in the coldest weather, the concrete would reach the point of deposit at a temperature ranging from 50 to 75° fahr., and would have attained its initial set while its temperature was still warm to the touch. The salamanders maintained a temperature in the dead air spaces between the beams, which often reached 100° fahr., and seldom fell below 60°, even in zero weather. Mr. Goodrich.

Of course, the special interest of this paper centers around the type of column. The speaker feels that, even at the present time, the designing of reinforced concrete columns is like working in a darkened room, and this is said even after personally making a large number of column tests, and after carefully analyzing nearly a hundred others. If carefully designed, and when a proper relation exists between the longitudinal and spiral steel, the speaker considers a Considère column entirely practicable for a reasonably high building with comparatively light floor loads; but, for more lofty structures, the opinion is gaining strength, in the speaker's mind, that a regular structural steel column should be used, in connection with reinforced concrete girders, beams, and floors, if desired. This structural column, however, should be of some open design, and it should be completely filled with concrete and surrounded by a fire-proofing at least 3 in. thick over all extreme edges. Such a composite building will be more economical than any other, in yearly carrying charges, including interest on first cost, insurance, maintenance, heat, etc., and a correspondingly larger income can be derived therefrom.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

NATHANIEL HENRY HUTTON,* M. Am. Soc. C. E.

DIED MAY 8TH, 1907.

Nathaniel Henry Hutton was born on November 16th, 1833, in Washington, D. C., and died in Baltimore, Maryland, on May 8th, 1907. His earliest ancestor in the United States of whom there is any record, John Strangeways Hutton, was born in New York City in 1684 and died in Philadelphia in 1792. His father was James Hutton, who married Salome Rich of Boston, Mass., in Washington, D. C.

Following the example of his elder brother, the late William Rich Hutton, M. Am. Soc. C. E., "Harry" (as he was familiarly called by those who knew him well) entered the service of the United States at an early age, adopting the profession of civil engineering. Neither had the advantage of a collegiate education, but they did have the good fortune to grow up under the thorough training of those days, in the specially excellent schools of Alexandria and Washington, taught by men like Ben Hallowell, Abbot and others. They made good use of those early opportunities, and by industry, faithful attention to duty, and continual study of the theory of engineering and the works of able engineers, their own experience and unusual natural talents enabled them to pass through the lower grades of the profession with credit to themselves, and with the respect and ever-increasing confidence of their superiors in their integrity and high tone, until they had come to rank well among the engineers of their period in the special lines to which their attention was called.

Mr. Hutton's work as a surveyor and engineer, up to 1896, may be summarized briefly as follows:

He was U. S. Assistant Engineer on explorations and surveys for the Pacific Railroad west of the Missouri River, on the 32d and 35th Parallels, from 1853 to 1856, inclusive; Chief Engineer of the El Paso and Fort Yuma wagon road (Department of Interior) during 1857 and 1858; Surveyor on the western boundary of Minnesota (Department of Interior) during 1859 and 1860; U. S. Assistant Engineer on the defenses of Baltimore from 1861 to 1865; U. S. Assistant Engineer in charge of the improvement of the Patapsco River from 1867 to 1876, and on the Western division of the Virginia Central Water Line (survey 1874 to 1875); and from 1876 until his death he was Engineer to the Harbor Board of Baltimore; he was also U. S. Assistant Engineer in charge of surveys for a ship canal to connect the Chesapeake and Dela-

* Memoir prepared by William P. Craighill, Past-President, Am. Soc. C. E.

ware Bays during 1878 and 1879; Consulting Engineer for a project for a ship canal between Philadelphia and the Atlantic Ocean in 1894 and 1895; and Consulting Engineer for a projected ship canal to connect Lake Erie and the Ohio River in 1895 and 1896.

For many years previous to 1896, and up to the time of his death, Mr. Hutton had been Chief Engineer to the Harbor Board of the City of Baltimore. That he held this office so many years, during the administrations of mayors and councils of opposing political parties, is proof that his services were considered so valuable as to be almost indispensable. Later, he became President of the Harbor Board, as well as Chief Engineer.

The following tribute from the Harbor Board shows the high esteem in which he was held by his associates, and it may be said with truth that this was the sentiment of the business men of Baltimore who were best acquainted with his work and ability:

"The death of Major Nathaniel H. Hutton, Engineer of the Harbor Board of Baltimore City, comes at a time and under conditions which cause especially deep feelings of sorrow and regret in the minds of the members of the Harbor Board.

"Immediately after the fire of February 7th and 8th, 1904, he was called upon by the citizens of Baltimore to suggest and design plans for the new docks and the improvements of the harbor of this City. The preparations of these plans, together with his other duties as engineer of the Harbor Board, devolved upon him a very great amount of skillful professional work, and it is probable that he unconsciously overtaxed his strength in this way.

"The influence which Major Hutton has exerted upon the plans for the improvement of the Harbor, cannot be estimated. He has not lived to see the realization of what he has planned, but there can be no doubt that his activity and experience in this great work will be appreciated by his successors, and the citizens of Baltimore, when the full effects of his labors and efforts are realized.

"Major Hutton was an engineer of rare ability and of vast and varied experience. He was a gentleman of the old school, and a most faithful engineer and honest public servant.

"*Resolved,* that in the death of Major Nathaniel H. Hutton, the City of Baltimore has been deprived of a noble and trusted citizen and a capable and conscientious public servant, who has devoted many years of his life to her interests.

"*Resolved,* that the members of the Harbor Board, who particularly appreciate the full measure of loss suffered by his death, tender their sympathies to the family of the deceased, and that these Resolutions be spread upon the Minutes of the Board."

There are also appended resolutions adopted May 19th, 1907, by the Board of Public Improvements, of which Mr. Hutton was a prominent member:

"At a special meeting of the Board of Public Improvements held this date called to take action on the death of Major N. H. Hutton, President and Chief Engineer of the Harbor Board, the following resolutions were adopted:

"*Resolved,* that by the death of Major Hutton the City of Baltimore has lost a most faithful and efficient public officer, whose long service as Harbor Engineer here and extended experience on important public works elsewhere made his services invaluable to this city.

"Also by his death, we, his fellow members of the Board of Public Improvements, have lost a trusted friend and wise counsellor, whose uniformly genial and courteous nature greatly endeared him to us.

"We extend to his family our sincere and heartfelt sympathy in their great sorrow."

Mr. Hutton was a Charter Member and Vice-President of the Engineers' Club of Baltimore. At his death the Club took the following action in his honor:

"*Whereas,* We, the members of the Engineers' Club of Baltimore, have learned with sincere sorrow of the death of our fellow member, Major N. H. Hutton; and whereas we recognize his earnest efforts, as a Charter Member and Vice-President, to promote the welfare of the Club, and the active, friendly and generous interest, manifested by him, in establishing its success:

"*Resolved.* that in his death the Engineers Club of Baltimore has been deprived of a distinguished member and a Loyal and Honoured Friend."

Mr. Hutton was also an architect of decided ability, as is shown by the outcome of the designs proposed by the firm of Hutton and Murdock, of which he was a member for several years, for the construction and alteration of a number of churches, dwelling-houses and warehouses in Baltimore, Washington, Virginia and Pennsylvania. One of his designs for a highway bridge in Baltimore was considered by a very judicious board to be the best among five that were submitted. Not only was Mr. Hutton esteemed as an able engineer and architect and a capable and faithful official, but he was admired and loved by his friends in an unusual degree. A few extracts are appended from many testimonials that have been received as proof of the statements already made.

After a long intercourse, under conditions which often test men's character, long-drawn-out surveys among the rough surroundings of camp life, in the midst of Indians and uncultivated and often lawless frontier people, both male and female, one of his closest friends writes:

"'Tis said that you must sleep with a man to learn his peculiarities. Well, if this is true, Harry and I ought to have become pretty well acquainted, for the nights we stretched ourselves on the ground under the same blanket, ate our grub out of the same tin pan, and drank our coffee out of the same tin cup, ran through years, and during the entire time our affection became closer. It was only necessary to know him to love him, and, of the many acquaintances I have made during a long and varied life, I have yet to meet the man who excelled him in the noble qualities of head and heart which he possessed. He was one of Nature's noblemen, a conscientious Christian whose only fear, if he knew what fear was, was to do wrong, and whose sense of honor was as firmly fixed as the everlasting hills."

Another, with whom Mr. Hutton had close professional and personal contact in Baltimore, gives the following high testimonial from himself and others of their mutual associate:

"All of us had the highest appreciation of his ability as an engineer and of the value of his services to the city. He had been our Harbor Engineer for so many years that he had become indispensable in the working of our city government. His advice was frequently sought by municipal engineers and other municipal officials, and his opinion was always respected on all engineering questions. He was progressive, broad and liberal in his views, yet conservative enough to hold down some of us younger and rasher engineers. He was a conciliating and harmonizing influence at all gatherings of engineers and meetings of boards and commissions. His personality was such, and his manner was so genial and kindly, that he could regulate or harmonize where others could not, and yet always retain the regard and affection of his associates.

"Because of his years of experience and of his broad learning, his place in our municipal government will be hard to fill. His place in our affections can never be filled."

Another who had served with Mr. Hutton very closely for many years adds:

"As an engineer, he was capable, careful, eminent and prominent, and was consulted in the development of many projects of National importance. On undertaking any new work he sought the results and opinions of others of distinction and after giving careful consideration formulated his plans.

"As a public official, he was earnest, honest and faithful, possessing a keen power of penetration, and his approval always carried weight.

"As a man, he was modest and retiring, affable and lovable, with ever a kind word for his fellow-man, be he high or low, and all in all a splendid type of a gentleman."

Still another says:

"I was thrown in intimate relations with him. He was always to me the embodiment of a true gentleman, in the highest and best sense of that word; honorable and truthful, above suspicion, always courteous and always manly.

"As an engineer, he was well trained and on broad lines. I had great confidence in him, and frequently consulted him about difficult problems coming up in my work, and always got sound and helpful advice. If I were called upon to name some special characteristic of Major Hutton, which distinguished him as an engineer, I should say that good judgment was his strong point.

"His death leaves a great blank, both professionally and socially. My feelings for Major Hutton were those of real, genuine affection, and I believe that most men who came in close contact with him had the same. It is difficult to imagine a true man having any sentiments for Major Hutton other than those of the profoundest confidence and respect."

The writer knew Mr. Hutton for more than forty years, both professionally and socially, and can fully bear testimony to the fact that what is said by others in what precedes is not exaggerated. His domestic life was charming and lovely.

In early manhood, Mr. Hutton married Miss Meta Van Ness, daughter of Colonel Eugene Van Ness of the United States Army, who was a member of the well-known and distinguished family of that name in the State of New York. One of Mrs. Hutton's ancestors was Admiral Van Ness of Holland, who lived in 1653; and in Scotland her lineage dates distinctly and honorably at least to 1542.

Mr. Hutton passed from time to eternity in May, 1907, and his devoted wife followed in September. They left three children, all resident in Baltimore, Mr. Harry Hutton, Mrs. S. S. Busby and Mrs. C. H. Wyatt.

Mr. Hutton was elected a Member of the American Society of Civil Engineers on June 3d, 1896.

William P. Morse

AMERICAN SOCIETY

OF

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INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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Vol. XXXIV. FEBRUARY, 1908. No. 2.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

THE ELECTRIFICATION OF THE SUBURBAN ZONE OF THE NEW YORK CENTRAL AND HUDSON RIVER RAILROAD IN THE VICINITY OF NEW YORK CITY.

By William J. Wilgus, M. Am. Soc. C. E.

To be Presented March 18th, 1908.

The recent successful completion of the electrification of the service of the New York Central and Hudson River Railroad entering the Grand Central Terminal, New York City, marks such an important step in the progress of the art of transportation that a paper seems at this time appropriate, explaining the reasons for the abandonment of steam, the general features of construction and operation, and the results.

Two decades have passed since electricity in the United States first commenced its important career in the field of lighter traffic; but only within the past few months has it fairly met its steam rival in heavy-traction trunk-line service.

Reasons for Delay in Electrification of Trunk Lines.—The reason for this delay is not far to seek. The steam locomotive, during its lifetime of eighty years, has been developed into a wonderfully reliable, efficient, and powerful machine, deep-seated in the affections of

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

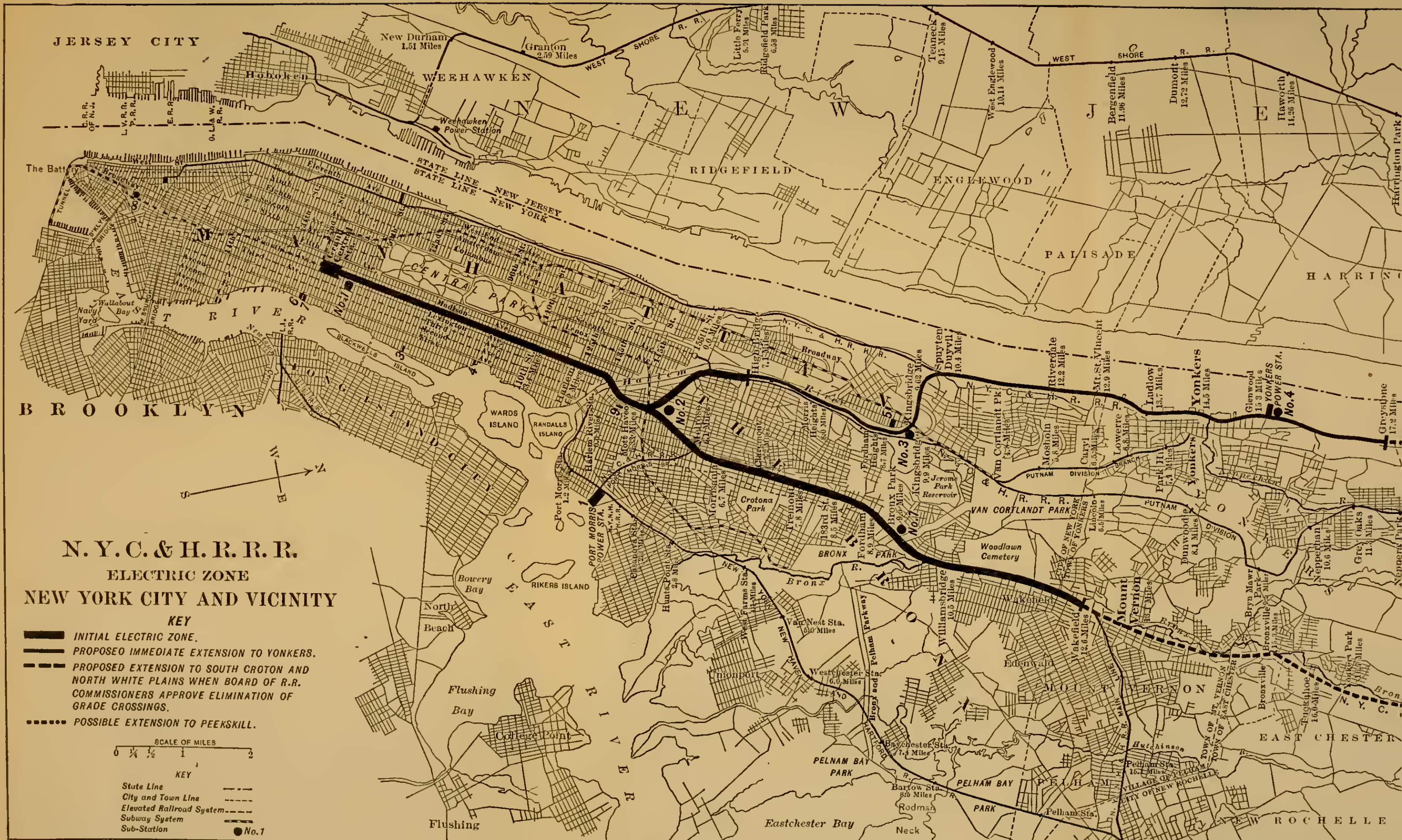
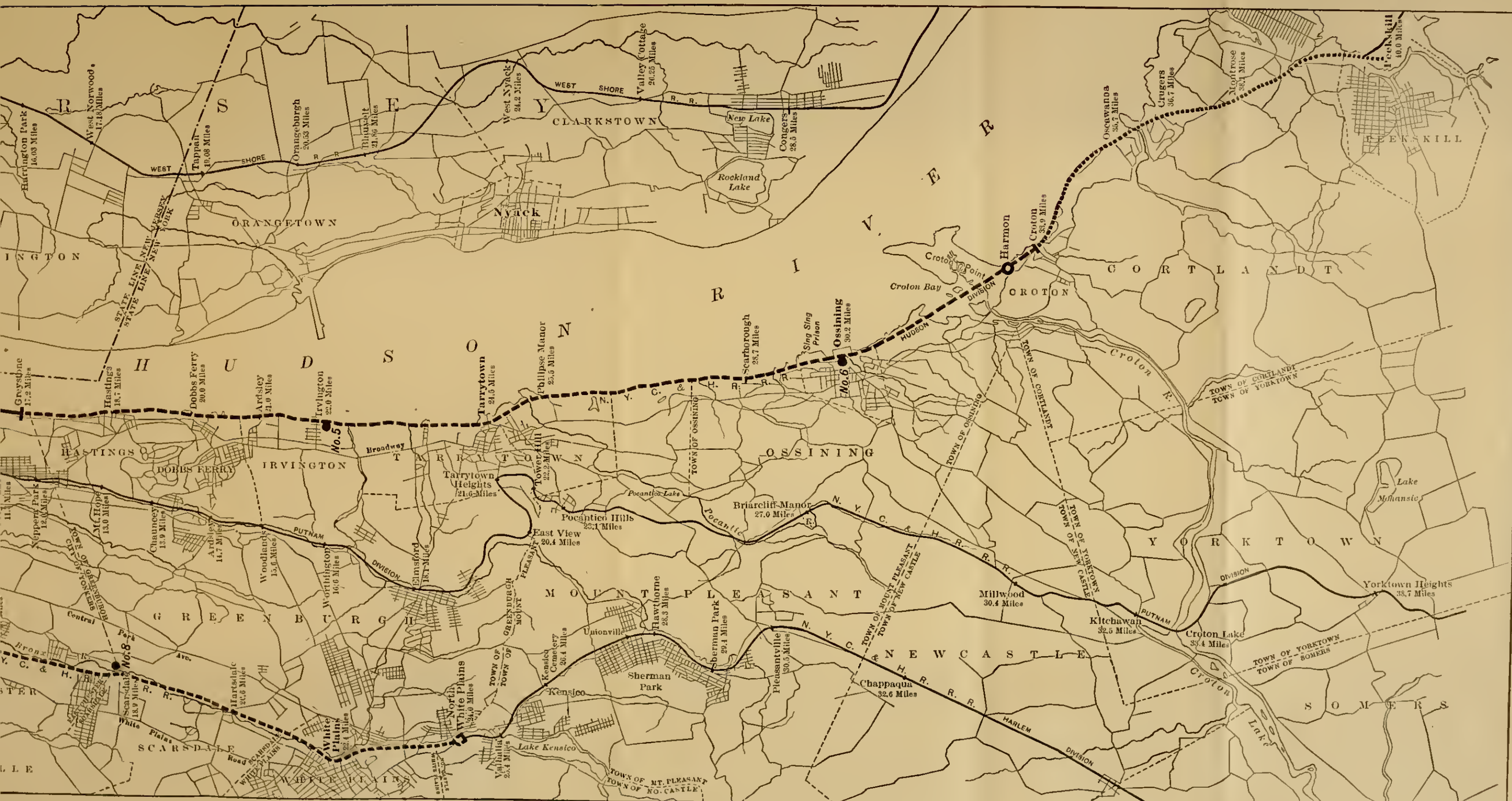

N. Y. C. & H. R. R. R.
ELECTRIC ZONE
NEW YORK CITY AND VICINITY
KEY
INITIAL ELECTRIC ZONE.
PROPOSED IMMEDIATE EXTENSION TO YONKERS.
PROPOSED EXTENSION TO SOUTH CROTON AND NORTH WHITE PLAINS WHEN BOARD OF R.R. COMMISSIONERS APPROVE ELIMINATION OF GRADE CROSSINGS.
POSSIBLE EXTENSION TO PEEKSKILL.
SCALE OF MILES
0 ¼ ½ 1 2
KEY
State Line
City and Town Line
Elevated Railroad System
Subway System
Sub-Station
No.1
JERSEY CITY
Hoboken
New Durham 1.51 Miles
Granton 2.59 Miles
WEST SHORE R. R.
WEEHAWKEN
Weehawken Power Station
Little Ferry 6.01 Miles
Ridgefield Park 6.58 Miles
Teaneck 9.15 Miles
West Englewood 10.14 Miles
Bergenfield 11.96 Miles
Dumont 12.72 Miles
Haworth 14.96 Miles
Harrington Park
RIDGEFIELD
ENGLEWOOD
PALISADE
HARRINGTON
STATE LINE NEW JERSEY
STATE LINE NEW YORK
The Battery
BROOKLYN
EAST RIVER
BLACKWELLS ISLAND
WARDS ISLAND
RANDALLS ISLAND
RIKERS ISLAND
CENTRAL PARK
Harlem River
N. Y. C. & H. R. R. R.
Broadway
Spuyten Duyvil 10.4 Miles
Kingsbridge
Riverdale 12.2 Miles
Mt. St. Vincent 12.9 Miles
Ludlow 13.7 Miles
Yonkers 14.5 Miles
Glenwood 15.3 Miles
YONKERS POWER STA.
No. 4
Greystone 17.2 Miles
No. 2
No. 3
PORT MORRIS POWER STA.
Port Morris Sta. 1.2 Miles
Crotona Park
Jerome Park Reservoir
VAN CORTLANDT PARK
PUTNAM DIVISION
Woodlawn Cemetery
BRONX PARK
Bronx R.
Williamsbridge 10.5 Miles
Wakefield 12.6 Miles
Mount Vernon
Van Nest Sta. 5.0 Miles
Westchester Sta. 6.0 Miles
Unionport
Baychester Sta. 7.4 Miles
Bartow Sta. 8.5 Miles
PELHAM BAY PARK
Rodman Neck
Eastchester Bay
Edenwald
MOUNT VERNON
PELHAM
Pelham Sta.
EAST CHESTER
NEW ROCHELLE
Bronxville
Tuckahoe 16.0 Miles
Bowery Bay
North Beach
Flushing Bay
College Point
Flushing

PLATE V.
PAPERS AM. SOC. C. E.
FEBRUARY, 1908.
WILGUS ON
ELECTRIFICATION OF SUBURBAN ZONE
OF N. Y. C. & H. R. R. R.



the railroad world. With the conservatism naturally born of these conditions is the reluctance of stockholders to spend vast sums for changes of unproven financial value.

There is no cause for surprise, therefore, that electricity, so commonly associated in the mind of the railroad officer with light street-car traffic, has not been seriously considered as a substitute for steam, until special problems have arisen demanding some escape from the limitations and nuisances incident to the use of steam locomotives.

The very fact that steam locomotives have grown so in size and power makes them more objectionable as emitters of increased volumes of noise, smoke, gas, and cinders.

The first important instance of the use of electricity on a large scale was in utilizing electric locomotives to push solid trains, with their inactive steam locomotives, through the Baltimore Tunnel of the Baltimore and Ohio Railroad. In this instance, electricity was adopted as an aid, not as a substitute for steam.

Reasons for Electrification of New York Central.—As early as 1899, thought was given to the use, on the New York Central, of electricity for curing the evils at the entrance to the Grand Central Terminal; but it was not until 1903 that the objectionable atmospheric conditions in the Park Avenue Tunnel, and the congestion of traffic at the terminal, precipitated legislative action directing the complete abandonment of the steam locomotive in Park Avenue south of the Harlem River, within a period of five years terminating July 1st, 1908.

In the same year the railroad company and the city agreed upon radical changes at the terminal, which were possible only with the abandonment of steam. From a civic standpoint, the most important of these changes is the depression of the whole terminal, so as to permit the extension of highways over the tracks from Forty-fifth to Fifty-sixth Streets, inclusive, and the continuation of Park Avenue, 140 ft. wide, within the same limits, thus joining two sections of the city hitherto separated for ¾ mile by an impassable barrier of railroad yards and structures.

Reasons for Extended Scope of Electrification.—A careful analysis of the situation soon proved the absurdity of terminating the electric zone at or near the Harlem River.

Immediately north of that point is Mott Haven Junction, where the line splits, one leg known as the Harlem Division continuing north to Chatham on the Boston and Albany Railroad, and the other constituting the main line of the Hudson Division, bearing to the west and north, along the banks of the Hudson River, to Albany and beyond. At Woodlawn Junction, on the Harlem Division, is the point of confluence with the New York, New Haven and Hartford Railroad, the very large passenger traffic of which flows over the rails of the New York Central to and from the Grand Central Terminal, a distance of 12 miles.

From the Grand Central Terminal to Woodlawn Junction, the Harlem Division is four-tracked, but for the remainder of the distance within the territory under discussion, but two tracks existed for handling all classes of traffic. Similarly, on the main line, from the junction at Mott Haven, two tracks were called upon to transport both passenger and freight trains, except on the section between Spuyten Duyvil and Scarborough, where a third track aided to some extent.

In addition to the extremely heavy through passenger train service from the New England, northern and western States of the Union, and Canada, there is an important local traffic extending as far out as Harmon, on the Hudson Division, a distanc of 33 miles, North White Plains, on the Harlem Division, a distance of 24 miles, and Stamford, on the New York, New Haven and Hartford Railroad, a distance of 34 miles, from the Grand Central Station.

A further burden on the four-track stem between the terminal and Mott Haven Junction is the hauling between those points of "dead" equipment, because of inadequate storage space at the station.

From this recital it will be seen that a termination of the electric zone at the Harlem River, or at Mott Haven Junction just above the river, would entail the stoppage and change of motive power from steam to electricity and *vice versa*, of all kinds of traffic, at a point peculiarly subject to congestion. Moreover, the physical conditions in the neighborhood precluded the construction of the necessary facilities for the storage and care of motive power.

Because of these fundamental objections, and, moreover, guided by the broad-minded policy that growth of traffic responds to the use of electricity, the company decided to extend the limits of the electric zone to the northerly termini of the suburban territory, at Harmon and North White Plains, where ample space is available for loops, yard tracks, and buildings. The geography of the territory is shown on Plate V.

PLATE VI.
PAPERS, AM. SOC. C. E.
FEBRUARY, 1908.
WILGUS ON
ELECTRIFICATION OF SUBURBAN ZONE
OF N. Y. C. & H. R. R. R.

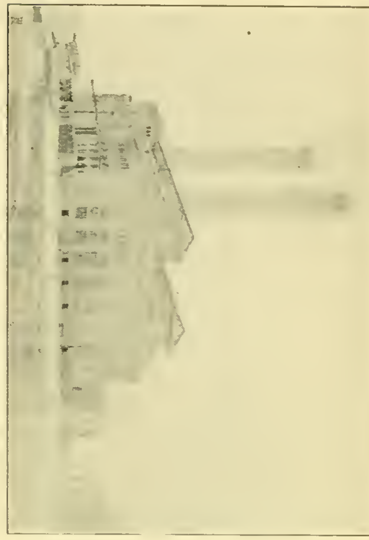


Fig. 1.—Port Morris Power Station.

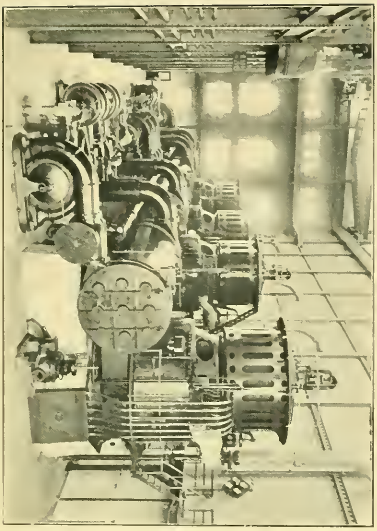


Fig. 2.—Interior, Port Morris Power Station.

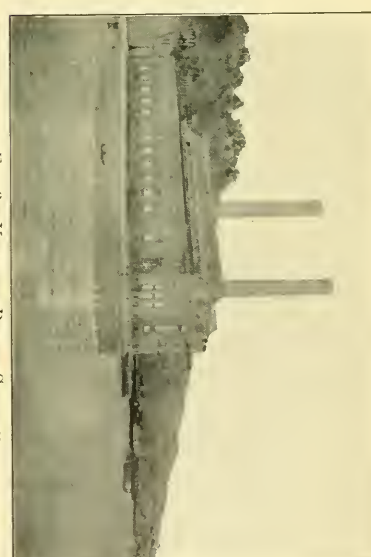


Fig. 3. Yonkers Power Station.

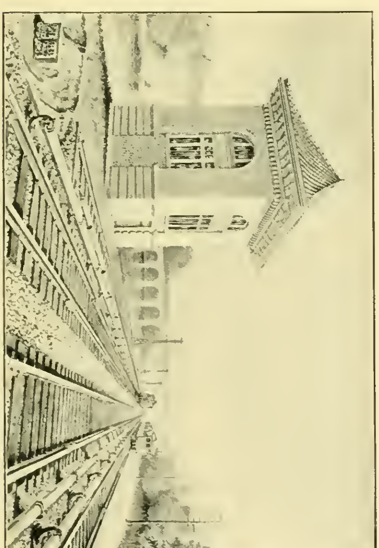


Fig. 4.—Cable Tower.

Reasons for Other Improvements.—This decision, and the demands of growing traffic for more and better facilities, led to the adoption of plans for a new Grand Central Station with two track levels; the separation of track grades and a new overhead eight-track station at Mott Haven; the elimination of all grade, street, and highway crossings; the "four-tracking" of both divisions as far as the termini; many new and enlarged passenger and freight stations; new electric automatic signals, and electric interlocking plants; and many important revisions of alignment and grades.

Reasons for Adopting the Direct-Current System.—About this time the battle had just opened in the United States between the two rival systems of electricity—direct and alternating current. Of course, the advocates of each argued that the other was unsuited to New York Central conditions, and it was only after lengthy and thorough consideration that the direct-current system was selected.

The principal reasons for this conclusion, apart from technical points, may be summarized as insufficient practical development of the alternating-current system for a trunk-line problem requiring absolute reliability of service, restricted clearances which forbade the use of overhead conductors, and legal obstacles to the use of overhead trolley wires carrying high voltages within the limits of the City of New York.

Reasons for Not Using Alternating-Current Equipment on a Direct-Current System.—Some time after this decision had been made, and apparatus had been ordered, the company was urged by outside interests to abandon the type of equipment suited exclusively to the direct-current system, and adopt another type which could operate on both direct and alternating currents. It was claimed that, by making this change, the equipment would be available for use on later extensions of the electric zone where there were no physical or legal objections to the use of alternating current. The wisdom of adhering to the type of equipment already chosen has been proven by recent comparative tests of locomotives of the two types under exactly the same conditions, which demonstrate that the one designed only for direct current consumes from 15 to 25% less current than the one intended for use on both systems. This will effect a saving to the company of at least $140 000 per annum. If to this item is added the economy resulting from less locomotive ton-miles per annum because of the lower weights of locomotive per unit of capacity, and lower wages, fixed charges, and maintenance of equipment, because of the smaller number needed to do the same work, the total saving for the ultimate electric zone, resulting from adherence to the adopted type of direct-current locomotive, will be approximately $300 000 per annum.

Reasons for Duplicate Power Stations and Transmission Lines.—One of the strongest arguments advanced against the substitution of electricity for the well-tried steam locomotive, for the movement of the most important passenger, mail, and express service in the country, is the vulnerability of power stations and distributing systems to failures of the class which affect, not one, but all, units. To overcome this well-founded criticism, two cross-connected power stations were decided upon, accessible to both rail and boat coal; and each with sufficient capacity, utilizing its spare unit, and working "overload," to carry the entire demand of the service at the rush hours should the other fail. It was considered that the growing familiarity of the operating force with the new conditions, and the elimination, in time, of unsuspected defects of installation, would later make the surplus capacity available for other uses, such as increased demands of traffic, the movement of freight trains by electricity, and the operation of the terminals of the company on the west side of Manhattan Island. As a further precaution, duplicate transmission lines were adopted in the more important portions of the territory, so that the failure of one would still leave the other effective for the uninterrupted movement of trains.

Reasons for Storage Batteries.—Even with these two safeguards, there appeared to be vulnerable places, where accidents might put essential features of the service out of commission, and to overcome this, as well as to make suitable regulation of violent fluctuations of load on the power stations and sub-stations, storage batteries were adopted with capacity sufficient to tide over the usual maximum periods of interruption of current supply, that experience elsewhere has shown may be expected.

Reasons for Combined Locomotive and Multiple-Unit Practice.—While, necessarily, through trains with cars originating at far distant points must be hauled by electric locomotives within the electrified territory, it was evident from the start that, for the company to reap the full advantage from its expenditures, the multiple-unit type of suburban equipment should be adopted that elsewhere had been shown was essential for the propagation of traffic, and the simplification of

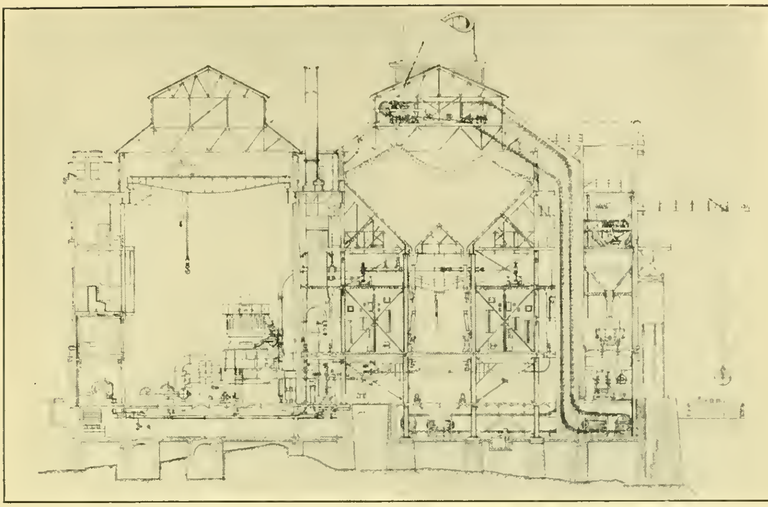


FIG. 1.—TYPICAL CROSS-SECTION OF THE PORT MORRIS POWER STATION.

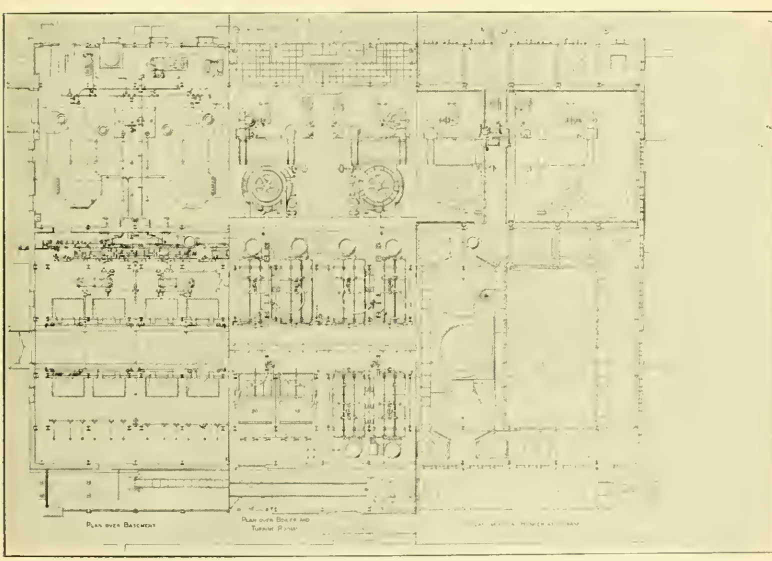


FIG. 2.—TYPICAL PLAN OF PORT MORRIS POWER STATION.

operation in congested terminals. By dispensing with locomotives in suburban service, and equipping the individual cars with electric motors controlled from either end of the train, it becomes possible to meet the demand of the public for less interval between trains, and at the same time regulate the cost of operation to the volume of traffic at various periods of the day. The absence of locomotives, and the distribution of power among the cars practically eliminates switching, and movements to and from engine-houses, with a resultant great reduction of the causes that congest terminals. A twofold character of equipment, therefore, was adopted—locomotives for through trains, and multiple-unit cars for the passenger service confined to the electric zone.

Awarding First Contracts.—With all the foregoing questions settled, plans and specifications were actively prepared, and contracts awarded in the fall of 1903 for the apparatus requiring the longest time for delivery, including power-station machinery and locomotives. Later, arrangements were made for the remaining items of the installation, either by contract or by company forces.

General Features of Construction.

Principal Elements of the Installation.—The principal elements of the installation are the duplicate power stations for generating 3-phase, 11 000-volt, 25-cycle alternating current; the high-tension transmission lines for distributing this current to the sub-stations; the sub-stations for transforming and converting the high-tension alternating current to 660-volt direct current, and for the storage of current in batteries; the direct-current transmission lines for the distribution of energy to the working conductors; the third-rail and overhead conductors at special places, known as working conductors, for the delivery of the 660-volt current to the contact shoes on locomotives and cars; the electrical equipment; repair shops and inspection facilities; interchange terminals for electric and steam power; and last, but not least, the building up of an operating organization to make all this intricate machinery a working success.

Power Stations.—Each power station is equipped initially with sixteen water-tube, 625-h.p. boilers, with superheaters and mechanical stokers, and four 5 000-k.w. Curtis turbo-generators, together with the necessary condensers, pumps, exciters, feed-water heaters, and appurtenances. Additional space is provided in the buildings, for a later expansion of capacity to the extent of 50% of the initial installation. It will thus be seen that each station has a present normal capacity of approximately 28 000 h.p. (20 000 k.w.), with provision for an ultimate increase to approximately 42 000 h.p. (30 000 k.w.), or a combined ultimate normal capacity of approximately 84 000 h.p. (60 000 k.w.). The two stations are electrically cross-connected, so that, for all practical purposes, they act as one.

Both power stations are supplied with mechanical plants for transferring coal from car or boat to overhead bins, each station having a storage capacity of 3 500 tons, equal to 9 days' supply under maximum conditions.

A pilot switch-board is located in the gallery of each power station, but the important control apparatus, including oil switches, is placed in a separate building, so that serious trouble in the main structure will not disable or injure what may be termed the brains of the system.

The buildings are constructed substantially, of concrete, brick and steel, on stable foundations, and with an architectural treatment suited to the purposes for which they are designed. The twin stacks at each station are of perforated radial brick, have an average internal diameter of 16 ft. 3 in., and rise to a height of 267 ft. above the ground.

A noteworthy fact may be recorded that illustrates one of the advantages of turbo-generators in the economical design of power-station buildings. The capacity required at the Yonkers station is only 110 cu. ft., and that at the Port Morris station 115 cu. ft. per k.w., as compared with from 170 to 255 cu. ft. at the more important reciprocating-engine plants in New York City.

The maximum calculated 4-min. peak load on both stations is 24 000 k.w., at which time 38 trains, of varying speeds, weighing in all 9 800 tons, are assumed to be in motion. The annual output is expected to aggregate 121 000 000 kw-hr., of which 107 000 000 kw-hr. are for the propulsion load and the remainder for lighting and other purposes. These figures do not include the future additional requirements for switching at various yards, the movement of freight trains, and the operation of labor-saving devices at terminals.

11 000-Volt Transmission Line.—The 11 000-volt alternating current from the power stations is led to the sub-stations by duplicate systems of insulated copper cables in ducts within the populous districts of the city; and by bare copper cables suspended on substantial steel poles set

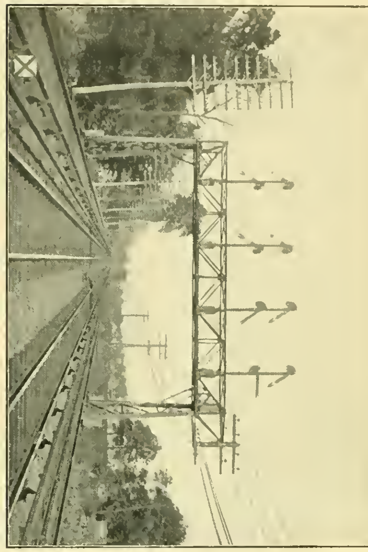


Fig. 1.—Aerial Transmission Line.

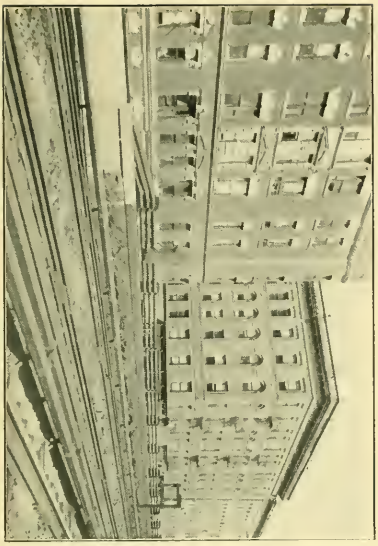


Fig. 2.—Duct Transmission Line on Viaducts.

Fig. 3.—Duct Transmission Line on Retaining Walls.

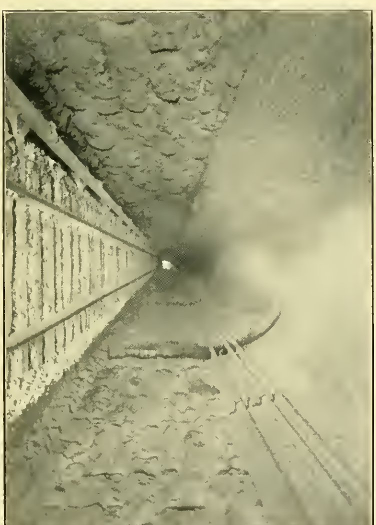


Fig. 4.—Duct Transmission Line in Park Avenue Tunnel

FIG. 1.—TYPICAL SPLICING CHAMBER.

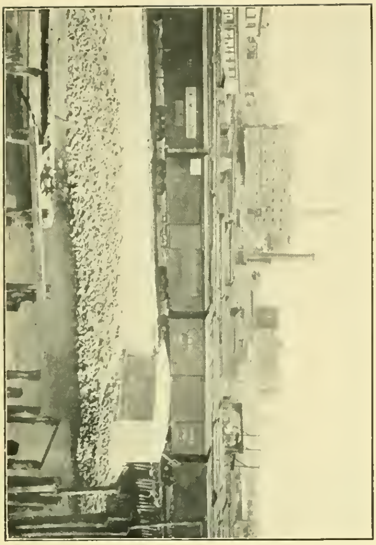


FIG. 2.—SUBMARINE CROSSING, HARLEM RIVER.

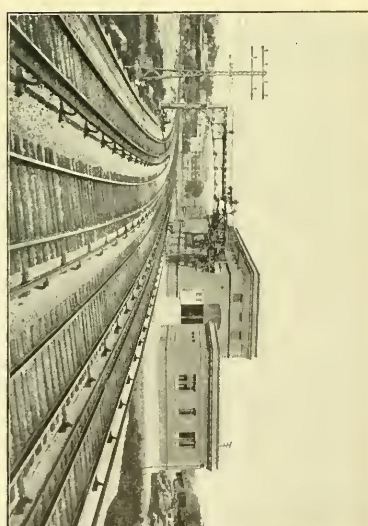


FIG. 3.—TYPICAL SUB-STATION.

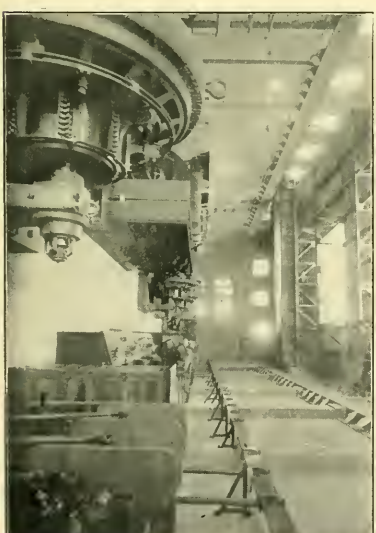


FIG. 4.—INTERIOR OF SUB-STATION.

PLATE X.
PAPERS, AM. SOC. C. E.
FEBRUARY, 1908.
WILGUS ON
ELECTRIFICATION OF SUBURBAN ZONE
OF N. Y. C. & H. R. R. R.

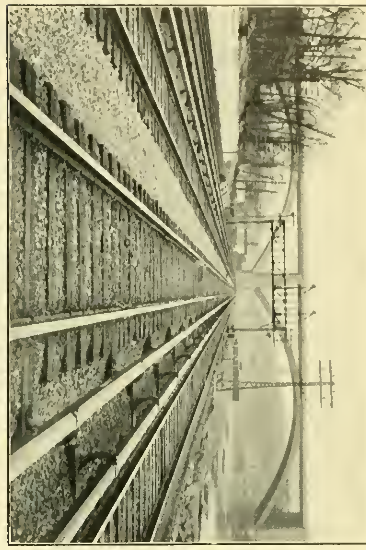


Fig. 1.—View of Completed Third-Rail.

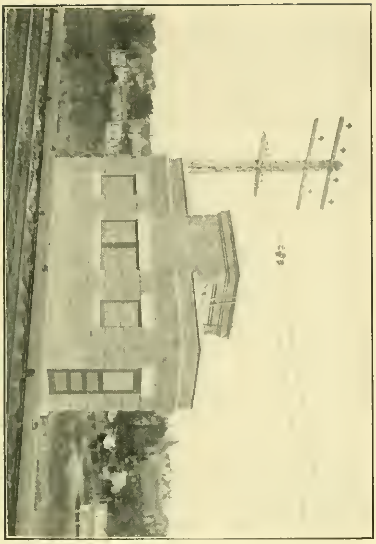


Fig. 2.—Circuit-Breaker House.

Fig. 3.—Operation of Third-Rail in Winter.

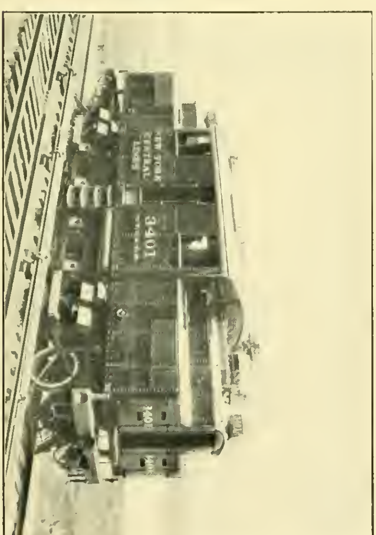


Fig. 4.—Electric Locomotive.

in concrete bases in the less densely settled districts. This arrangement was adopted only after an exhaustive investigation of line construction throughout the country had proven the greater safety and reliability of well-built aerial wires, where the population is sparse and the line is located on private right of way.

Where the cables pass from one type of construction to the other, they are led through brick towers equipped with lightning arresters.

Owing to the failure of the city to grant the right to place the cables beneath the surface of neighboring streets, it was necessary to locate them within the right-of-way limits of the company, and this required many varied types of construction, often taxing the ingenuity of the engineers to place the conduit pipes where they would be safe from injury. A few of these conditions are illustrated in the accompanying photographs. Altogether, there will be 16 miles of conduit territory, and 46 miles of pole lines, together with 383 splicing chambers.

Sub-stations.—There are to be eight sub-stations, four of which are now in operation. Their total normal rotary capacity will be 27 000 k.w.

Each station contains transformers for reducing the voltage from 11 000 volts primary to 450 volts secondary, and rotary converters for changing the current from alternating to direct at 660 volts. Storage batteries, "floating on the line," are also provided, to regulate the sharp fluctuations of the peculiarly severe short-period demands incident to heavy traction service, and to safeguard the continuity of traffic should perchance the supply of current be interrupted by power station or distributing failures. This insurance of reliability of service has already demonstrated the wisdom of its adoption. The aggregate momentary capacity of the batteries will be 37 786 k.w., with an hourly capacity of 12 595 k.w.

660-Volt Feeder System.—The 660-volt direct-current system, for conveying energy to the working conductors, consists of copper cables protected and arranged similarly to the high-tension lines already described.

Working Conductors.—The working conductors deliver 660-volt current to locomotives and cars. Third-rail is used at all points, except where intricate switch lay-outs prohibit a continuous conductor near the level of the track. At such places overhead conductors are used,

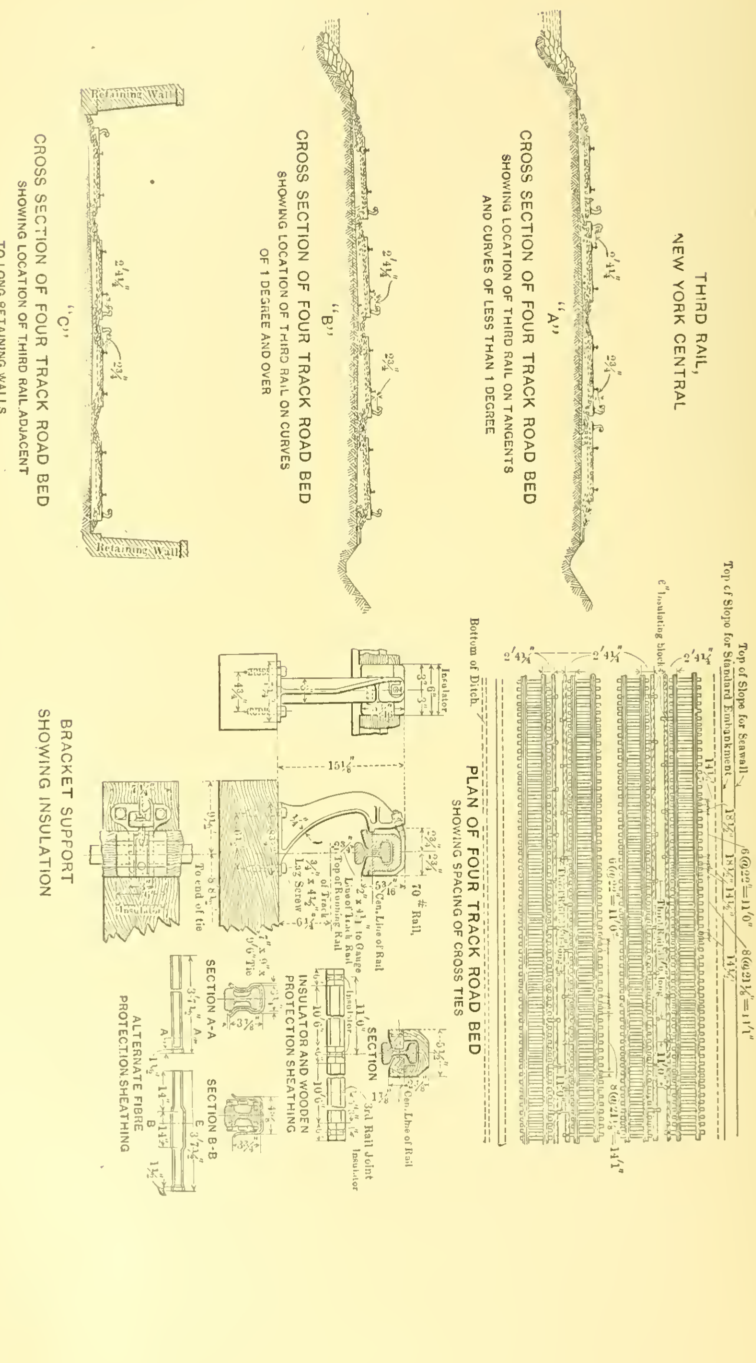

THIRD RAIL,
NEW YORK CENTRAL
CROSS SECTION OF FOUR TRACK ROAD BED
SHOWING LOCATION OF THIRD RAIL ON TANGENTS
AND CURVES OF LESS THAN 1 DEGREE
"A"
CROSS SECTION OF FOUR TRACK ROAD BED
SHOWING LOCATION OF THIRD RAIL ON CURVES
OF 1 DEGREE AND OVER
"B"
CROSS SECTION OF FOUR TRACK ROAD BED
SHOWING LOCATION OF THIRD RAIL ADJACENT
TO LONG RETAINING WALLS
"C"
Retaining Wall
Top of Slope for Seawall
Top of Slope for Standard Embankment
Bottom of Ditch
PLAN OF FOUR TRACK ROAD BED
SHOWING SPACING OF CROSS TIES
BRACKET SUPPORT
SHOWING INSULATION
Insulator
70 # Rail
Cen. Line of Rail
Top of Running Rail
Lag Screw
To end of tie
INSULATOR AND WOODEN
PROTECTION SHEATHING
SECTION A-A
SECTION B-B
ALTERNATE FIBRE
PROTECTION SHEATHING
SECTION
3rd Rail Joint

Fig. 1.

FIG. 1.—MULTIPLE-UNIT TRAIN.

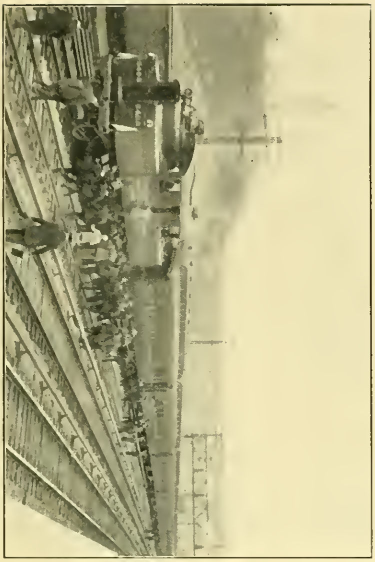


FIG. 2.—FIRST ELECTRIC TRAIN LEAVING HIGH BRIDGE FOR GRAND CENTRAL STATION.

FIG. 3.—"JUMPER" CONNECTION BETWEEN DIRECT-CURRENT FEEDER AND THIRD-RAIL.

FIG. 4.—REACTANCE BOND.

either of a temporary character where future track changes are contemplated, or permanently suspended from overhead bridges and buildings. The adopted type of third-rail is unique, for the reason that the current is collected from beneath instead of from the top. This permits the sides and upper parts of the rail to be sheathed in wood or other insulating material in a way that safeguards employees and others from accidental contact, and protects the contact surface from sleet and snow which, with the usual types of top-contact rail, so frequently cause tie-ups of traffic. The manner of construction is such as to secure all these advantages, without encroachment within the clearance lines of the steam equipment, and without precluding the interchange of electric equipment with other lines already using the top-contact type.

At frequent intervals, the direct-current cables pass through small circuit-breaker houses, in which circuit-breakers automatically open and interrupt the flow of current, when, because of accident or injury, there is an improper leak in the third-rail system or the direct-current feeder system. This safety device, therefore, automatically checks the delivery of current to the working conductors, when a continuation of the supply might be disastrous. The circuit-breakers are controlled by cables connected with neighboring sub-stations. Numerous other precautionary measures have been taken for shutting off power promptly in case of accident, such as, for instance, continuous indicator wires for each of the four tracks in the Park Avenue Tunnel, that enable the power to be shut off immediately on any desired track.

In all, there will be 52 miles of territory, embracing 285 miles of track equipped with third-rail, of which more than one-third is completed and in use.

Track Bonds.—The bonding of the track rails for the return current was a task of considerable proportions, because of the intimate relation of the work to traffic. Several ingenious devices were used in expediting the drilling of rails and placing the bonds. The concealed type of bond was used as a protection against the thefts that embarrass traffic and entail pecuniary loss, and to obviate injury by trackmen.

Electrical Equipment.—The electrical equipment now in use comprises 35 locomotives and 180 suburban cars. Of the cars, 125 are equipped with motors. The remainder, for the present, act as trailers, although motors will be added when the electrical service is extended

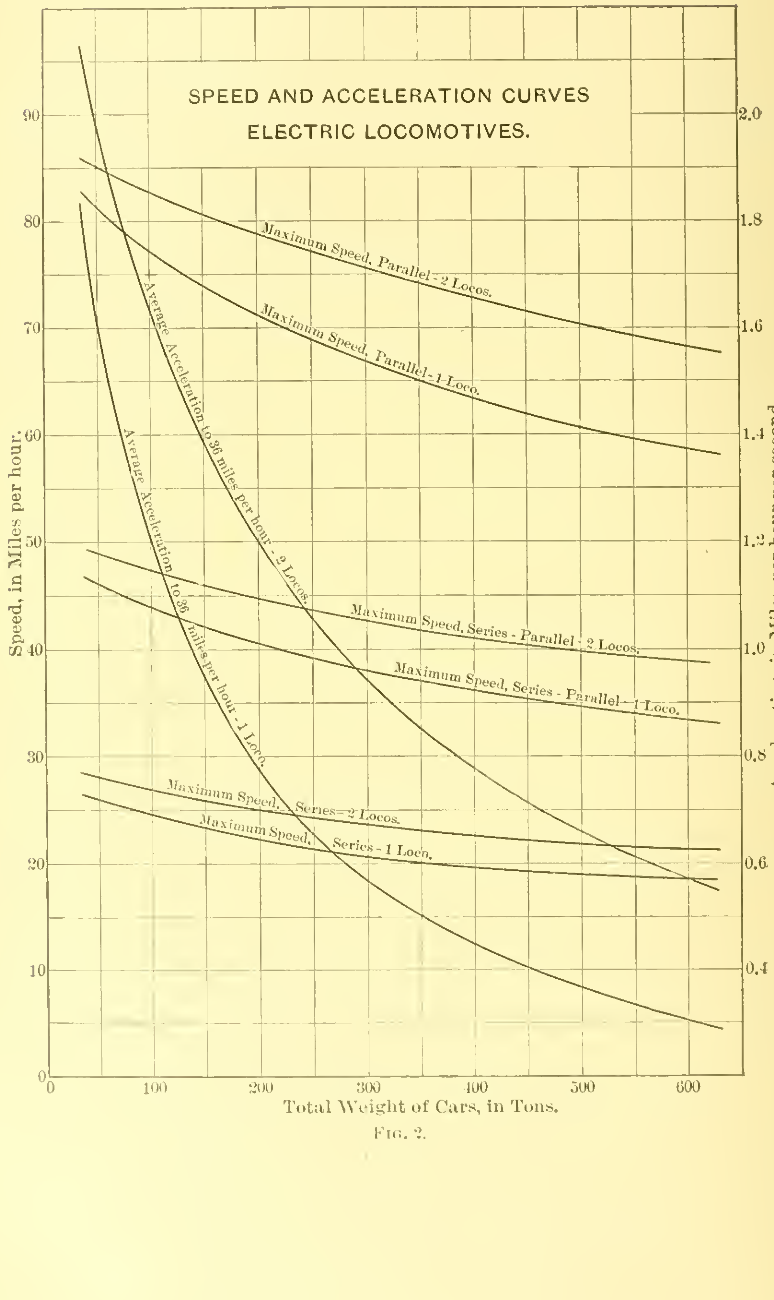

SPEED AND ACCELERATION CURVES
ELECTRIC LOCOMOTIVES.
Maximum Speed, Parallel-2 Locos.
Maximum Speed, Parallel-1 Loco.
Average Acceleration to 36 miles per hour-2 Locos.
Average Acceleration to 36 miles per hour-1 Loco.
Maximum Speed, Series-Parallel-2 Locos.
Maximum Speed, Series-Parallel-1 Loco.
Maximum Speed, Series-2 Locos.
Maximum Speed, Series-1 Loco.
Speed, in Miles per hour.
Acceleration, in Miles per hour per second.
Total Weight of Cars, in Tons.
0
10
20
30
40
50
60
70
80
90
100
200
300
400
500
600
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0

Fig. 2.

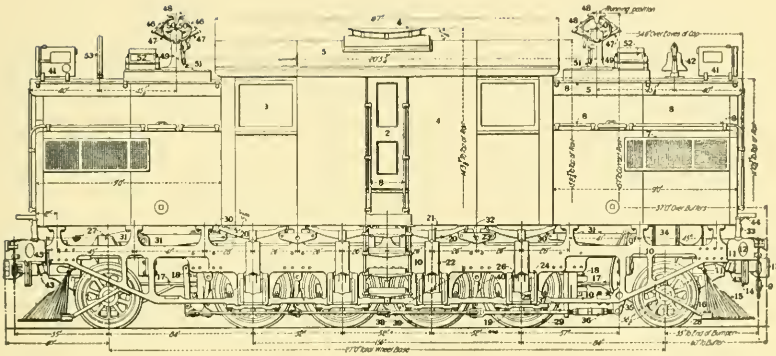


FIG. 1.—ELECTRIC LOCOMOTIVE.

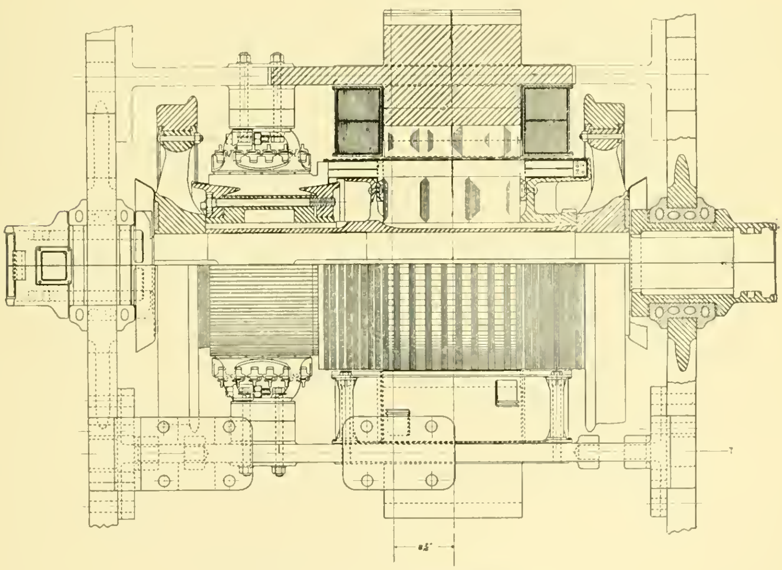


FIG. 2.—BI-POLAR GEARLESS MOTOR.

the full distance to Harmon and North White Plains. The aggregate normal rating of both classes of equipment is 127 000 h.p.

Locomotives.—The locomotive is a peculiarly efficient and powerful machine. Although weighing 94.5 tons, complete, as compared with the 171-ton weight of the heaviest steam passenger locomotives in use by the company, its normal rating of 2 200 h.p. is practically twice that of its rival; it has 76½ tons less weight to haul about, thus effecting a saving of 45% for energy in moving dead tonnage; its concentrated weight per driving axle, 34 250 lb., is 27% less than that of the steam locomotive, without decreasing the total driver weight available for traction; it is capable of running at will in either direction, without the delays and expense of going to the turn-table; it occupies little more than half the track space of the steam locomotive—an important advantage in terminals—and it is much more quickly started and stopped. These advantages have been demonstrated strikingly in practice, both in comparative trials on the 6-mile experimental track near Schenectady, where all the new equipment was tested exhaustively before acceptance, and in regular service in the New York zone.

The principal characteristics of the locomotive are:

Length over all	37 ft. 0 in.
Rigid wheel base	13 " 0 "
Total wheel base	27 " 0 "
Diameter of drivers	44 "
Diameter of truck wheels	36½ "
Total weight	94½ tons.
Weight on four drivers	68½ "
Weight on two trucks	26 "
Horse-power per ton of weight—normal capacity	23
Horse-power per ton of weight—overload capacity	35
Number of motors	4
Normal capacity of each motor	550 h.p.
Normal capacity of each locomotive	2 200 "
Over-load capacity of each locomotive	3 300 "
Type of motors	Gearless, bi-polar.
Type of control	Sprague-General Electric multiple-unit.
Type of heaters for train supply	Westinghouse oil-fired.
Air brakes	Westinghouse graduated-release.

Cars.—The suburban cars are constructed of steel and other non-inflammable material, and, while simple in design, have all the features conducive to the safety and comfort of the public. Their leading characteristics are as follows:

Length, over all	62 ft. 0 in.
Length of car body	50 " 0 "
Distance between truck centers	38 " 6 "
Distance between axles of motor trucks	7 " 0 "
Distance between axles of trailer trucks	6 " 0 "
Diameter of wheels—motor trucks	36 "
Diameter of wheels—trailer trucks	33 "
Number of motors on each motor truck	2
Normal capacity of each motor	200 h.p.
Normal capacity of motor car	400 "
Total weight of motor car	53 tons.
Total weight of trailer car	44½ "
Total weight of car body	33½ "
Weight per motor car, due to electrical equipment	8½ "
Horse-power (normal capacity) per ton of weight of electrical equipment	47
Seating capacity	64
Heating system	Both steam and electric.
Lighting system	Both electric and Pintsch gas.
Cooling system for summer season	Two 14-in. electric fans.
Type of control	Sprague-General Electric multiple-unit.
Acceleration, in miles per hour per second	1.2

Comparative Train Weights.—The comparative weights of steam and electric trains in the two classes of service, through and suburban, are interesting, as illustrative of the saving in consumption of energy, and therefore in cost of operation, that accompanies the lower electric train weights; and, also, as justifying the adoption of the multiple-unit instead of locomotive practice for suburban operation.

THROUGH SERVICE.

Steam.	Tons.	Electric.	Tons.
Pacific type locomotive	171.0	Electric locomotive	94.5
8 Pullman cars	400.0	8 Pullman cars	400.0
Total	571.0	Total	494.5

Saving in favor of electric traction = 76½ tons = 13 per cent.

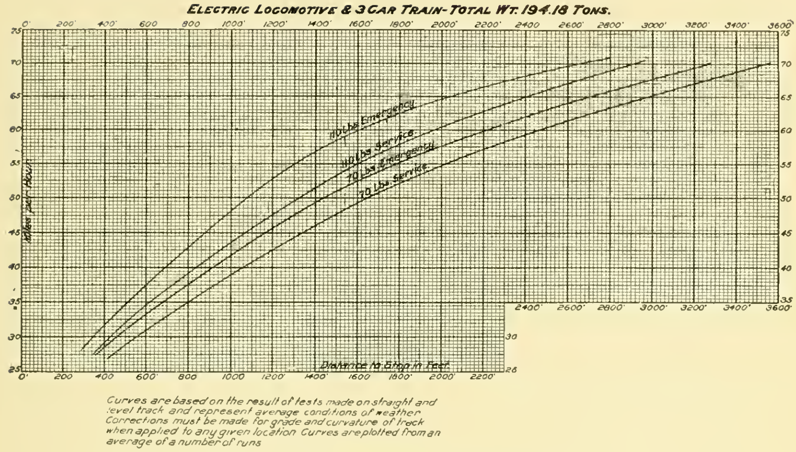


FIG. 1.—TRAIN BRAKING CHART—DISTANCE.

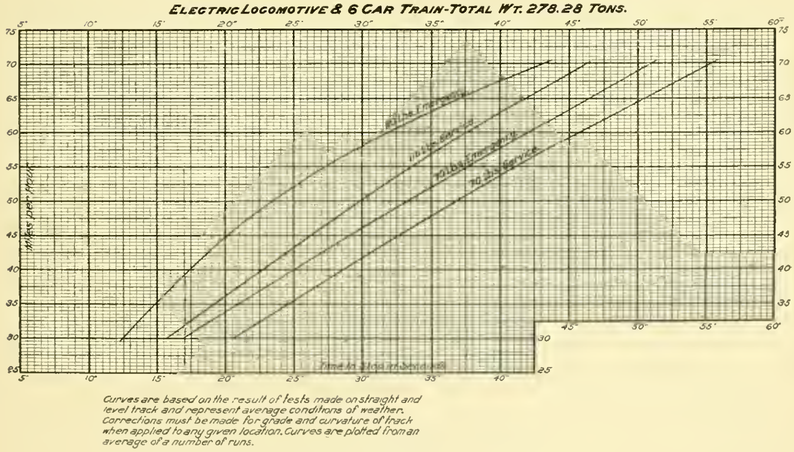


FIG. 2.—TRAIN BRAKING CHART—TIME.

SUBURBAN SERVICE.

Average Number of Cars.

Electric Locomotive.	Tons.	Multiple-Unit Cars.	Tons.
Locomotive	94.5		
4½ steel trailer cars.....	200.0	4½ motor cars..........	238.5
Total............	294.5	Total............	238.5

Saving in favor of multiple-unit practice = 56 tons = 19 per cent.

Shops and Inspection Sheds.—The maintenance of electrical equipment in a high degree of efficiency requires suitable repair shops and inspection sheds, located where the dead mileage will be reduced to a minimum. At both Harmon and North White Plains permanent inspection sheds have been built, and at the former point ample modern shop facilities are provided. As the equipment on both divisions is pooled, any car or locomotive needing repairs can be sent while in regular service to either place, without the expense and loss of time incident to special dead movements.

Interchange Terminals.—At North White Plains, the existing steam engine-house plant is to be enlarged when the extension of electric operation requires added facilities for the interchange of power. At Harmon, space has been provided for ample facilities for the same purpose. Owing to the present curtailment of electric operation because of the backwardness of the State in acting on the abolition of grade crossings north of the limits of the City of New York, temporary terminals have been constructed, at High Bridge on the Hudson Division and at Wakefield on the Harlem Division, with convenient yard arrangements and structures for the care and exchange of power.

Operating Organization of Electrical Department.—The success of a new plant of such magnitude, especially when a change from old to new conditions must be effected without embarrassing an enormous passenger traffic, depends very largely on the organization and personnel of the electrical operating force. It was recognized, at an early stage of the work, that the operation and maintenance of the entire installation required to deliver current to equipment, as well as the maintenance of locomotives and cars, should be under the supervision of those responsible for their construction, leaving to the regular steam organization the operation of trains with electric current and equipment thus furnished.

As the work on the power stations, distributing system, and equipment progressed, competent men were gradually employed for inspection and testing purposes, so that, when all was ready for regular operation, there was in existence a skilled, energetic corps of veterans, equal to any emergency, and imbued with a spirit that meant success.

Telephone System.—A word should here be spoken of the independent telephone system which has been constructed for the purpose of bringing all parts of the electric zone in close touch with each other and with the load and train dispatchers.

Other Improvements.—While the principal purpose of this paper is to give an outline of the elements of the electrification of the New York Central suburban zone, it would be incomplete without at least a passing mention of the other important improvements undertaken in conjunction with the change of motive power.

Grand Central Terminal.—Within the territory bounded by Forty-second Street, Fifty-seventh Street, Madison Avenue and Lexington Avenue, the old Grand Central Terminal occupied a parcel of irregular shape, with an area of about 23 acres. The four main tracks from the north descend on grades of from 26 to 53 ft. per mile to the south end of the Park Avenue Tunnel at Fifty-sixth Street; thence they ascend at the rate of 62 ft. per mile in an open cut in the middle of Park Avenue to Fiftieth Street; thence spreading out into the yard, on a slight descent to Forty-fifth Street; and thence on a gentle declivity to the terminal in the train-shed near Forty-third Street. The vital defect of this arrangement was the absence of switching tracks for drilling the yard, north of Fiftieth Street, which necessitated the use of two of the main tracks for that purpose. Consequently, the entrance to the terminal really consisted of but two main tracks for the accommodation of the traffic pouring to and from a four-track line. To increase the congestion, one of these tracks, assigned to drilling service, had also to be used for the storage of steam locomotives at rush hours of the day.

By the use of electricity, it became possible to depress the roadbed south of the low point at Fifty-sixth Street, so as to pass beneath the surface of Park Avenue on either side of the railroad, and thus permit the utilization of the full width of the avenue, 140 ft., without affecting

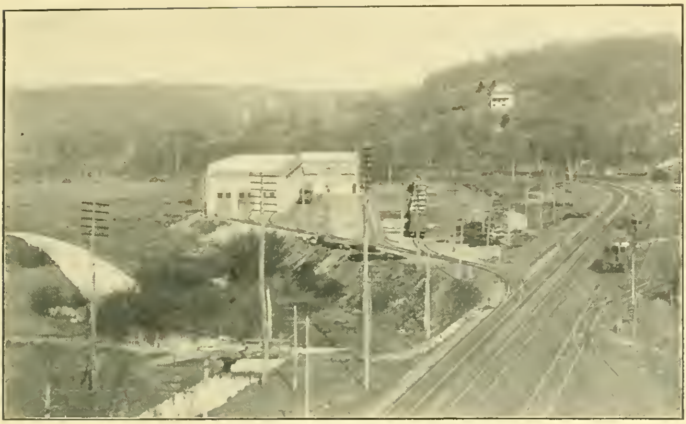


FIG. 1.—NORTH WHITE PLAINS TERMINAL.

FIG. 2.—HIGH BRIDGE TEMPORARY TERMINAL.

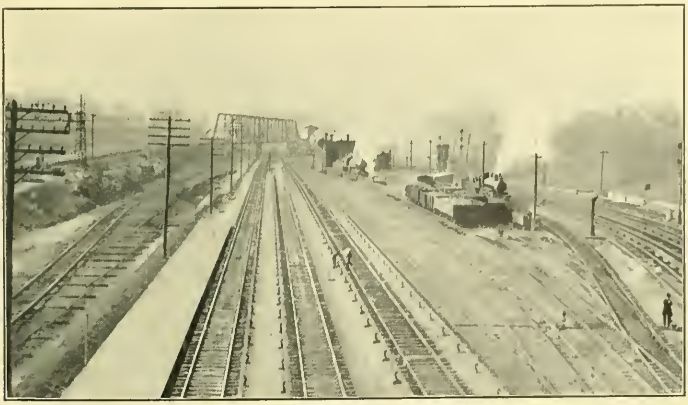


FIG. 3.—WAKEFIELD TEMPORARY TERMINAL.

its use by the public. This gave space for ten instead of four tracks from Fifty-sixth to Fiftieth Streets, of which four are for a legitimate main-line entrance to the enlarged upper yard, two are for drilling the yard, and two on each side, or four in all, are for ingress and egress of the lower-level suburban station. The upper level for through trains will have stub tracks, while the lower level will have a double-track loop at the south end near Forty-third Street.

The depression also admits of the extension of Park Avenue, for its full width, south from Fiftieth to Forty-fifth Streets over the tracks of the yard, and the connection by east and west viaducts of the ends of streets from Forty-fifth to Fifty-sixth Streets, inclusive, now separated by the terminal.

To the 23 acres in the old terminal has been added by purchase 17 acres, making a total area of 40 acres. With the 24 acres obtained by excavating for the suburban station, there will be a total area in the new terminal, when completed, of more than 64 acres. This is equal to an increase over the present space of 178 per cent.

These radical changes make necessary the tearing down of the old station and train-shed, originally built in 1871 and enlarged in 1898 and 1900; and the substitution of a much larger and handsomer structure, suited to the new motive power and more adequate for the proper handling of a rapidly increasing traffic.

It should here be added that electricity brings with it an unexpected boom in the permissible use of overhead spaces termed "air rights," that is denied with steam traction. A vast area in the heart of the greatest city on the continent is thus reclaimed for use as desired for various revenue-producing purposes. In time, this feature will add very largely to the company's assets.

An idea of the difficulties of construction, due to the nature of the underlying material—solid rock—and the necessity of subordinating all efforts to the safe and uninterrupted movement of an exacting and constantly increasing train service, is illustrated in the accompanying photographs.

The magnitude of the new terminal, which has thus to be built while trains and passengers pour in and out, is evident from the quantities of material involved. After being loaded on cars, 3 000 000 cu. yd. of rock and earth are dispatched, at times when the passenger service will permit, to the Hudson Division, for building additional main tracks. In the construction of retaining walls, suburban stations, viaducts, subways, and tunnels, 100 000 tons of steel and 260 000 cu. yd. of concrete are used, in addition to numerous other materials.

The final result will be an electrically operated station and yard with quadruple the capacity of the old one, and with many appurtenances for usefulness and profit, that are additional to the parent purpose of a railroad terminal.

Bronx Improvement.—At Mott Haven Junction, in the Borough of the Bronx, 5.3 miles from the Grand Central Terminal, two four-track lines, making eight tracks in all, merge into the single four-track stem that leads to the terminal. With increased frequency of train service, the grade intersections at such an important junction are inadmissible on the grounds of safety and non-delay to traffic. Therefore, plans have been adopted and work commenced on the raising and lowering of tracks by means of viaducts and tunnels, so as to effect trailing junctions free from grade crossings.

The points of junction are to be moved south about ¾ mile, to the vicinity of the Harlem River; and, near the present connection, on One Hundred and Forty-ninth Street, a new large overhead station is to be built, with eight main tracks. This will permit the abandonment of the old station at One Hundred and Thirty-eighth Street, and remove another of the causes for congestion on the four-track entrance to the Grand Central Terminal. Moreover, this new station will serve the rapidly growing population in the Bronx, which is fast approaching the half-million mark.

Elimination of Grade Crossings.—At the time of the decision to proceed with electric zone improvements, there were, within that territory, forty-four street and highway grade crossings, the abolition of which was deemed precedent to the commencement of electric operation. Of these, one-half were located within the city limits of New York, and these, by agreement with the City, have since been carried over the tracks. None of the remainder has yet been completed, owing to the delay of the State authorities to make effective the provisions of the statute governing grade crossings, and also owing to difficulties in acquiring the necessary additional right of way. However, due to the energetic action of the new Public Service Commission, decisions on many of the crossings have been reached; and the remainder are expected soon. The majority of these eliminations require either

PLATE XV.
PAPERS, AM. SOC. C. E.
FEBRUARY, 1908.
WILGUS ON
ELECTRIFICATION OF SUBURBAN ZONE
OF N. Y. C. & H. R. R. R.

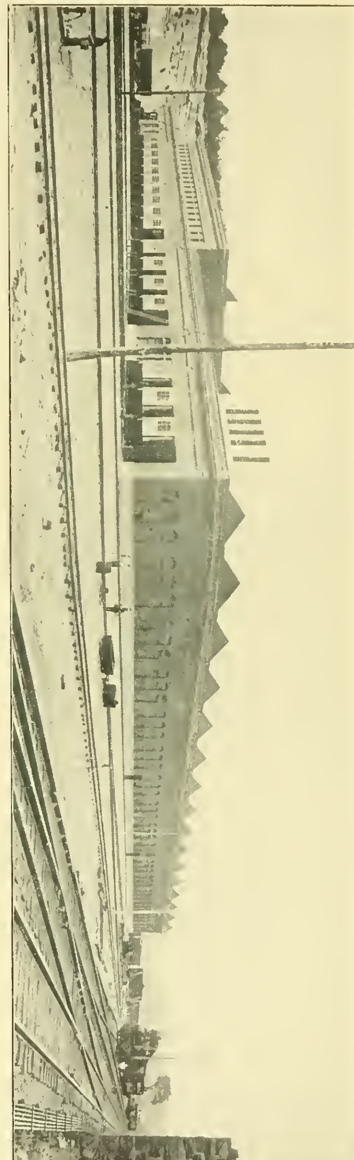


FIG. 1.—HARMON SHOPS

FIG. 2—GRAND CENTRAL YARD, LOOKING SOUTH FROM FIFTIETH STREET.

changes of the line of the railroad for considerable distances, as for instance at Mount Vernon and White Plains, or the lifting of the grade of the tracks so that the streets may pass under, as at Yonkers and Tarrytown.

Local Improvements.—The elimination of grade crossings north of New York City, and the growth of business, make obligatory many extensive and costly local improvements, with new passenger and freight stations and yards. The more important ones are at Yonkers, Hastings, Tarrytown, Ossining, and Harmon, on the Hudson Division; and at Mount Vernon, Bronxville, Tuckahoe, and White Plains, on the Harlem Division. Features of the design of these new stations are the avoidance, by means of subways and overhead bridges, of all grade crossings of tracks by passengers; and the placing of the tops of the local platforms on a level with the car floor.

Four-Tracking and Loops.—The anticipated increase in frequency of train service with electric traction, and the urgent necessity of removing causes of congestion in this important entrance to New York City, make mandatory the construction of additional main tracks, so that there will be separate tracks in each direction for high- and slow-speed service; and, where possible, additional tracks for the exclusive movement of freight.

In line with this policy, new main tracks are under construction within the suburban zone, in conjunction with the elimination of grade crossings and improvement of local facilities. The four tracks on the Harlem Division are being extended from Woodlawn Junction to North White Plains, with long middle sidings at frequent intervals, for the passage of passenger trains around freights. The double and triple main tracks on the Hudson Division, as far out as Harmon, are being increased to four, and, at some places, as for instance between Spuyten Duyvil and Yonkers, two additional tracks have been provided for the exclusive use of freight trains. As on the Harlem Division, middle tracks are being built, where needed, for keeping freight trains out of the way of the passenger service.

At Harmon and North White Plains, loops are to be built, for the turning of suburban trains without crossing the express traffic at grade. It will be noted that, with loops at all three termini and the freedom from grade crossings at Mott Haven Junction, opportunity is given for a constant flow of traffic with an absence of the usual obstructions that cause congestion.

Increased Capacity of Entrance to Grand Central Terminal.—From the fact that two four-track lines feed into a single four-track stem from Mott Haven Junction to the terminal, the question naturally arises as to what solution the future holds for this restriction on growth of traffic. The present plans of the terminal provide for a future four-track cross-town tunnel connection with the West Side line of the Company, over which the Hudson Division can then enter the terminal without burdening the Harlem Division tracks. This, when built, will afford to the terminal an eight-track entrance connected with both train levels.

Improvements in Alignment and Grades.—In conjunction with these radical changes in the physical condition of the property, it has been considered wise to make at the same time other desirable changes that could not be accomplished later without undue extra cost. At many places, on both divisions, alignment and grades have had careful study, and alterations have been approved which will result in material saving in rise and fall, and in curvature. Many have been completed, and others have been deferred, awaiting the acquisition of right of way and the settlement of legal questions. Among those still in embryo is the improvement between Croton and Peekskill, more than 8 miles in length, which when completed will admit of a still further extension of electric operation.

The advantages to be gained by the principal changes of alignment are as follows:

	Saving in Distance.	Saving in Curvature.
Marble Hill cut-off, including Spuyten Duyvil Tunnel cut-off	3 944 ft.	137°
Croton to Peekskill	4 338 "	333°
Spuyten Duyvil to Mt. St. Vincent	9 "	24°
Irvington cut-off	50 "	65°
Totals	8 341 "	559°

Signals and Interlocking.—Under the old order of affairs, traffic on the Hudson Division from the north ran right-handed to Spuyten Duyvil, where it was transposed to left-handed operation so as to harmonize with the left-handed practice on the Harlem Division. The design of the new Grand Central Terminal and a possible future con-

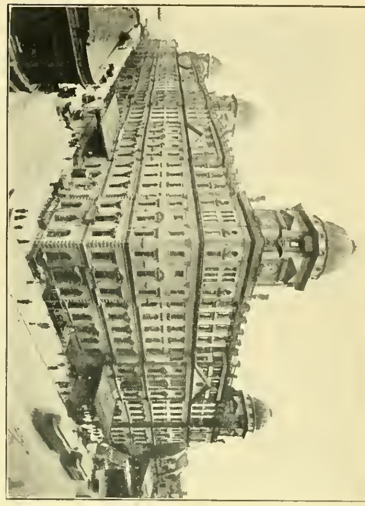


FIG. 1.—PRESENT GRAND CENTRAL STATION.

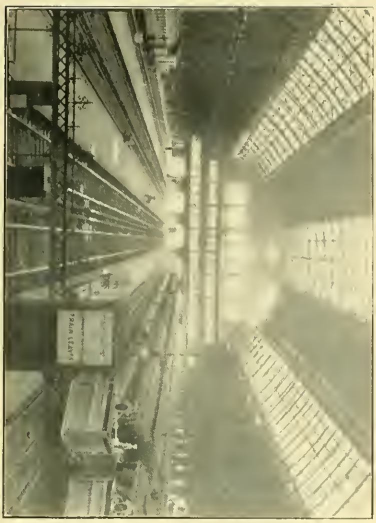


FIG. 2.—PRESENT GRAND CENTRAL TRAIN-SHED.

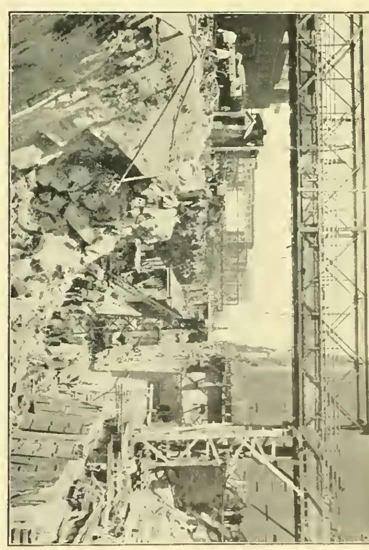


FIG. 3.—EXCAVATION IN PROGRESS, GRAND CENTRAL TERMINAL.

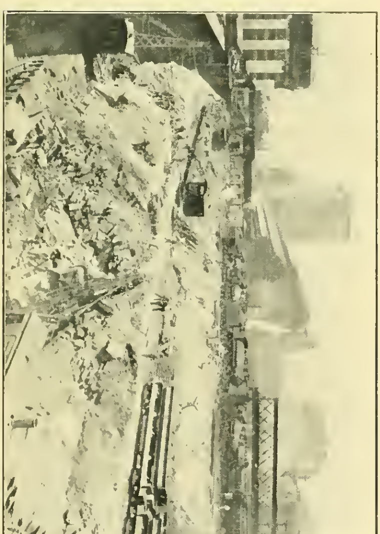


FIG. 4.—EXCAVATION IN PROGRESS, GRAND CENTRAL TERMINAL.

nection with the city subway system, as well as the desirability of avoiding the grade crossing at Spuyten Duyvil, led to the decision to make the right-handed system of operation uniform throughout the suburban zone. In considering the effect of this reversal of traffic on the existing signals and interlocking plants, it was also realized that the controlled-manual system in use on a large portion of the territory was insufficiently elastic for the quick handling of a frequent electric train service on four or more tracks. Accompanying these traffic reasons for radical changes in the old signals and interlockings was the equally important fact that the use of track rails for return propulsion current to the power stations completely deranged the signal circuits. Then, too, the many additions and changes to tracks made imperative the abandonment of the larger part of the old plants.

All these causes led to the adoption of new electric automatic signals and electric interlocking plants for the entire zone, the predominant feature of which is the reactance bond, which permits the free passage of propulsion current through the track rails, but, where desired, stops the passage of the alternating signal current circuit.

Fences.—One of the worst evils with which American railroads have to contend is trespass. Right of way and tracks are considered public highways, and the petty courts refuse or neglect to impose adequate punishment on those who thus risk their lives in dangerous places. This freedom of use of the railroad's property also leads to thieving which, in the aggregate, causes large losses to the company. The change to electric traction by no means minimizes these evils. More frequent trains, and the presence of electricity, increase the risks, while copper cables and bonds attract the thief. To guard against these increased dangers, the entire electric zone is to be enclosed with man- and boy-proof fences. The portion within the settled districts consists of iron pickets and concrete posts of a pleasing design.

Chronology.—Following the placing of orders in the fall of 1903, work was pushed energetically on all items of construction required for the operation of the initial electric zone south of Wakefield and High Bridge. It should be borne in mind that the larger part of the work had to be performed on or about tracks congested with traffic, which entailed danger to employees, delay to many parts of the work, and expense. It is a pleasure to record, however, that not an accident occurred to regular train service, nor, with a few minor exceptions, any delay to traffic, due to construction. Such accidents and delays as did occur were from other causes.

The following dates mark the progress of the electrical features:

Initial informal test of first electric locomotive	October 27th, 1904.
First formal test of Electric locomotive	November 12th, 1904.
Port Morris Power Station:	
Commencement	May 15th, 1904.
First current	July 1st, 1906.
Transmission Lines:	
Commencement	February 17th, 1905.
Ready for service	September 30th, 1906.
Sub-stations:	
Commencement	July 6th, 1905.
Ready for service	September 30th, 1906.
Working Conductors:	
Commencement	January 2d, 1906.
Ready for service	December 11th, 1906.
Electrical Equipment:	
First operated in New York City	July 20th, 1906.
First train into Grand Central Terminal	September 30th, 1906.
Electrical Operation:	
First schedule multiple-unit train	December 11th, 1906.
First schedule electric locomotive	February 13th, 1907.
First regular shop train	April 14th, 1907.
Completion of change of motive power:	
Schedule trains	July 1st, 1907.
Reversal of traffic	August 25th, 1907.

Because of the burdensome conditions of traffic, and complicated changes in the signal and interlocking systems, about 6 months were thus consumed in making the change of motive power complete, after the first schedule train was operated.

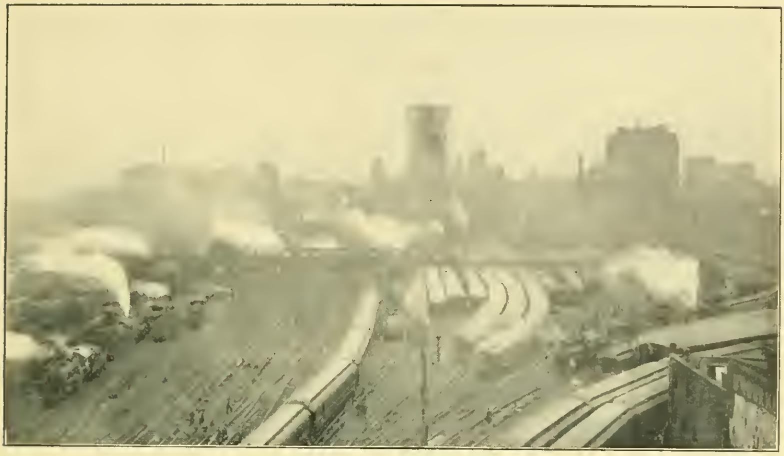


FIG. 1.—BEFORE.

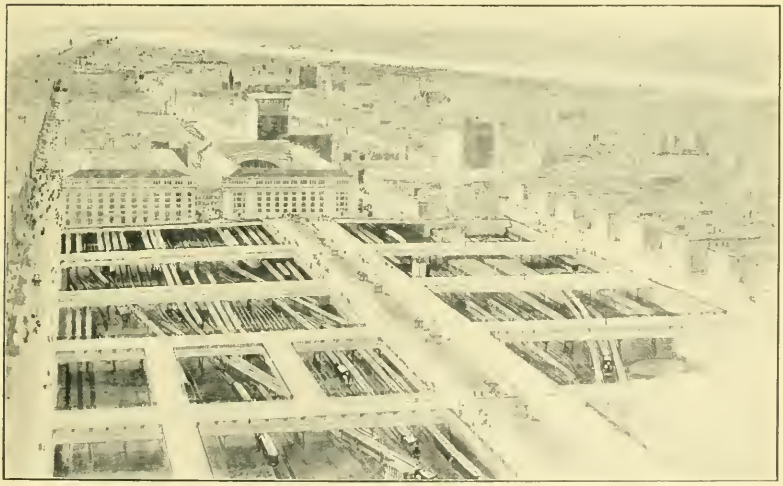


FIG. 2.—AFTER.

CONTRAST BETWEEN THE SMOKE CONDITIONS, AS THEY EXISTED AT THE GRAND CENTRAL TERMINAL IN 1906, AND THE ABSENCE OF SMOKE IN THE NEW TERMINAL, DUE TO THE USE OF ELECTRICITY.

Initial Zone Operation.—As previously stated, the company was forced to confine temporarily the change of motive power to the operation of the suburban zone terminating at High Bridge, 7 miles out; and at Wakefield, 13 miles from the terminal. This postpones for two or three years the extension of electrical service to the northerly termini of the suburban zone. In the meantime, the power on through trains is changed at the temporary termini. At the same points, multiple-unit trains north-bound have steam locomotives attached and thence proceed as non-electric trains; and south-bound the steam locomotives are detached and the trains continue by electricity without locomotives. The average time required for making the changes, including that lost in slowing down and regaining speed is as follows:

Through trains with locomotives........................	4½ min.
Multiple-unit trains, north-bound........................	3 "
Multiple-unit trains, south-bound........................	2½ "

On the Hudson Division this delay has been largely compensated by shortening the line at Marble Hill and the elimination of grade track crossing at Spuyten Duyvil.

Results.

Expectations from Electrification.—Now that the change of motive power in the initial electric zone has been completed for sufficient time to gain at least a preliminary idea of the results, the question naturally arises, with what success has the change met expectations?

It has already been explained that the principal reasons for undertaking the work were twofold:

(1).—Demand of the public for the abolition of the nuisances incident to the use of steam locomotives south of the Harlem River; and

(2).—Need for increased capacity of the terminal, by the elimination of a large proportion of the switching movements required with steam locomotive practice; and relief to the main line entrance to the terminal by reducing its use for haulage of dead locomotives and cars to Mott Haven.

As secondary considerations there were:

(3).—The possibility of sufficient economy in operation at least to offset largely the additional fixed charges on the cost of the electrical installation; and

(4).—Opportunities for an ultimate large increase in traffic and corresponding growth of revenue to justify the expenditure for all improvements within the suburban zone.

What do the observations made thus far disclose?

The first two expectations have been completely realized.

Park Avenue Tunnel.—The atmospheric conditions in the Park Avenue Tunnel show marked improvement, even with the presence of the remaining New Haven Company's steam service.

Increased Terminal Capacity.—The effect on the operating efficiency of the terminal has been very gratifying, the increased capacity being estimated at one-third. There has also been a large reduction in the number of shop or "dead" trains to and from Mott Haven.

Reduced Cost of Operation.—The results, as regards the third expectation, have been most surprising. The operation, for a considerable period, of steam and electric equipment side by side has afforded an unexampled opportunity for a true comparison of costs of operation. Until now, data on this subject have been based on theory, ignoring many of the indeterminate features of actual operation that have such a weighty effect on costs. For instance, among the variables entering into an analysis of this character are:

(a).—Cost and quantity of coal and water at the power station, and on the steam locomotive tender;

(b).—Relation of ton-mileage of the motive power to total ton-mileage, including motive power and cars;

(c).—Frequency and volume of traffic;

(d).—Mechanical and electrical design of motive power as affecting repairs, and hours available for active service;

(e).—Fixed charges, depreciation, and maintenance on all items of both kinds of service, that have a bearing on comparative results, including land, structures, and equipment.

In other words, to obtain a true comparison, observations must be made under like conditions in a known service.

Fig. 1.—Proposed Bronx Station.

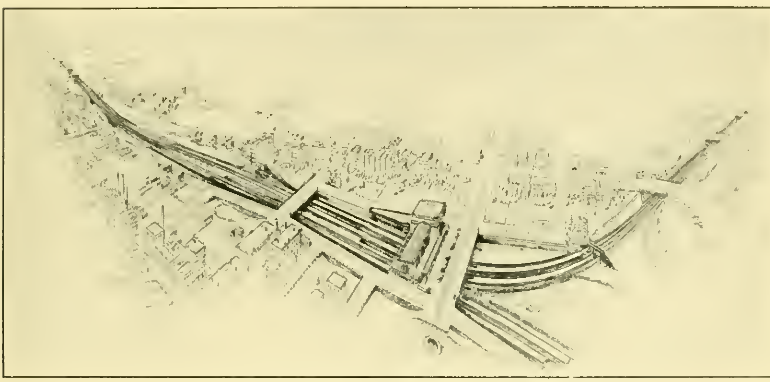
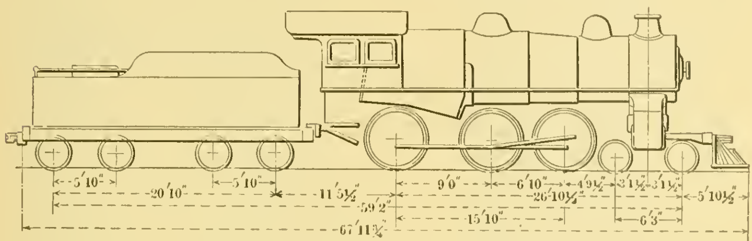


Fig. 2.—Orthographic View of Bronx Improvement.

Fig. 3.—Typical Grade Crossing Elimination, with Overhead Station and Tracks Beneath (University Heights).

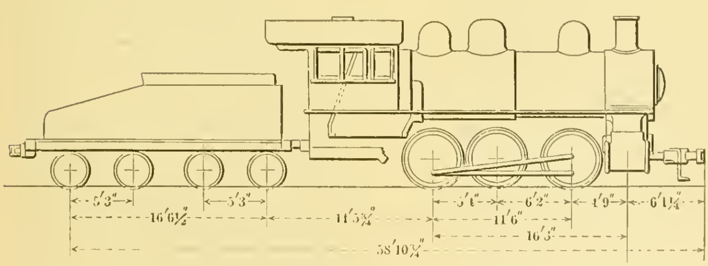
With this object in view, a typical steam switching locomotive, engaged in terminal service, and a steam passenger locomotive, assigned to road service, were each selected for observation in the same class of traffic with electric locomotives. The terminal service embraced switch-

STEAM LOCOMOTIVES USED IN COMPARATIVE TESTS



Weight on drivers, working order .... 156 000 lb.
Weight on truck, working order........... 16 500 lb.
Weight, total of engine.................. 191 500 lb.
Weight of tender, loaded................. 118 000 lb.

STEAM LOCOMOTIVE USED IN
ROAD TESTS.
CLASS-F-2-*d*.
(No.1978.)



Weight on drivers, working order .... 152 500 lb.
Weight, total of engine.................. 152 500 lb.
Weight of tender, loaded
4500-gal. tank .... 89 500 lb.
5100-gal. tank .... 91 500 lb.

STEAM LOCOMOTIVE USED IN
SWITCHING AND HAULING TESTS.
CLASS B-10.
Fig. 3.

ing at the Grand Central yard, and hauling dead cars to and from Mott Haven storage yard, a distance of 6 miles. The road service comprised the hauling of schedule trains by the electric locomotive between the Grand Central Terminal and Wakefield, 12½ miles; and the same trains by steam between Wakefield and North White Plains, 11½ miles.

Observers constantly rode the locomotives for the period of the tests, namely, September 12th to 27th, 1907, in terminal service, and October 4th to 18th, 1907, in road service. Cyclometers and wattmeters registered actual distances, speeds, and current consumption. Record was also kept of the number of cars switched and hauled, and the proportion of time each day engaged in actual service, awaiting duty, and laid up for inspection and repairs.

The coal used contained 14 000 B. t. u. per lb., and the cost, per ton of 2 240 lb., was:

Steam locomotive in terminal service (anthracite)....	$5.00 per ton.
Steam locomotive in road service (bituminous)......	3.50 " "
Port Morris power station (bituminous)............	3.05 " "

Water, per 1 000 gal., cost as follows:

Terminal service and at power station..................	13½ cents.
Road service..	5 "

The cost of electric current, when the power station designed load is attained, is taken at 2.6 cents per kw-hr., delivered at the contact shoes of the equipment, and includes all operating and maintenance costs, interest on the electrical investment required to produce and deliver the current, depreciation, taxes, insurance, and transmission losses. The details of this cost are:

Items.	Operating costs.	Fixed charges.	Total.
Power station......................	$0.58	$0.44	$1.02
Transmission losses................	0.19	0.15	0.34
Distributing system and sub-stations..........................	0.32	0.92	1.24
Totals.....	$1.09	$1.51	$2.60

Locomotive wages are practically identical for each class of service.

Table 1 shows the details of locomotive repairs, maintenance, and fixed charges for each class of service, from which it will be noted that, although the fixed charges and depreciation of the electric loco-

Fig. 1.—Typical Grade Crossing Elimination, with Overhead Station and Tracks Beneath (High Bridge).

Fig. 2.—Marble Hill Cut-off, by which Seven Grade Crossings Have Been Abolished.

Fig. 3.—Yonkers Grade Crossing Work in Progress.

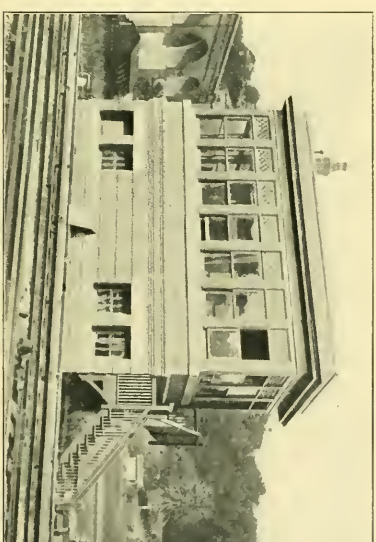


Fig. 4.—Typical Signal Tower.

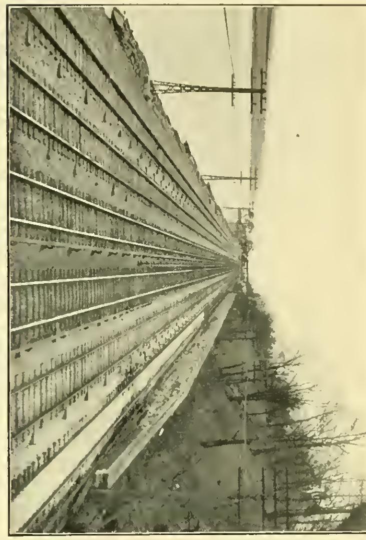


Fig. 1.—Six-Tracking, Hudson Division.

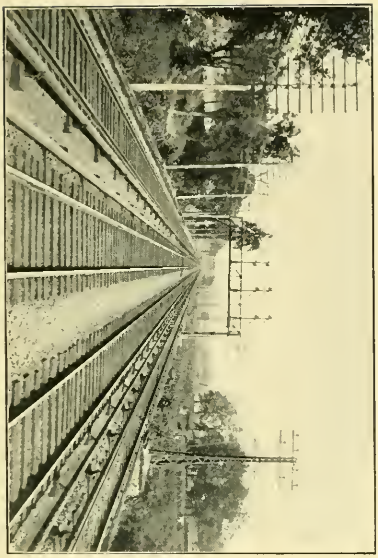


Fig. 2.—Four-Tracking, Harlem Division.

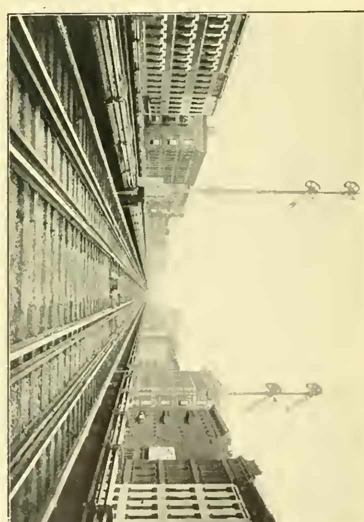


Fig. 3.—Typical Signals, Park Avenue Viaduct.

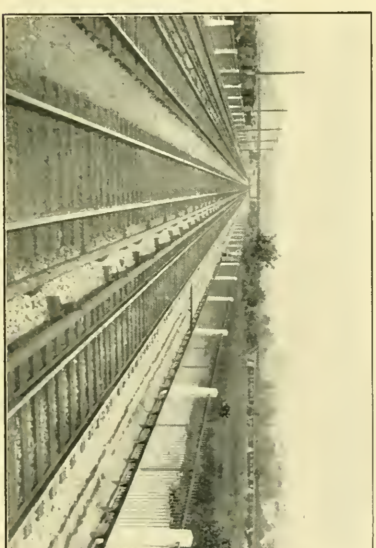


Fig. 4.—Typical Fencing, New York City

TABLE 1.—Comparison of Costs per Day of Available Service of Steam and Electric Locomotives for Interest, Depreciation, Repairs, Inspection, and Handling.

Subject.	Steam. Description.	Amount per annum.		Per day.	Electric. Description.	Amount per annum.		Per day.
Interest	4¼% on $15 000		$637.50		4¼% on $30 000		$1 275	
Depreciation	5% on $15 000		750.00		5% on $30 000		1 500	
Repairs	General at West Albany	$1 170			General at Harmon	$468		
	Running at Mott Haven	414			Running at High Bridge and Wakefield.	166		
	Trips to shops, 300 miles	168			Trips to shops, 60 miles	34		
	Use of shops	90	1 842.00		Use of shops	36	704	
	Total for 335 days available for service.		$3 229.50	$9.64	Total for 350 days available for service.		$3 479	$9.94
Handling and inspection, including fixed charges and maintenance of land and structures	Mott Haven engine-house plant, 365 days		1 231.00	3.37	High Bridge and Wakefield inspection sheds, 365 days		200	0.55
Total			$4 460.50	$13.01			$3 679	$10.49

The saving in favor of the electric locomotive, therefore, is $2.52 per day, equal to 19 per cent.

motive are higher than those of the steam, owing to the greater first cost, the net result is in favor of the electric locomotive, due to lower costs for repairs and maintenance. These results are based on actual observations of the steam locomotive covering a period of several years; and of the electric locomotive for two years on the experimental track near Schenectady and one year in the New York zone. The reasons for the lower cost of repairs on the electric machine are the simplicity of construction and the minimum number of mechanical parts. It is also worthy of comment that the electric locomotive costs very much less per day for repairs and maintenance, due to lower expenses for land and structures, and fewer days out of service. For instance, the fixed charges and cost of maintenance and operation of the extensive steam engine plant on costly land, are comparable with the simple inspection-shed charges of the electric locomotive.

The Schenectady experiments indicated that the cost of repairs of the electric locomotive of this type is about two-fifths of that of the steam locomotive of a corresponding age and capacity.

The results of these observations are shown in detail in Table 2, Plate XXI, and are summarized in Table 3. They show that, under the stated conditions, the electric locomotive has the following advantages over its steam rival:

19% saving in locomotive repairs and fixed charges.
18% saving in dead time for repairs and inspection.
25% greater daily ton-mileage.
6% saving in locomotive ton-mileage in hauling service.
11% saving in locomotive ton-mileage in switching service.
16% saving in locomotive ton-mileage in road service.
12% net saving in cost in hauling service.
21% net saving in cost in switching service.
27% net saving in cost in road service.

Even better results may be expected during winter months, when steam locomotives are subjected to many conditions that cause additional expenses not incident to the electric locomotive.

Reduced Cost of Grand Central Terminal Operation.—Owing to the partial use of steam switching locomotives, and the presence of the New Haven Company's steam road locomotives at the terminal, the full benefits of change of motive power have not yet been secured.

Items.	Switching.	Hauling.	Total.
Pounds of coal per ton-mile of cars....	3.35	0.46	0.97
Gallons of water per ton mile of cars...	3 33	0.47	0.69
Watt-hours per ton mile of cars........	2.54	44.8	39.9

TABLE 2.—Comparative Tests of Steam and Electric Locomotives in Switching and Hauling Service: Grand Central Terminal, N. Y. Runs made between September 12th and 27th, 1907.

Switching, Grand Central Station.

No. of test.	Kind of Power.	Hours: Busy.	Idle: Waiting.	Idle: Dead.	Idle: Total.	Miles run.	Cars handled.	Ton-miles: Total.	Ton-miles: Cars only.	Ton-miles: Per busy hour.	Coal: Tons.	Coal: Pounds, ton-miles.	Water: Gallons.	Water: Gallons, ton-miles.	Current: Kilowatt-hours.	Current: Watt-hours, ton miles.	Total costs: Supplies.	Wages.	Fixed charges and repairs.	Total.	Unit costs, Ton-mile: Total tons.	Car tons.	Car units.
1	Steam....	1.69	6.55	12.45	19.10	11 06	99	1 952	540	820	1.23	1.42	3 066	1.08	......	......	6.56	6.96	11.27	24.76	0.02905	0.0460	0.83
2	Electric.	7.66	15.27	8.12	10.87	45 29	200	9 542	5 088	689	......	......	......	......	930	96	24 33	14.22	10.49	40.16	0.00515	0.00065	0.16
3	Steam..	1.80	3.19	6.91	10.06	13.53	22	6 004	517	288	2 1	0.07	5 590	0.80	......	......	11.55	4.18	6.41	21.08	0.00314	0.042	1.50
4	Electric	0.64	1.92	2.95	4.17	8 59	18	943	612	1 183	......	......	......	......	108	187	3.34	1.41	2.08	6.81	0.00730	0.011	0.86
5	Steam..	0.79	1.92	2.81	5 68	5.17	40	1 141	523	663	0.34	1.20	1 270	1.11	......	......	8.40	2.18	3.96	8.79	0.00770	0.0156	0.82
6	Electric..	1.04	4.08	6.81	9.89	5.60	118	861	120	118	......	......	......	......	91	140	2.68	2.62	4.60	10.60	0.01620	0.0980	0.97
7	Steam	2.65	6.31	4 54	11.06	13.61	82	4 130	2 562	969	1.34	1.00	4 370	1.06	......	......	9.84	7.00	7.44	24.88	0.00903	0.00072	0.19
8	Electric.	1.44	5.48	4.84	10.37	7.73	13	1 349	316	310	......	......	......	......	140	135	3.60	6.81	5.12	14.29	0.01961	0.0453	0.48
9	Steam...	1.12	2.52	6.54	9 16	6.88	60	1 114	482	430	0.64	1.17	1 340	1.20	......	......	3.14	3.49	6.37	12 26	0.01020	0.0253	0.61
10	Electric.	0.82	2.68	5.22	7.90	4 97	29	784	269	328	......	......	......	......	123	180	3.19	3.37	3.94	10.40	0.01490	0.0623	0.80
11	Steam	1.84	4 06	6 66	12.92	8.81	50	1 730	690	507	1.20	1.60	2 120	1.24	......	......	6.37	4.31	7.73	18.41	0.01065	0.0271	0.31
12	Electric	0 16	0.30	0.30	0.50	0.72	10	160	100	050	......	......	......	......	94	142	0.53	0.31	0.28	1.22	0.00724	0.0122	0 12
13	Steam...	6.92	10 19	2.15	12 98	27.53	187	7 298	2 798	465	4.67	1.41	6 800	1.14	......	......	24.18	11.00	9.89	45.13	0.00630	0.0161	0.34
14	Electric.	2.85	5 63	1.06	6.69	12 84	46	1 064	730	814	......	......	......	......	268	187	0.99	5.31	2.91	16 21	0.00825	0.0519	0.55
15	Steam...	1 43	2 82	5 76	8 58	6.60	41	1 427	810	450	0.66	1.28	2 140	1.50	......	......	4.78	5.16	6.49	18.53	0.00635	0.0219	0.63
16	Electric.	1 86	4.07	0.71	4.78	9 98	48	1 646	905	482	......	......	......	......	287	169	6.70	3.06	2.01	13.57	0.00785	0.0150	0.28
17	Steam...	1 73	2 60	6.46	7.96	7 58	25	1 160	244	147	0.75	1.40	2 080	1.78	......	......	4 07	3.06	6.26	12.39	0.01045	0.0500	0.54
18	Electric	1.05	3 78	4 66	7.63	4.77	19	777	324	309	......	......	......	......	127	163	3.32	3 12	3.79	10. 3	0.01330	0.0316	0.54
19	Steam...	1 61	0.01	4 48	10.49	2.80	46	1 711	674	856	1.00	1.48	2 640	1.54	......	......	5.87	0.54	6.56	17.77	0.01030	0.0010	0.39
20	Electric.	1.67	2.92	0.00	4.58	3.31	63	1 457	662	410	......	......	......	......	178	122	4.54	3.63	9.62	11.37	0.00778	0.0171	0 34
21	Steam...	1.17	3 05	6.91	9.96	7.61	81	1 449	535	457	1.04	1.01	2 480	1.27	......	......	6.59	3.61	6 04	15 14	0.01045	0.0283	0.40
22	Electric.	2 61	0 07	1 56	2 63	12.77	57	2 218	927	373	......	......	......	......	284	128	7.40	6 68	4.65	18.81	0.00850	0.0301	0.34
23	Steam..	2 50	5 92	4 18	8.05	16.40	100	3 545	1 665	674	1.97	1.30	5 830	1.50	......	......	10.70	6.66	5.73	23 81	0.00643	0.0135	0.29
24	Electric.	1.99	6.98	0.97	6.85	11.50	91	1 596	425	216	......	......	......	......	200	131	5.92	5 96	3.86	14.34	0.00340	0.0330	0.83
25	Steam	1.00	4 06	11 81	15.94	6 50	63	1 010	730	730	0.56	0.98	2 310	1.83	......	......	3.73	6.83	9.13	16.65	0.07106	0.0229	0.51
26	Electric.	1 94	4 31	0.06	4.07	9 49	50	1 550	547	333	......	......	......	......	225	145	6.67	4.11	3.02	13.50	0.00637	0.0201	0.36
27	Steam...	0.40	2 40	5.06	11 65	2.24	6	407	133	333	0.40	2.21	1 060	2.56	......	......	2.17	2.87	6.27	10.01	0.02980	0.0622	1.80
28	Electric	5.28	6.78	1.06	7 81	18.93	00	3 300	1 657	663	......	......	......	......	409	119	10.47	6.69	4.04	21.00	0.00650	0.0154	0.87
Total	Steam....	26 21	69.54	90.35	169.00	160.61	734	35 581	13 617	......	16.96	......	43 650	......	......	......	101.60	37 27	96.06	265.14	......	......	......
Total	Electric.	28.10	67.10	33 54	100.69	155.75	743	27 763	12 600	......	......	......	......	......	3 276	......	65.10	67 28	50.32	211.76	......	......	......
Average.	Steam....	1.83	4.83	6.70	11.16	10.01	53	2 540	916	501	1.37	1.19	3 110	1.21	......	......	7.37	4.63	6.07	19.24	0.00716	0.02101	0.652
Average.	Electric.	2 01	4.76	2.40	7.19	11.13	53	1 980	914	445	......	......	......	......	840	121	6.80	4.60	4.00	15.15	0.00762	0.01653	0.285

Hauling—G. C. S. to and from Mott Haven Yard.

No. of test.	Kind of Power.	Hours: Busy.	Idle: Waiting.	Idle: Dead.	Idle: Total.	Miles run.	Cars handled.	Ton-miles: Total.	Ton-miles: Cars only.	Ton-miles: Per busy hour.	Coal: Tons.	Coal: Pounds, ton-miles.	Water: Gallons.	Water: Gallons, ton-miles.	Current: Kilowatt-hours.	Current: Watt-hours, ton-miles.	Total costs: Supplies.	Wages.	Fixed charges and repairs.	Total.	Unit costs, Ton-mile: Total tons.	Car tons.	Car units.	No. stops en route.	Average speed.	Maximum speed.
1	Steam....	3.27	0 0	-0.0	0.0	27.80	69	19 220	14 675	4 580	3.00	0.53	5 000	0.26	......	......	15.94	2.14	1.74	19.58	0.00103	0.00135	0.83	0.2	11.8	30
2	Electric.	0.0	0.0	0.0	0.0	0.0	0	0	0	0	......	......	......	......	0	......	0.0	0.0	0.0	0 0	......	......	......	......	......	......
3	Steam..	2.88	2.18	7.11	9.99	66.76	46	15 675	11 637	4 000	1.16	0.17	9 100	0.20	......	......	6.51	4.31	0.63	17 12	0.00112	0.00160	0.89	1.0	11.7	30
4	Electric	3.88	3.38	13 08	16.46	44.47	40	13 719	9 494	2 480	......	......	......	......	479	34 4	13.46	5.74	8.48	20.63	0.00194	0.00280	0.67	3.7	11.9	40
5	Steam..	6.01	3.06	0.91	11.97	84.04	80	94 523	16 847	2 800	3.43	0.31	6 820	0.23	......	......	18 59	6 97	0 73	34.51	0.00141	0.00306	0.56	1.6	9.4	25
6	Electric..	4.00	2.60	6.91	9.89	60.22	66	19 799	14 404	3 610	......	......	......	......	683	34.6	13.06	4 71	5.94	28.79	0.00145	0.00199	0.83	1.7	14.1	40
7	Steam	4.97	1.91	3.42	5.38	50.60	70	28 435	18 750	3 780	2.63	0.83	6 410	0.23	......	......	15.63	6.75	6.57	28 85	0.00108	0.00142	0.85	0.9	11.8	35
8	Electric.	5.78	1.48	5.08	6.68	67.58	86	31 014	14 400	2 530	......	......	......	......	780	33.3	19.16	6.79	5 37	30.82	0.00168	0.00209	0.87	2.6	11.8	45
9	Steam...	3.54	3.32	6 60	11.18	68.46	37	14 684	10 670	4 200	1.92	0.33	4 400	0.30	......	......	10.54	4.66	7.41	22.66	0.00164	0.00212	0.01	0.5	13 2	40
10	Electric.	4.20	1.87	9.15	11.03	47.67	60	16 944	15 892	3 700	......	......	......	......	760	41.5	10.63	5.85	6.85	23.38	0.00178	0.00204	0 50	2.0	10.4	35
11	Steam	1.09	1.77	5.00	7.82	32.70	37	12 204	9 079	4 730	2.90	0.39	8 940	0.31	......	......	19.10	2.90	5.38	30.04	0.00138	0.00224	0.55	0.8	14.1	35
12	Electric	6.70	6.02	7.03	10.60	85.11	100	33 600	29 974	3 840	......	......	......	......	1 160	35.9	31.05	11.00	10.91	52.26	0.00157	0.00234	0.49	1.6	16.5	45
13	Steam...	2.74	3.33	0.08	5.01	33.85	48	14 281	10 250	3 750	3.63	0.85	4 040	0.32	......	......	11.08	5 49	3.12	18.50	0.00130	0.00181	0.41	0.3	11.7	25
14	Electric.	6.58	4.65	1.77	6.42	95.84	116	33 635	26 676	5 050	......	......	......	......	1 222	34.8	21.97	8.97	6.66	47.08	0.00133	0.00170	0.41	1.8	11 0	43
15	Steam...	3.33	2.61	8.00	10.97	42.30	31	13 522	8 878	2 610	3.08	0.53	6 060	0.37	......	......	11.42	4.43	7.59	23.44	0.00174	0.00290	0.76	0 4	12.8	35
16	Electric.	9.78	5.77	1.84	7.61	109.28	144	44 341	35 547	8 450	......	......	......	......	1 543	34.0	40.28	10.59	7.50	58.18	0.00131	0.00174	0.40	1.8	11.6	44
17	Steam...	4.88	1.67	6.04	9.71	56.80	50	21 004	14 408	3 150	5.83	0.55	9 020	0.43	......	......	18.21	4.69	7.75	30.48	0.00145	0.00211	0.54	0.6	12.9	30
18	Electric	4.77	2.69	6.06	10.55	53.25	60	16 948	11 673	2 500	......	......	......	......	638	40.6	18.23	6.50	6.70	30.43	0.00180	0.00257	0.61	2.0	11.1	45
19	Steam...	5.01	2.54	4.41	7.95	44.70	53	21 450	15 914	4 040	3.41	0.36	8 280	0.39	......	......	18.49	5.27	6.45	30.21	0.00140	0.00180	0.67	2.1	11.3	......
20	Electric.	8.23	6.01	2.60	5.61	106.21	153	46 027	29 716	3 679	......	......	......	......	1 312	30.0	34.34	10.44	7 97	53.45	0.00129	0.00100	0.34	1.9	12 9	41
21	Steam...	5.84	1.34	7.99	9.33	50.41	46	15 190	10 620	8 060	2.73	0.40	6 850	0.42	......	......	14.94	4.08	6.97	25 69	0.00171	0.00289	0.58	1.4	10.3	33
22	Electric.	7.11	3.67	2.18	5.85	100.35	109	44 433	34 922	4 910	......	......	......	......	1 426	32.1	27.23	7.71	5.50	50.50	0.00114	0.00145	0.89	0.9	14.1	40
23	Steam..	5.12	2.09	2.31	6.35	56.83	41	14 749	10 879	2 330	2.06	0.31	5 860	0.40	......	......	11.31	6.12	4.58	21.02	0.00143	0.00205	0.51	2.1	11 8	......
24	Electric.	7.79	6.70	1.87	7.87	103.01	145	45 581	34 341	4 380	......	......	......	......	1 440	32.8	27.63	0.00	6.63	53.28	0.00121	0.00157	0.37	1.9	13.8	49
25	Steam	1.76	0.41	5.02	5.45	10.61	13	5 120	5 140	1 820	0.55	0.34	1 960	0.38	......	......	3.30	1.63	3.88	8.81	0.00172	0.00281	0.69	0.7	9.6	26
26	Electric.	10.07	5.36	1.84	7.02	112.73	143	43 204	39 421	3 840	......	......	......	......	1 805	50.8	40.68	10.17	7.47	68.63	0.00135	0.00184	0.41	2.9	11.8	43
27	Steam...	1.00	1.02	9.35	10.85	28.92	94	10 247	7 020	5 700	3.53	0.55	6 670	0.64	......	......	13.91	2.48	-8.54	23.05	0.00225	0.00326	0.90	0.4	11.2	23
28	Electric	6.25	3.42	1.26	4.68	111.81	142	46 974	36 831	4 410	......	......	......	......	1 408	29.9	36.67	7.70	6.60	50.08	0.00107	0.00189	0.85	1.4	18.5	41
Total	Steam....	46.42	25.18	83.10	107.27	531.31	636	228 010	161 089	......	35.50	......	70 870	......	......	......	182.31	67.28	83.50	322.00	......	......	......	......	......	......
Total	Electric.	89.76	50.12	61.27	117.39	1 020.48	1 323	425 119	326 100	......	......	......	......	......	14 459	......	377.36	102.92	90.54	571.82	......	......	......	......	......	......
Average.	Steam....	3.36	1 82	5.90	7.78	40.00	46	16 640	11 730	3 400	3.43	0.53	5 670	0.34	......	......	12.92	4.15	6.04	23.41	0.00149	0.00199	0.616	0.9	11.9	48
Average.	Electric.	6.41	4.01	4.68	8.89	78.87	96	30 370	23 310	3 540	......	......	......	......	1 030	33.9	26.93	7.36	6.46	40.80	0.00134	0.00175	0.482	2.9	13.8	43

Total for Both Classes of Service.

No. of test.	Kind of Power.	Busy: Hours.	Busy: Per cent.	Idle: Waiting.	Per cent.	Dead.	Per cent.	Total.	Per cent.	Ton-miles: Total.	Cars only.	Per busy hour.	Total costs: Supplies.	Wages.	Fixed charges and repairs.	Total.	Average cost, car ton-mile.
1	Steam....	4 00	20.5	6.53	27.7	12.45	51.8	19.10	79.5	21 172	16 216	3 110	23.80	3.10	13.01	44.91	0.00996
2	Electric.	7.63	31.6	15 25	60.2	3.12	18 0	16.27	68.2	9 542	5 098	569	24 38	14.28	10.49	40 16	0.00065
3	Steam..	4.68	19.6	5.60	23.1	14.06	58.4	19.32	80.5	22 569	12 084	2 680	17 87	8.40	13.97	39.27	0.00237
4	Electric	4.97	18.1	4.00	10.2	15.03	60.7	19.53	81.9	14 652	10 106	2 310	16 80	7.16	10.49	33.44	0.00281
5	Steam..	6.80	28.3	5.38	16.6	13.69	56.1	17.90	71.7	25 664	17 370	2 530	21.79	8.60	13.01	43.90	0.00250
6	Electric..	5.04	20.2	6.73	28.0	12.35	51.1	18.98	79.1	20 480	14 584	2 810	15.85	6.58	10.49	32.28	0.00269
7	Steam	7.62	31.7	8.42	35.1	7.06	33.2	16.36	68.3	30 565	21 315	2 800	25.17	13 55	13.01	51.58	0.00246
8	Electric.	7.17	30.0	6.01	28.5	0.89	41.2	16.84	70.0	33 903	14 810	2 070	23.88	11 80	10.49	44.61	0.00310
9	Steam...	3 05	13.3	4.64	20.2	15.60	64.5	20.34	84.7	15 800	11 150	3 050	13 96	8.17	13.01	34.55	0.00318
10	Electric.	5.02	20.9	4 55	19.0	14.45	60.1	18.98	79.1	18 978	16 161	3 290	23.07	9.22	10.49	42 76	0.00265
11	Steam	3 36	13.6	5.83	24.8	14.01	58.1	20.74	86.4	14 684	9 789	2 900	18.47	7.97	13.01	38.78	0.00307
12	Electric	6.60	28.6	0.92	11.4	7.23	30.1	17.16	71.6	33 160	23 474	3 290	21.60	11.67	10.49	53.48	0.00288
13	Steam...	8.71	36.2	12.46	51.9	2.83	11.9	16.99	63.8	31 604	13 037	1 500	34.10	14.59	13.01	65.77	0.00468
14	Electric.	11.00	45.8	10.18	42.4	2.82	11.8	13.00	54.2	37 589	27 316	2 400	20.26	14.28	10.49	58.78	0.00238
15	Steam...	4.75	19.8	5.43	22.0	13.82	57.6	10.35	80.2	14 049	8 988	1 890	16.17	7.69	13.01	36.77	0.00409
16	Electric.	11.61	48.3	9.84	41.0	3.55	10.7	12.30	51.7	46 197	34 452	2 980	45.98	14.28	10.49	71.75	0.00208
17	Steam...	6 63	26.4	4.17	17.2	12.50	50.3	17.97	73.6	22 960	14 716	2 820	22 28	7.58	13.91	42.87	0.00291
18	Electric	5.82	24.2	5.50	22.9	12 58	52.5	18.18	75.8	17 725	12 197	2 180	21.65	7.89	10.49	40.56	0.00332
19	Steam...	6.66	23.2	9.85	36.7	5.80	37.1	18.44	76.8	23 161	16 483	2 960	24.36	10 01	13.01	47.98	0.00291
20	Electric.	10.51	43.6	9.93	41.4	3.50	14.8	13.40	56.2	44 484	30 409	3 160	38.68	14.09	10.49	65.76	0.00191
21	Steam...	4.71	19.5	4 39	16.3	14.90	60.2	19 29	80.5	16 645	11 855	2 410	20 45	7 69	18.01	41.09	0.00352
22	Electric.	9.62	40.1	10.84	45.3	4.04	16.7	14.38	59.9	46 581	35 859	3 780	44.66	14.99	10.49	86.41	0.00194
23	Steam..	5.62	22.5	5.84	21.3	7.44	29.2	13.38	70.5	16 294	12 004	2 130	29.01	11.56	10.81	40.63	0.00364
24	Electric.	9.78	40.7	11.56	48.3	2.54	11.0	14.02	59.3	45 400	34 478	3 580	49.56	14.98	10.49	97.09	0.00190
25	Steam	2.73	11.4	4.44	18.0	16.83	76.1	21.27	88.6	6 630	8 670	1 420	7.00	6 45	18 01	26.49	0.00659
26	Electric.	12.01	50.1	9.69	40.4	2.80	9.5	11.92	49.9	44 780	38 068	2 760	46.76	14.38	10.49	71.62	0.00315
27	Steam...	2.30	9.6	2.42	14.3	18.38	76.1	21.70	90.4	10 654	7 153	3 110	16.06	4.66	18.01	38.94	0.00474
28	Electric	11.51	48.0	10.16	42.3	2.24	9.8	12.40	52.0	50 343	37 983	3 300	47.14	14.20	10.40	71.96	0.00186
Total	Steam....	71.68	......	84.62	......	174.65	......	259.87	......	263 591	174 556	......	284.00	104.60	179.44	568.04	......
Total	Electric.	117.09	......	193.27	......	94.81	......	218.06	......	453 909	338 966	......	466.06	176.20	148.36	783.11	......
Average.	Steam....	5.10	21.7	6.16	25.6	12.50	59.7	16.81	78.3	19 110	12 660	2 440	20.60	9.04	13.01	42 60	0.00297
Average.	Electric.	8.42	35.1	8.80	36.7	6.78	28.2	15.66	64.9	32 350	24 220	2 860	33.28	12.16	10.40	55.96	0.00261

Harlem Division, N. Y. C. & H. R. R. R. Runs made between October 4th and October 18th, 1907.

Switching, Grand Central, Wakefield and North White Plains Yards.

1	Steam....	1.94	6.90	7.80	14.29	7.57	0	1 294	0	0	1.00	1.79	3 110	1.59	......	......	3.92	6 87	8 89	19.16	0.0145	......	......
2	Electric..	1.00	7.80	1.01	6.01	7.90	1	702	16	15	......	......	......	......	90	185	9.61	5 76	4.20	12.48	0.0163	0.83	12 48
3	Steam...	0.84	2.50	8.38	14.74	4.56	0	818	0	0	0.00	2.48	1 330	1.64	......	......	3 98	6 76	6 45	16.48	0.0327	......	......
4	Electric..	1.00	3 12	2.44	5.50	5.64	4	309	01	58	......	......	......	......	122	180	2.19	3.87	2 80	9.45	0.0106	0.16	2.88
5	Steam...	1.67	2 81	7.62	10.85	4.59	0	803	0	0	0.27	2 14	1 300	1.62	......	......	2.83	5.16	6.77	12.70	0.0163	......	......
6	Electric..	0.20	0.82	2 80	2.61	1.97	0	187	0	0	......	......	......	......	91	112	0.56	0.71	1.06	2.00	0.0150	......	......
7	Steam...	1.60	6.89	6.50	13.39	5.06	0	1 043	0	0	1.05	2.21	1 400	1.94	......	......	3.76	5 86	8.07	17.69	0.0160	......	......
8	Electric..	0.07	2.06	2.61	4.67	5.14	0	498	0	0	......	......	......	......	64	138	1.78	2.82	2.33	6.88	0.0167	......	......
9	Steam...	1.38	4.06	6 97	11.00	5.98	0	942	0	0	0.87	2.07	1 760	1.87	......	......	3.21	4.63	6 71	14 28	0.0161	......	......
10	Electric..	0 80	4 46	3.15	7.61	2 89	0	632	0	0	......	......	......	......	76	141	2 04	3 61	3 70	9 25	0.0168	......	......
11	Steam...	0 85	2.85	5 86	8.71	4.37	0	870	0	0	0.86	2.31	1 680	1.67	......	......	3.15	2 96	5.17	11.28	0.0130	......	......
12	Electric..	0 80	2.79	0.00	3 85	4.73	0	451	0	0	......	......	......	......	30	153	1.80	2.03	4.22	6.55	0.0192	......	......
13	Steam...	0 62	5.58	5.06	10.64	4 82	0	844	0	0	0.66	2.06	1 690	1.99	......	......	3.12	4 16	6.21	13.48	0.0150	......	......
14	Electric..	1.01	4 47	2.99	7 46	6.80	0	646	0	0	......	......	......	......	69	150	2.35	2 68	3.70	8.71	0.0150	......	......
15	Steam...	0 77	4.76	7.70	12 46	7.21	1	1 348	56	119	1.47	2.44	2 200	1.83	......	......	6.13	3 87	7.17	16.89	0.0129	1.91	8.90
16	Electric..	0.84	2.76	3.06	6.87	6.74	0	545	0	0	......	......	......	......	73	189	1.00	2.93	2.93	7.87	0.0144	......	......
17	Steam...	0.76	3.96	7.48	11.39	4 89	0	850	0	0	0.00	2.61	1 160	1.86	......	......	3 48	0.99	6.56	12 99	0.0159	......	......
18	Electric..	0.46	1.00	5 22	6 23	2 57	0	344	0	0	......	......	......	......	21	127	0.52	1.16	2.42	4 06	0.0201	......	......
19	Steam...	0.78	4 67	7.17	11.84	6 45	2	957	4	6	1.12	2.08	1 690	1.97	......	......	4.08	1.93	0.89	14.90	0.0150	3.78	7.45
20	Electric..	0.98	3.19	3 29	6 46	6.11	0	580	0	0	......	......	......	......	66	148	2.17	2.62	2.77	7.46	0.0129	......	......
21	Steam...	0.97	4.93	7.16	12 08	6 01	2	1 122	89	71	1.66	2.10	2 080	1.65	......	......	4.01	3.90	7.08	15.60	0.0142	0 63	7.93
22	Electric..	0.98	4.97	2.49	7.16	6 59	4	683	57	80	......	......	......	......	90	152	2.85	3.41	3.55	9.31	0.0136	0.16	2.38
23	Steam...	0 81	5 94	7.46	13 30	4 86	0	816	0	0	1.00	2.86	1 570	1.92	......	......	2.62	4.53	7.70	15.85	0.0194	......	......
24	Electric..	0.80	3.28	3.43	6.78	6.00	2	676	4	4	......	......	......	......	76	131	1.99	2.74	2.90	7.63	0.0122	1.91	5.82
25	Steam...	1.06	6.61	6.49	12.08	6.08	4	1 073	30	19	1.41	2.94	1 960	1.84	......	......	5.12	4.40	7.09	16.01	0.0156	0 83	4.15
26	Electric..	0.79	2 78	2.41	5 10	6 86	0	651	0	0	......	......	......	......	91	140	2.38	2.48	2.62	7.48	0.0115	......	......
27	Steam...	0 97	6.01	7.45	13 46	5 40	0	1 081	70	79	1.28	2 86	2 020	1.96	......	......	4.48	4.64	7.83	16.89	0.0124	0.24	2.82
28	Electric..	0.54	2.74	2.84	6.06	6.83	0	549	0	0	......	......	......	......	80	137	2.82	2.83	2.59	7.93	0.0112	......	......
Total	Steam...	14.28	71.67	98.36	170.23	77.08	16	18 843	249	......	14.98	......	24 000	......	......	......	54.51	62.07	100.00	216.58	......	......	......
Total	Electric..	11.31	46.67	40.43	87.00	61.96	11	7 020	187	......	......	......	......	......	1 077	......	28.17	39.66	42.98	110.70	......	......	......
Average.	Steam...	1 07	6.14	7.05	12.17	6.56	1	960	20	90	1.07	2.43	1 720	1.74	......	......	2.00	4 46	7.14	13.47	0.0156	0 870	13.65
Average.	Electric	0.60	3.83	2 89	6.90	5.86	1	576	10	10	......	......	......	......	77	156	2.01	2.69	2.97	7.91	0.0140	0.800	10.00

Hauling—Steam, Wakefield and North White Plains—Electric, G. C. S. and Wakefield.

1	Steam....	3.78	0.72	4.06	4.78	74.50	28	28 027	11 979	3 170	4.29	0.43	9 070	0.46	......	......	18.94	5.80	4.82	27.56	0.00100	0.00229	0.96	6.1	10.7	50
2	Electric..	6 54	6.23	1.62	7.76	156.27	50	68 812	24 567	3 756	......	......	......	......	1 244	31.8	33.55	6.56	6.29	47.50	0.00189	0.00190	0.93	3.2	33.6	50
3	Steam...	3.61	0.28	4.53	4 81	56.67	26	28 244	11 678	3 210	6.41	0.62	9 600	0.41	......	......	23.79	3.35	4.60	31.08	0.00168	0.00277	1.23	8.6	18.5	43
4	Electric..	6.48	4.40	6.42	10.91	147.96	58	49 035	28 867	4 400	......	......	......	......	1 453	64.0	58.21	3.84	7.60	54.65	0.00127	0.00159	0.94	3.3	22.6	56
5	Steam...	3.47	1.10	7.06	8.13	74.35	30	27 890	14 809	4 280	6.76	0.54	11 330	0.41	......	......	25.10	2.25	8.00	24.54	0.00123	0.00231	1.14	3.8	21.0	50
6	Electric..	3.28	1.60	15.30	16.93	77.03	23	18 866	16 550	4 790	......	......	......	......	653	28.6	14.16	2.76	8 63	28.74	0.00142	0.00172	1.16	1.3	23.7	45
7	Steam...	3.00	1.24	2.97	5.27	79.80	28	26 783	12 768	3 970	6.63	0.56	8 970	0.34	......	......	24.59	3.00	4.94	33.00	0.00194	0.00250	1.18	6.0	20.6	56
8	Electric..	6.80	5.75	9.11	12.80	183.60	49	83 613	20 762	3 050	......	......	......	......	1 064	53.7	36.60	5.11	3.16	44.90	0.00136	0.00217	1.07	4.0	31.8	50
9	Steam...	4.10	0.97	6.55	7.52	60.23	36	43 417	14 377	3 510	6.53	0.51	12 220	0.47	......	......	24.56	4.00	6.93	34.03	0.00163	0.00243	0.97	9.0	19.6	56
10	Electric..	6.18	4.67	6.78	10.45	138.47	49	62 592	20 853	4 030	......	......	......	......	1 142	35.1	39.94	0.46	6.79	43.19	0.00133	0.00207	1.06	3.5	23.6	48
11	Steam...	3.39	2.18	8.60	11.07	68.40	25	29 370	11 400	3 370	5.76	0.56	10 890	0.43	......	......	21.58	4.50	7.84	33.92	0.00145	0.00338	1.50	8.7	20.2	52
12	Electric..	2.85	2.49	9.01	11.50	86.58	26	16 570	10 575	3 710	......	......	......	......	681	63.9	16.54	3.91	6.27	26.79	0.00162	0.00253	1.03	2.5	21.9	50
13	Steam...	4.57	2.43	5.54	7.97	89.80	30	30 472	14 792	3 240	7.08	0.50	14 650	0.45	......	......	28.00	4.54	6.80	30.94	0.00171	0.00270	1.88	8.4	19.6	50
14	Electric..	6.09	3.03	6.49	9.44	145.74	49	37 099	21 000	3 450	......	......	......	......	1 201	24.9	34.12	6.75	6.70	47.85	0.00126	0.00198	0.27	3.0	25.9	50
15	Steam...	2.28	1.62	6.27	7.49	68.40	27	23 282	12 812	5 760	6.51	0.61	9 890	0.41	......	......	24.52	6.14	5 84	33.50	0.00138	0.00272	1.84	9.0	20.9	64
16	Electric..	6.42	5.40	5.38	10.57	149.58	40	37 850	23 092	3 620	......	......	......	......	1 504	34 6	34.00	7.01	7.56	49.57	0.00131	0.00212	1.01	2.9	23.4	54
17	Steam...	3.46	1.14	7.93	8.59	66.78	23	25 024	15 023	3 770	7.06	0.55	8 560	0.54	......	......	26.12	2.86	6.45	35.41	0.00140	0.00272	1.26	8.5	20.1	50
18	Electric..	1.93	1.65	13.58	16.28	50 90	24	16 632	11 861	6 150	......	......	......	......	534	31.7	13.98	3.01	7.67	24.56	0.00146	0.00205	1.02	9.0	26.1	53
19	Steam...	3.87	1.54	6.49	6.08	68.40	25	23 870	11 400	3 800	6.56	0.50	11 540	0.49	......	......	25.45	3.83	6 18	35.28	0.00151	0.00309	1.41	6.5	20.4	50
20	Electric..	6.21	6.06	0.27	11.45	147.05	50	53 633	24 663	3 970	......	......	......	......	1 377	65.6	36.09	7.00	7.72	50.81	0.00131	0.00206	1.01	3.3	28.7	49
21	Steam...	2.82	1.63	9.00	7.63	68 40	23	23 870	11 400	4 040	7.03	0.07	10 640	0.46	......	......	23.04	3.26	5.90	36.25	0.00161	0.00309	1.41	8.7	30.6	59
22	Electric..	3.07	4.94	4 68	9.62	148 90	49	38 487	24 821	4 000	......	......	......	......	1 270	68.0	33.32	0.70	6.94	46.20	0.00122	0.00193	0.96	2.8	24.5	55
23	Steam...	4.12	0 64	5 14	5.58	68 40	26	23 870	11 400	2 770	7.53	0.72	11 230	0.48	......	......	27.81	2.36	5.31	36.11	0.00156	0.00317	1.44	10.9	16.6	54
24	Electric..	6.59	6.38	2 40	11.78	137.61	44	54 914	21 841	3 310	......	......	......	......	1 144	25.8	30.08	0.05	7.59	41.84	0.00127	0.00202	1.00	2.1	24.6	45
25	Steam...	3.91	1.56	6.36	7.01	76 60	29	27 189	18 224	3 260	8.80	0.73	12 660	0.46	......	......	32.78	3.56	6.92	42 89	0.00160	0.00320	1.46	8.6	30.4	58
26	Electric..	6.17	4 54	7.25	11.66	146 30	45	37 791	23 712	3 850	......	......	......	......	1 312	54.9	34.60	7.40	7.87	49 95	0.00133	0.00211	1.04	3.0	24.0	40
27	Steam...	3.97	0.76	4 94	5 70	79 80	23	26 733	12 706	3 000	7.96	0.87	12 070	0.49	......	......	29.52	3.04	5.16	37.74	0.00141	0.00295	1.38	8 6	20.6	54
28	Electric..	5.90	4.96	7.12	12.09	147.72	52	59 723	23 692	4 290	......	......	......	......	1 561	34.3	23.09	7.17	7.90	50.76	0.00198	0.00198	0.98	2.8	24.6	50
Total	Steam...	52.16	17.41	81.90	99.40	1 066.56	289	358 631	177 285	......	96.67	......	156 080	......	......	......	259.34	49.68	89.14	401.10	......	......	......	......	......	......
Total	Electric..	74 80	59 35	109.85	165.00	1 767.00	606	465 020	301 150	......	......	......	......	......	16 718	......	412.90	91.99	105.83	608.07	......	......	......	......	......	......
Average.	Steam...	3.72	1.94	5 66	7.10	74.04	96	25 620	12 660	3 400	6.90	0.00	11 150	0.43	......	......	25 67	3.55	6.87	36 09	0.00137	0.00277	1.36	8 6	10.9	50
Average.	Electric	5.31	4.94	7.40	11.84	136.22	43	53 910	21 610	4 060	......	......	......	......	1 195	35.8	29.44	6.57	7.42	43.44	0.00131	0.00202	1.00	2 9	23.6	55

Total for Both Classes of Service.

1	Steam....	6.09	36 9	7.85	21.2	11.83	47.2	18.98	79.1	26 851	11 979	2 890	22.83	10.37	13.01	46.54	0.00398
2	Electric..	7.54	31.8	15.85	57.7	2.53	10.6	16.20	68 2	40 675	24 562	3 220	56.16	14.26	10.49	80.06	0.00240
3	Steam...	4.45	18.5	6.04	27.7	12.91	33.8	19.56	81.5	24 057	11 678	2 600	27.07	10.83	13.01	50 46	0.00420
4	Electric..	7.55	31.4	7.61	31.7	8.88	36.9	16.47	68.6	43 826	28 028	3 840	41.40	12.31	10.49	64.10	0.00228
5	Steam...	6.04	21.0	4 41	18.4	14.50	60.6	18.86	79.0	26 688	14 889	2 050	27.03	9.10	13.01	47.84	0.00315
6	Electric..	3.54	14.8	2.21	9.2	18.26	76 0	20.46	85.2	19 084	15 550	4 390	14.71	4.36	10.49	29.56	0.00191
7	Steam...	5.40	23.6	8.13	33.9	10.47	43.6	18.59	77.5	27 776	12 768	2 360	26 26	6.48	13.01	50.75	0.00508
8	Electric..	6.47	27.0	5.81	24 2	11.72	48.8	17.53	73.0	63 501	20 762	3 210	36.42	10.48	10.49	51.31	0.00347
9	Steam...	6.45	33.6	5.06	21.1	13.52	50.3	18.57	77.4	60 289	14 377	2 660	27.77	8.89	13.01	49.17	0.00643
10	Electric..	6.94	25.2	9.00	37.6	8.93	37.2	17.06	74.8	83 144	20 863	3 460	31.96	9.97	10.49	52.44	0.00251
11	Steam...	4.24	17.9	5.06	21.0	14.75	61.4	19.78	82.4	24 940	11 400	2 700	24.72	7.46	13.01	45.20	0.00396
12	Electric..	3.55	15.2	5.28	22.0	15.02	62.6	20.55	84.8	16 971	10 575	2 900	18.34	6.94	10.49	35.37	0.00334
13	Steam...	5.60	23.4	6.01	25.4	10.00	44.2	18.61	77.6	31 310	14 792	2 740	31.72	6.50	13.01	58.42	0.00361
14	Electric..	7.10	29.6	8.42	35.1	8.48	35.3	16.90	70.4	33 569	24 060	3 390	56 45	10.43	10.49	67.37	0.00298
15	Steam...	4.96	16.9	6.96	24 9	12.97	56.2	10.95	63.1	25 830	12 394	2 060	20.87	7.01	13.01	40 99	0.00409
16	Electric..	7.25	30.2	9.28	33.7	7.46	31.1	16.74	69.8	66 076	23 292	3 210	35.19	10.60	10.49	67.94	0.00248
17	Steam...	4.99	17.6	5.10	21.2	14.68	61.2	19.78	82.4	20 080	15 023	3 000	28.80	6.79	13.01	48.40	0.00371
18	Electric..	2.40	10.0	2.65	11 8	18.77	78.2	21.60	90.0	17 076	11 991	5 000	14 80	4 17	10.49	29 46	0.00246
19	Steam...	4.13	17.2	6.21	26.0	13.56	50.0	19.87	82.3	24 327	11 404	2 760	22 55	7.62	13.01	50.16	0.00439
20	Electric..	7.07	29.4	8.97	34.6	8.66	30.1	16.93	70.6	30 219	24 663	3 490	38.06	9.52	10.49	59.97	0.00256
21	Steam...	4.99	17.9	6 56	27.4	13.16	54.7	19.71	82 1	24 492	11 469	2 670	20.95	7.16	13.01	61.12	0.00446
22	Electric..	7.06	29.6	9.01	40.0	7.87	30.7	12.98	70.7	30 160	21 878	3 470	35.67	10.11	10.49	50.27	0.00231
23	Steam...	4.93	20.6	9.48	27.0	12.69	53.4	19.07	79.4	24 186	11 400	2 910	31.63	7.82	13.01	51.86	0.00456
24	Electric..	6 49	27.0	6.00	26.1	8 56	35.9	17.51	73.0	50 409	21 845	3 370	33.01	9.87	10.49	51.87	0.00257
25	Steam...	4.96	20.7	7 28	30.3	11.78	49.0	19.04	79.3	28 264	18 941	2 670	27.97	8.08	13.01	50.00	0.00448
26	Electric..	6.96	29.0	7 86	32.7	9 86	40 9	17.01	71.0	38 443	23 712	3 470	30.97	9.97	10.49	57 48	0.00249
27	Steam...	4 94	20.2	6 77	28.2	12 38	51.0	19.10	79.8	27 753	12 838	2 650	24.00	7.02	13.01	54.03	0.00406
28	Electric..	6.88	28.6	7.76	32.1	9.47	39.4	17.17	71.5	40 374	23 692	3 700	23.01	9.62	10.49	50.00	0.00236
Total	Steam...	56 37	......	86 28	......	181 60	......	209.63	......	379 594	177 634	......	413.85	111.75	182.14	707.74	......
Total	Electric..	86.00	......	108.52	......	144.08	......	250.00	......	472 946	301 298	......	440.27	121.58	146.86	718.71	......
Average.	Steam...	4.73	19 7	6 38	26.6	12 66	53.7	19.27	80.3	20 610	12 660	2 680	23.57	7.96	13 01	50.56	0.00369
Average.	Electric	0.14	25.6	7.50	31.6	10.30	43.9	17.80	74.4	33 780	21 520	3 600	27.45	9.40	10.49	61.34	0.00259

Items.	Hauling.	Total.
Pounds coal per ton mile of cars.......	1.29	1.41
Gallons water per ton-mile of cars......	0.98	1.01
Watt-hours per ton-mile of cars.......	33.8	33.8

Tons of coal are based on 2 240 lb.
Ton-miles are based on 2 000 lb.

TABLE 3.—SUMMARY OF COMPARATIVE TESTS OF STEAM AND ELECTRIC LOCOMOTIVES.

Kind of locomotive.	Miles per day.	Cars per day.	Busy hours per day.	Hours ready for duty daily.	Percentage of time dead.	Total ton-miles daily.	Car ton-miles daily.	Percentage of car ton-miles to total.	Car ton-miles per busy hour.	Coal or current per car ton-mile.	Total cost per car, in cents.	Speed and stops. Average miles per hour.	Speed and stops. Maximum miles per hour.	Speed and stops. Stops.	Cost per 1 000 car ton-miles. Supplies.	Cost per 1 000 car ton-miles. Wages.	Cost per 1 000 car ton-miles. Interest, depreciation, and repairs on locomotives.	Cost per 1 000 car ton-miles. Total.	Watt-hours required to do work of 1 lb. coal.
SWITCHING SERVICE—GRAND CENTRAL TERMINAL.†																			
Steam	10.91	55	† 1.83	† 6.16	† 0.52	2 580	916	0.35	501	3.36 lb. coal.	35.2	....	....	....	$8.06	$5.34	$7.61	$21.01	
Electric	11.13	53	† 2.01	† 6.80	† 0.26	1 980	914	0.46	445	264 watt-hr.	28.5	....	....	....	6.88	5.25	4.40	16.53	79
Advantages in favor of electric locomotives.	0.22	....	† 0.18	† 0.64	† 0.26	......	......	0.11	......	....	6 7	....	....	....	1.18	0.09	3.21	4.48	
HAULING TO AND FROM MOTT HAVEN.†																			
Steam	40.0	45	† 3.36	† 5.18	† 0.53	16 540	11 720	0.71	3 490	0.46 lb. coal.	51.6	11.9	48	0.9	1.12	0.35	0.52	1.99	
Electric	78.4	95	† 6.41	† 10.42	† 0.30	30 370	23 310	0.77	3 640	44.3 watt-hr.	43.2	12.3	45	2.0	1.16	0.31	0.28	1.75	96
Advantages in favor of electric locomotives.	38.4	50	† 3.05	† 5.24	† 0.23	13 830	11 590	0.06	150	....	8.4	0.4	3	1.1	......	......	0.24	0.24	
ROAD SERVICE.*																			
Steam	74.04	28	3.72	‡ 11.11	‡ 0.54	25 620	12 660	0.49	3 400	1.22 lb. coal.	126.0	19.9	60	8.6	2.03	0.28	0.46	2.77	
Electric	126.22	43	5.34	‡ 13.70	‡ 0.43	33 210	21 510	0.65	4 030	52.3 watt hr.	100.0	23.6	55	2.9	1.37	0.31	0.34	2.02	43
Advantages in favor of electric locomotives.	52.18	15	1.62	‡ 2.59	‡ 0.09	7 590	8 850	0.16	630	....	26.0	3.7	5	....	0.66	......	0.12	0.75	

* Portion of time of locomotives engaged in other service not shown in this table. † Switching and hauling done by same locomotives.
‡ Total time of locomotives in all classes of service.

However, on the same wage basis for 1907 as for 1906, the month of August, 1907, showed a decrease in cost of terminal locomotive and yard operation of nearly $3 000, although the number of cars in and out increased from 64 984 to 68 519. In other words, the cost of operation decreased 9% while the work done increased 5½%, which is equivalent to a net saving of 13½ per cent.

Increased Revenue.—As to the fourth expectation—increased revenue from a larger volume of business—no definite conclusions can be reached until the extension of electrical service and the completion of the various other improvements afford an opportunity for increase in frequency and speed of train service; for the production of revenue from various sources at the terminal; and for the expansion of business that is sure to follow the enlargement of the facilities of the company throughout the suburban zone, not only as regards the local service, but in an even larger degree from long-haul freight and passenger traffic.

Summary of Results.—To summarize, the observations thus far made demonstrate that this pioneer electric installation in heavy-traction trunk-line work in the United States has fully accomplished the purposes that prompted its adoption, namely:

(1).—Abolition of nuisances incident to the steam locomotive; and

(2).—Increased capacity of the Grand Central Terminal,

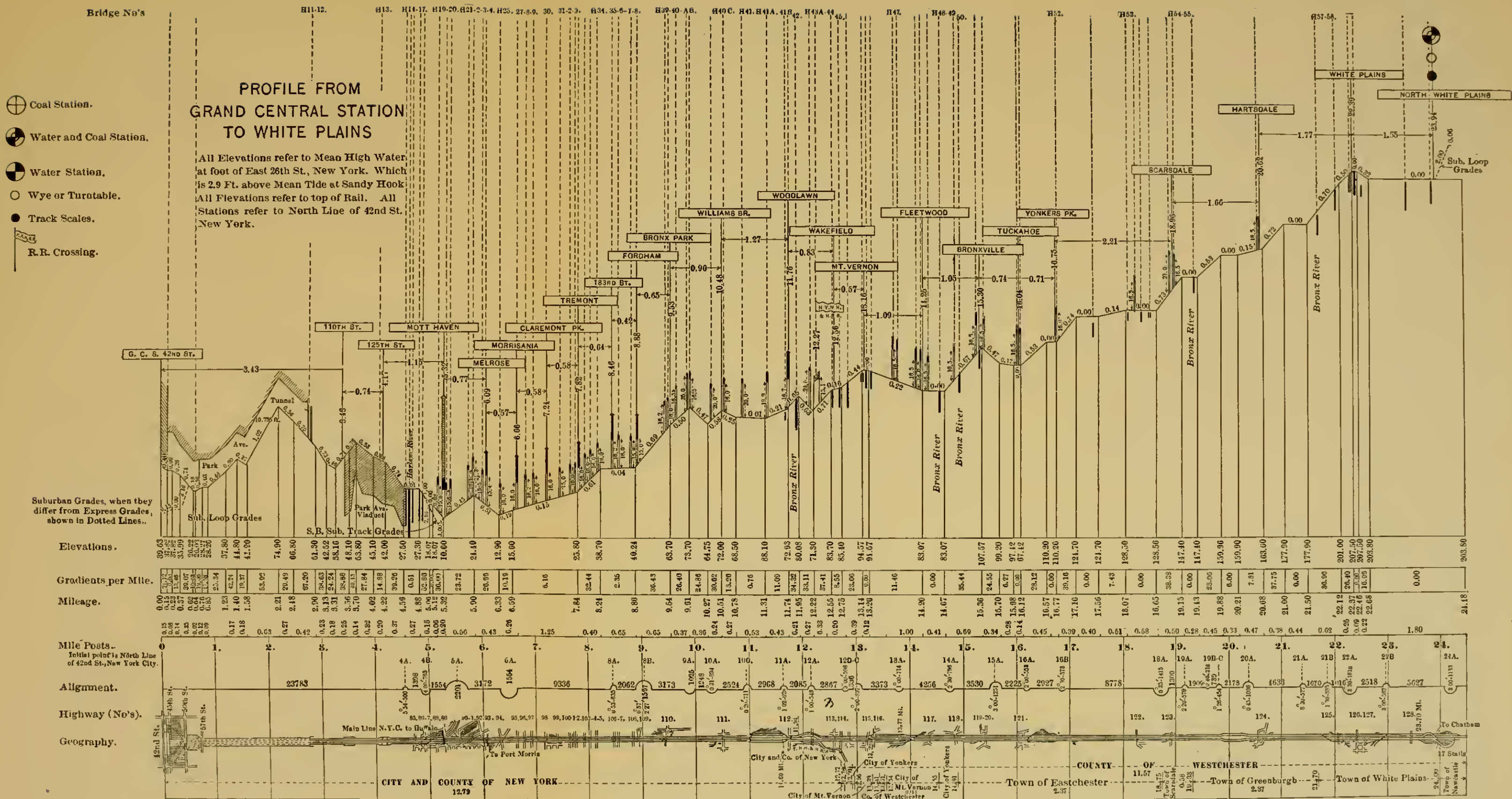
a full year in advance of the date fixed by law; and in addition:

(3).—The promise, with the completion of the changes, of a saving, in cost of operation, of from 12 to 27%, after providing for increased capital charges for electrification; and

(4).—The outlook of a large future growth of remunerative traffic, and other sources of revenue attendant on the use of electricity, much more than sufficient to provide for the increased capital charges for the other improvements.

Several years will be consumed in the gradual rounding out of the work as a whole; but it is gratifying to have this early indication of the success of the undertaking from both the engineering and financial standpoints.

PLATE XXII.
PAPERS, AM. SOC. C. E.
FEBRUARY, 1908.
WILGUS ON
ELECTRIFICATION OF SUBURBAN ZONE
OF N. Y. C. & H. R. R. R.



Other Operating Conclusions.—Apart from these results, it is interesting to note the conclusions, suited to this particular problem, that may be drawn from a study of the various observations.

Equipment designed for the electric system over which it is to operate offers economies so superior as to overshadow any other advantages that may be claimed for a kind of equipment that can be operated over several systems.

In switching service, the economy of electric traction lies in savings for supplies, and in lower unit fixed charges and repairs due to less lost time for repairs and care.

In slow-speed hauling, the advantage lies in the lower unit fixed charges and repairs of the electric locomotive, due to its ability to do more work while busy, and to less lost time for repairs and care.

High-speed road service shows advantages for electric traction in all three items: supplies, wages, and fixed charges and repairs. The small 18% increase in current consumption for the greater speed of road service, as compared with hauling service, is in marked contrast to the 165% increase in coal consumption for steam traction.

Opportunities for large economies lie in the thorough training of motormen in the manipulation of their controllers, a very simple problem as compared with the difficulties of teaching both the enginemen and firemen on steam locomotives to perform their duties so as to result in fuel economy.

Maintenance of Track and Structures.—It is yet too early to express in dollars the comparative effect of steam and electric traction on the cost of maintaining and renewing tracks and structures. Repeated systematic inquiries of all foremen in charge of electric zone track maintenance, and of the motormen operating electrical equipment, have brought out the practically unanimous opinion that the effect of electric locomotives, apart from slightly greater wear on switches, does not differ from steam motive power, on either line or surface of tracks, but that the former has better riding qualities. The superiority of electric traction is manifest, of course, in the cessation of costly corrosive action of locomotive gas on metallic structures, and the freedom from cinders which, with the steam locomotive, cause heavy maintenance costs for cleaning, rock ballast, and pointing brick tunnel arches.

Personnel of Engineering Department.—This paper would be incomplete without an expression by the writer, first as Chief Engineer and later as Vice-President of the Company, of his deep appreciation of the enthusiastic co-operation, and the able and earnest assistance of those associated with him in bringing this work to a successful issue.

The general principles and policies on electrical matters were determined by an Electric Traction Commission, the members of which, in addition to the writer as Chairman, were Bion J. Arnold, M. Am. Soc. C. E., M. Am. Inst. E. E.; Frank J. Sprague, M. Am. Soc. C. E., M. Am. Inst. E. E.; George Gibbs, M. Am. Soc. C. E.; and the General Superintendent of Motive Power, Rolling Stock and Machinery, New York Central Lines, at first Arthur M. Waitt, M. Am. Soc. C. E., M. Am. Soc. M. E., later succeeded by Mr. John F. Deems. Edwin B. Katte, M. Am. Inst. E. E., M. Am. Soc. M. E., Electrical Engineer of the company, acted as Secretary.

The other improvements, including the preparation of plans and specifications, and the execution of all work, including electrification, by contract and by company forces, were under several corps, the heads of which, for the purpose of co-ordinating their efforts, formed the Construction Committee. Among the members of this Committee, including the writer as Chairman, were Mr. Charles A. Reed, Executive of the Associated Architects for the Electric Zone, Messrs. Reed and Stem, and Warren and Wetmore; Mr. W. H. Knowlton, Principal Assistant Engineer; Mr. E. B. Katte, Electrical Engineer; George A. Harwood, M. Am. Soc. C. E., Terminal Engineer; Mr. Azel Ames, Signal Engineer; Mr. George A. Berry, Engineer of Company Forces; Mr. Henry A. Stahl, Office Engineer; Mr. Victor Spangberg, Designing Engineer; and Mr. J. L. Holst, Bridge Engineer. Associated with this Committee were also Mr. Ira A. McCormick, General Superintendent of the Electric Zone, and A. T. Hardin, Assoc. M. Am. Soc. C. E., Assistant General Manager, as representatives of the operating department.

To the many contractors, who with few exceptions performed their work with skill and fidelity, appreciative acknowledgments are also extended.

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PAPERS AND DISCUSSIONS.

THE FLOOD OF MARCH, 1907, IN THE SACRAMENTO AND SAN JOAQUIN RIVER BASINS, CALIFORNIA.*†

By W. B. Clapp, M. Am. Soc. C. E., E. C. Murphy, Assoc. M. Am. Soc. C. E., and W. F. Martin, Jun. Am. Soc. C. E.

Introduction.

The Sacramento and San Joaquin Valleys were visited, in March, 1907, by one of the most destructive floods that have ever occurred in California, the resulting financial loss being unquestionably greater than that from any other flood of which there is record. The greatest damage was done in the valleys of the trunk streams, especially Sacramento Valley. The Lower Sacramento River and its two largest tributaries, Feather and American Rivers, reached the highest stages ever recorded, and record stages were reached by other tributaries of the Sacramento and by the San Joaquin and its tributaries.

The flood was remarkable in many respects. In the first place, it was preceded by a period of heavy precipitation, and consequent flood stages of all the streams, a condition which had prevailed intermittently for several preceding weeks. As a result, the earth was thoroughly

* The data upon which this paper is based were collected by the Water Resources Branch of the United States Geological Survey in co-operation with the State of California, and the paper is published by permission of the Director of the Survey.
Further acknowledgments are due to Mr. J. H. Scarr, the district forecaster of the United States Weather Bureau, and to the engineering department of the Southern Pacific Railway, for data furnished.

† This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

saturated, and all the surface basins which impound and store flood waters temporarily were full. Particularly was this true of the large flood basins on each side of the Sacramento River. Then, too, this flood was due to a general precipitation of extraordinary intensity throughout the entire drainage basin (the storm covering a period of several consecutive days), and also to comparatively high temperature and consequent rapid melting of snow in the higher altitudes.

This flood was remarkable, also, because of the record-breaking stages of so many of the streams, such as the Lower Sacramento River and the Feather, Yuba, and American Rivers. Not only were they higher than ever known before, but they maintained their high stages for a moderately long period. All the other streams of the water-shed also maintained high stages for a like period, so that the resultant was a flood of exceptional height and extent, and of considerable duration. For the 4-day period, March 18th to 21st, the mean rate of run-off from the mountains and foot-hills of the Sacramento Basin alone was about 530 000 cu. ft. per sec., or more than 22 cu. ft. per sec. per sq. mile.

During this flood, special effort was made by the engineers of the United States Geological Survey to obtain valuable flood data. The flow of nearly all the important tributaries of both the Sacramento and San Joaquin River systems was gauged in the foot-hills above the point of débouchure. The flow from 83% of the mountains and foot-hills in the Sacramento Basin was measured at eleven gauging stations. In the San Joaquin Basin the flow from 41% of the mountains and foot-hills was measured at six gauging stations. Unfortunately, no gaugings were made of the San Joaquin itself.

It is believed that the data obtained during this flood will fully repay the State of California for its generous co-operation with the United States Geological Survey in the study of its water resources. Data are now available for planning for these great valleys a more comprehensive reclamation system than has been possible heretofore. The importance of the data collected will be appreciated when it is recalled that the rate of run-off from the mountains and foot-hills of the Sacramento Basin alone for a period of 4 consecutive days, March 18th to 21st, was 112% greater than the rate used as a maximum by the 1904 Commission of Engineers, after a careful study of all flood data on record, including those of the 1904 flood. It is doubtful if any combination of causes or conditions will ever produce a larger rate of delivery of water to this valley for a 4-day period than occurred during the flood of March, 1907.

Topography and Drainage of the Water-shed.

California is traversed, in a general northwest-southeast direction, by two distinct and approximately parallel ranges of mountains which extend almost the entire length of the State. Near the eastern border is the Sierra Nevada; not far from the shore line on the west is the Coast Range. These two ranges merge into each other about 40 miles south of the California-Oregon boundary line, the meeting point being Mount Shasta, which has an elevation of 14 380 ft. They are merged again south of Bakersfield by a cross-range known as Tehachapi Mountains.

The elevation of the Sierra Nevada ranges from about 6 000 ft. east of Mount Shasta at the north, to 14 501 ft. south of Yosemite National Park where the range culminates in Mount Whitney. Beckwith Pass, about 150 miles south of the northern boundary line, is the lowest pass through the range, and has an elevation of 5 300 ft. The Coast Range is comparatively low, and is unbroken except at Carquinez Strait and the Golden Gate which permit the drainage through Suisun Bay to reach the Pacific.

The Sierra Nevada and Coast Ranges, merging at the north and south, inclose a water-shed approximately 58 000 sq. miles in area, with a single outlet near the middle of the western side. This water-shed is somewhat elliptical in shape, and has a length of about 540 miles from north to south and a width varying from 120 to 150 miles. It is drained by two large river systems, the Sacramento in the north and the San Joaquin in the south, and these are quite commonly referred to as the Sacramento and the San Joaquin River Basins.

Sacramento River has its source in the region of Mount Shasta, and flows almost due south through the trough of the water-shed until it discharges into Suisun Bay. The San Joaquin rises in the Sierra Nevada, in the region of Mount Lyell, just east of Yosemite National Park, at an elevation of 13 000 ft., and flows southwestward until it emerges from the foot-hills into the trough of the valley, when it turns and flows northwestward to its junction with the Sacramento near Suisun Bay, through which the combined volume of the two systems finds an outlet to the Pacific by way of San Pablo and San Francisco Bays and the Golden Gate.

The drainage of the water-shed determines its division into three distinct basins: On the north is the Sacramento Basin, 27 100 sq. miles in area, drained by the Sacramento River and its tributaries; in the center is the San Joaquin Basin, about 18 300 sq. miles in area, drained by the San Joaquin and its tributaries (excluding Kings River, which, for reasons given later, is classified under the Lake Basin); in the south is the Lake Basin, with an area of about 12 600 sq. miles, containing several lakes with their tributary drainage, but at the present time having no outlet discharging to the sea.

That portion of the three basins which is inclosed by the sharply-defined line of the foot-hills is called the "Great Valley of California." This valley has a length of about 400 miles from north to south, an average width of about 40 miles, and an area of probably 15 000 sq. miles, and is surrounded by steep mountains. The western mountain slope—that of the Coast Range—is comparatively narrow, having an average width of about 18 miles. Considering the entire length of the district, from north to south, the precipitation, as a whole, is light, and perennial streams are few, but, in the region about Clear Lake and Mount St. Helena, in the Lower Sacramento Basin, the precipitation is remarkably heavy, and occurs almost entirely as rain. The eastern mountain slope, which has an average width of about 58 miles, is visited by rather heavy precipitation throughout almost its entire length from north to south, particularly in the central part of the Sacramento Basin. A large percentage of the precipitation occurs as snow on the higher elevations. From this slope come all the larger tributaries to the Sacramento and San Joaquin Rivers and the San Joaquin itself, as well as the principal tributaries to the Lake Basin. The change from mountain to valley is quite abrupt along a well-defined line, but the slope of the valley is gentle and uniform.

What is commonly called the Sacramento Valley extends northward only to Iron Canyon, near Red Bluff. In the Report of the Commissioner of Public Works to the Governor of California, in 1894 (page 28), the valley is described as having a total area of about 4 250 sq. miles, divided as follows: 2 510 sq. miles of high lands, not subject to overflow; 450 sq. miles of lower lands, overflowed occasionally by high floods; 1 250 sq. miles of low lands, overflowed periodically;

MAP OF WATER-SHED
OF
SACRAMENTO AND SAN JOAQUIN RIVERS
IN CALIFORNIA
SHOWING
TRIBUTARY STREAMS, UNITED STATES GAUGING AND
PRECIPITATION STATIONS,
AND PROPOSED RESERVOIR SITES
IN SACRAMENTO BASIN

FIG. 1.

and 38 sq. miles of perennial stream surface. Below the mouth of Stony Creek (Plate XXIII and Fig. 1) the central portion of the valley is a flood plane of unusual extent, the immediate river banks being from 5 to 20 ft. higher than the land on either side some distance from the river. In the vicinity of the river banks the ground slopes rapidly from the river toward the trough of the flood basins on either side, but, as the bottom of the trough is approached, the slope becomes more gradual. The lowest portions of the flood-basin troughs are from 2 to 7 miles from the river channel.

The large flood basin on the west side of the Sacramento is divided into two smaller basins by a ridge of débris brought down by Cache Creek. These are the Colusa Basin in the north and the Yolo Basin in the south. The large flood basin on the east side of the Sacramento is divided into four smaller basins by Marysville Buttes and the Feather and American Rivers. From north to south, they are called Butte Basin, Sutter Basin, American Flood Basin, and Sacramento Flood Basin. Fig. 1 shows the position of the flood basins. The following data regarding the area and capacity of these smaller flood basins are taken from the Report of the Commissioner of Public Works to the Governor of California, for 1894:

Colusa Basin is 50 miles long, from 2 to 7 miles wide, and has a capacity of 690 000 acre-ft. at flood stage. It discharges into Sacramento River above Knight's Landing through Sycamore Slough.

Yolo Basin has a length of 40 miles, an average width of 7 miles, and a capacity of 1 115 000 acre-ft. at flood stage. It discharges through Cache Slough into Steamboat Slough, and thence into the Sacramento near the foot of Grand Island, about 25 miles above the head of Suisun Bay.

Butte Basin is north of Marysville Buttes, and has an area of from 30 to 150 sq. miles, depending upon the river stage, and a capacity of 460 000 acre-ft. at flood stage. It discharges through Butte Slough into Sutter Basin.

Sutter Basin is south of Marysville Buttes and north of the Feather River. It has an area of 138 sq. miles, and a capacity of 895 000 acre-ft. at flood stage. It discharges through sloughs into Sacramento River above the mouth of Feather River.

The American Flood Basin is south of Feather River and north of the American. It has an area of 110 sq. miles, and a capacity of

MAP OF
THE LOWER PORTION OF THE
SACRAMENTO AND SAN JOAQUIN
VALLEYS
STATE OF CALIFORNIA
SHOWING
APPROXIMATE CONTOURS REFERRED TO MEAN SEA LEVEL,
FLOOD BASINS IN SACRAMENTO VALLEY, AND
THE MARGIN OF THE FLOOD PLAIN OF
MARCH, 1907

NOTE: The margin of the flood plane shown thus, does not necessarily indicate the area actually covered by water, but it shows approximately the area which was below the water surface in the river.

571 000 acre-ft. at flood stage. It discharges into the Sacramento north of the City of Sacramento, but, owing to its great depth, it is never free from water.

The Sacramento Flood Basin is a narrow strip south of the American River, extending from the City of Sacramento to Walnut Grove. Its area and capacity are unknown. It is flooded by overflow from the Mokelumne River and the breaking of levees on the east side of the Sacramento, but not as frequently as the other basins.

What is popularly known as the San Joaquin Valley includes the real valley of the San Joaquin Basin and the valley portion of the Lake Basin. This area is one, physiographically and historically, but, for the purpose of this discussion, it has been found advantageous to divide it. What is here meant by the San Joaquin Valley, therefore, is that portion of the San Joaquin Basin below the line of the foothills. It comprises an area of about 5 890 sq. miles south of Sacramento Valley and north of Kings River and the lakes of the Lake Basin. A portion of the discharge from Kings River reaches San Joaquin River, but discussion of it is excluded from this paper because of the impossibility of estimating its amount from the data at hand. The San Joaquin Valley has no well-defined flood basins, like the Sacramento Valley; it has, however, considerable areas of marshy lands adjoining the San Joaquin River, especially along the lower course of the river, and also a large flood plane which is overflowed annually.

The Lake Basin was originally a part of the present San Joaquin Basin. Many years ago, the southern half of the Great Valley of California received its drainage from the Sierras through a series of practically parallel streams flowing in a southwest direction to the trough of the valley, where their waters were gathered into one main channel which discharged into Suisun Bay. The San Joaquin River was one of these streams. In later years, however, those streams south of the San Joaquin built up large deltas which projected into the trough of the valley. Kings River, just south of the San Joaquin, and Kern River, near the southern end of the valley, built up particularly pronounced deltas, which extended completely across the old valley trough, practically isolating that portion south of Kings River Delta. This region contains several lakes at present. Among them are Kern and Buena Vista Lakes, south and west of Bakersfield, and Tulare

Lake, about 50 miles farther northwest. Kern River flows into Kern and Buena Vista Lakes, and, during high river stages, a portion of Kings River flows into Tulare Lake. During moderately low stages, practically all the water from these streams is taken out near the head of the deltas and used for irrigation, so that only the surplus water reaches the lakes. In very wet years, a portion of the water from Buena Vista Lake may pass northward into Tulare Lake, and, under very exceptional conditions, Tulare Lake would drain into San Joaquin River. Of late years, however, there is no record of any overflow from Tulare Lake into the San Joaquin; on the other hand, the lake has been dry a portion of the time, as in 1905, although since that year it has partially filled again. Further discussion of this basin is omitted from this paper.

Along the lower courses of the Sacramento and San Joaquin Rivers a large delta has been built up. In this delta region, each of the rivers has two or more channels in certain portions of its course, especially at the higher stages. Numerous distributing and cross-sloughs extend from one channel to another, and even from one river to the other at flood stages, and many islands are thus formed. These islands are very fertile, but they are overflowed every year unless protected by levees. Many of them, more particularly those along the Sacramento River, which vary in size from 1 600 to 43 000 acres, have been reclaimed, and are now protected from overflow by levees. (See Fig. 1.)

The chief tributaries of the Sacramento River are the Pit, Feather, and American Rivers, named in order from north to south. They have their sources in the summit of the Sierras, and flow in southwesterly courses. McCloud River is the principal tributary of the Pit, and enters it from the north. Yuba and Bear Rivers are the main tributaries of the Feather, and join it from the east. The most important tributaries of the Sacramento from the west are, in order from north to south, Stony, Cache, and Puta Creeks, but the last two are lost in the flood basins, and do not really reach the main channel of the river.

San Joaquin River receives all its principal tributaries from the east. They are, in order from north to south, Mokelumne, Calaveras, Stanislaus, Tuolumne, and Merced Rivers. They rise in the Sierras and flow westward in practically parallel courses. Cosumnes River is tributary to the Mokelumne from the northeast. (See Plate XXIII.)

Climate.

The climate of California is probably one of its most valuable assets. The principal factors affecting the climate are proximity to the Pacific Ocean, and diversified topography. The warm Japanese ocean currents, which bathe about 1 000 miles of the coast line, serve to equalize the temperature as normally affected both by seasons and latitude. The influence of the topography is such that altitude rather than latitude is the chief factor affecting temperature.

As regards precipitation, the year is divided into two well-defined seasons, the "rainy season" from November to March, and the "dry season" from April to October. The rainy season is usually marked by a series of storms, of greater or less severity, which form in the Pacific Ocean and move eastward to the coast, depositing their moisture before crossing the Sierra Nevada. The centers of the most severe storms generally strike the coast in the State of Washington and then move southward through Oregon into California between the mountain ranges. These storms almost invariably make their appearance in late winter or early spring, being, as a rule, most severe about the time of the vernal equinox. At this season the precipitation is quite general throughout the State, increasing with altitude and also with latitude.

Flood Conditions and Causes.

During the winter and early spring of each year, toward the end of the rainy season, the various streams of the Sacramento and San Joaquin Basins generally reach their highest stages. The most serious flood conditions invariably exist on the lower courses of the trunk streams, the Sacramento and the San Joaquin. On the Sacramento River, in particular, serious damage is inflicted on crops and transportation interests almost every year. Of course, the destructiveness of any flood is measured largely by its height and duration. In these basins the maximum height, and generally the greatest duration, of floods on the primary streams result from the simultaneous flooding of all the secondary and tertiary streams, a condition which obtains when there is a period of long-sustained precipitation throughout the entire water-shed, accompanied by high temperature and rapid melting of snow on the higher elevations. It was such a condition that brought about the flood of March, 1907.

Other conditions that contribute more or less to all floods in this area are the following:

1.—The steep, barren, and impervious slopes of the mountains and foot-hills, which result in streams of heavy grades and the rapid delivery of water to the valleys.

2.—The broad, flat valleys, with light grades and sluggish streams.

3.—The limited channel capacity. It is said that some of the trunk channels are not large enough to carry even one-third of the flood flow. Particularly is this true of the Sacramento River. Here the surplus water overflows into the flood basins, the result being either to increase or diminish the stage of the lower course of the river, depending on the volume of water in the flood basins at the beginning of the flood period and the duration of the period.

4.—The common outlet of the two river systems, with large tributaries of each system discharging into trunk streams near this outlet.

5.—The constriction of the flood area in the delta of the two rivers through the reclamation of large areas of overflow land by levees.

6.—The deposition of the débris resulting from hydraulic mining in several tributaries of the Sacramento River, the result of which has been the filling of channels and the reduction of gradients, thereby raising the flood plane several feet.

7.—The tidal and wind action in the delta of the two rivers.

Precipitation.

In the Sacramento Valley, the mean annual precipitation varies from 15 in. in the southern to 20 in. in the northern part, while, in the tributary foot-hill and mountain areas, it varies from 20 to 60 in., with an occasional maximum of 100 in. In the San Joaquin Valley, the mean annual precipitation varies from 10 in. in the southern to 15 in. in the northern part, and in the foot-hill and mountain areas it varies from 15 to 40 in. In the Sierras, the greater part of the precipitation is normally in the form of snow, and the magnitude of floods depends largely on its rate of melting. A heavy, warm rain on a deep, freshly fallen snow produces a maximum run-off.

In January and February, 1907, there were two periods of heavy and long-sustained precipitation, one from January 2d to 17th, and the other from January 24th to February 4th. The precipitation was unusually heavy over the Sacramento Basin, diminishing gradually toward the south. The precipitation during the first of these periods produced the ordinary winter stages of the tributaries of the Sacramento River; that during the second period produced flood stages on the tributaries of both the Sacramento and San Joaquin Rivers and high stages in the Lower San Joaquin. American and Bear Rivers reached stages almost as high as in the great flood of the following month. Yuba River was higher than at any time previously recorded.

In March there were two precipitation periods, one from the 2d to the 11th, in which the amount of rainfall was moderate, and the other from the 16th to the 25th, in which it was extraordinarily heavy. The precipitation of the latest period was accompanied by unusually high mean temperatures, especially in the higher altitudes, from the Feather River south to the Tuolumne, causing very rapid melting of snow and exceedingly large run-off. The average from 24 fairly representative meteorologic stations throughout the basin shows that the mean temperature for March 17th to 20th was about 5° above the mean for the month, with low daily maxima resulting from cloudiness and rain, and high daily minima due to the liberation of heat by the storm. The average greatest daily range in this period was only 16 degrees. These facts indicate that probably all stations with a monthly mean temperature as high as 25° had scarcely any freezing conditions from March 16th to 20th, when the precipitation was heaviest. Further, they show that, out of 113 stations located at various elevations throughout the Sacramento and San Joaquin drainage basins, at 105 of them all the precipitation from March 17th to 20th was probably in the form of rain or of snow in a melting condition.

Table 1 shows the monthly precipitation from January to March, 1907, the daily precipitation for the three days, March 17th, 18th, and 19th, when it was greatest, and the precipitation for the ten days, March 17th to 26th, for 120 places in the Sacramento and San Joaquin Basins, varying in altitude from 20 to 7 017 ft., arranged according to basins of tributary streams. Where possible, the monthly precipitation for the period of January to March, 1904, is also given, for comparison with the great flood of that year.

Table 2 shows the average precipitation and average mean temperatures for March at 113 stations arranged according to stream basins and in order of altitude.

Table 3 shows the results at 24 stations ranging in altitude from 60 to 5 270 ft., the data having been taken from the Climatological Report of the United States Weather Bureau for March, 1907. These particular stations were selected because they are the only ones in the basin for which daily temperatures have been published. They are fairly well distributed, both as regards area and altitude, and are probably as representative as any that could have been chosen. This table also shows the extraordinary intensity of precipitation from March 17th to 26th by percentages with reference to the total for the month, and also the normal for the month, covering a period of 21 years on an average.

These tables show conclusively that the precipitation from March 17th to 26th, and particularly on March 17th, 18th, and 19th, was phenomenally heavy for this section of the country. This large precipitation is rather evenly distributed throughout all the river basins, but there is a very noticeable and quite rapid, though comparatively regular, increase with the altitude. During the month, sixteen stations, with elevations of more than 3 500 ft., had more than 30 in. in depth of precipitation; about forty stations, with elevations of more than 1 500 ft., had more than 20 in.; and fully one-third of the total precipitation for the month fell on March 17th, 18th, and 19th. On one of these three days, seventeen stations, with an altitude of more than 2 000 ft., had precipitations of from 5 to 8 in. in 24 hours. It is noteworthy that the range of temperature with altitude was quite regular, and that there were no very low temperatures even at very high elevations. It is highly probable that at elevations of 5 000 ft. a large part of the precipitation occurred as rain or as snow which melted rapidly. Indeed, at Inskip, in the Feather Basin, with an elevation of 4 850 ft., a 24-hour rainfall of 8 in. was reported. Taking a record of 21 years on an average throughout the basin, it is seen that about 88% of the normal precipitation for March occurred on March 17th, 18th, and 19th, 1907, or, counting 20 days as normally rainy in this month, the intensity of this 3-day period was about 600% of the normal intensity for the month. During these 3 days the average precipitation at the sixteen stations, principally in the Feather and Yuba Basins, having more than 30 in. during the month, was 145% of the normal for the month, or at an average intensity of 1 000% of the normal.

TABLE 1.—Precipitation.

Rainfall Stations.				Precipitation, in Inches.								Mean temperature for March.
No.	Name.	Elevation.	Date.	Jan.	Feb.	Mar.	Total, 3 months.	Mar. 17th	Mar. 18th.	Mar. 19th.	Mar. 17th–26th.	
	Sacramento Drainage Basin.											
1	Delta	1 138	1904	3.96	21.19	23.93	49.08					
			1907	12.26	9.21	24.45	45.92	1.10	3.50	3.50	18.10	47.3°
2	Redding	565	1904	2.34	14.10	15.89	32.33					
			1907	8.57	9.09	7.28	24.94	1.36	0.88	0.11	4.57	49.0°
3	Dunsmuir	2 285	1904	5.03	24.00	22.90	51.93					
			1907	20.53	8.27	18.64	47.44	2.00	2.87	2.72	13.61	46.9°
4	Sisson	3 555	1904	3.26	10.91	15.90	30.07					
			1907	9.48	2.84	13.16	25.48	0.00	3.55	1.27	11.27	37.6°
5	Nimshew	2 000	1904	10.82	8.72	13.27	32.81					
			1907	17.64	13.12	27.69	58.45	5.54	4 00	3.02	18.36	42.4°
6	Sacramento	71	1904	0.15	5.26	5.43	11.14					
			1907	4.63	2.37	7.28	14.28	0.42	1.74	0.56	4.75	50.9°
7	Fruto	624	1904	0.75	6.13	7.07	13.95					
			1907	6.43	1.95	4.67	13.05	0.45	0.40	0.30	2.90	50.1°
8	Shasta	1 148	1904	2.79	24.86	16.37	44.02					
			1907	13.65	7.89	14.47	36.10	1.54	2.25	1.03	10.98	48.6°
9	Corning	277	1904	0.60	4.95	7.30	12.85					
			1907	3.60	2.60	5.05	11.25	0.00	0.95	0 28	2.68	49.1°
10	Red Bluff	307	1904	1.44	6.63	8.33	16.40					
			1907	6.10	3.13	5.92	15.15	0.62	0.28	0.00	2.93	48.4°
11	Tehama	220	1904	1.01	4.67	7.19	12.87					
			1907	4.75	2.96	5.38	13.09	0.86	0.28	0.30	2.79	50.8°
12	Chico	189	1904	0.80	5.64	9.33	15.77					
			1907	6.28	2.09	8.03	18.67	1.58	0.73	0.36	4.79	49.2°
13	Durham	160	1904	1.70	5.75	10.32	17.77					
			1907	6.45	2.09	8.39	16.93	1.61	0.87	0.45	4.49	50.4°
14	Willows	136	1904	0.45	3.44	7.61	11.50					
			1907	4.84	1.02	3.63	9.48	0.70	0.13	0.05	1.98	49.1°
15	Colusa	60	1904	0.66	3.13	5.67	9.46					
			1907	5.63	0.75	3.80	10.18	0.58	0.22	0.00	2.21	49.9°
16	Suisun	20	1904	1.12	6.50	7.52	15.14					
			1907	8.89	3.59	7.57	20.05	1.61	0.08	1.55	5.95	52.0°
17	Dunnigan	65	1904	0.66	5.33	8.87	14.86					
			1907	7.63	1.63	6.98	16.24	1.10	0.04	1.40	3.91	52.9°
18	West Branch	3 150	1907	.....	17.96	39.69	.....	....	....	....	.....	.....

TABLE 1.—(*Continued.*)

Rainfall Stations.				Precipitation, in Inches.								Mean temperature for March.
No.	Name.	Elevation.	Date.	Jan.	Feb.	Mar.	Total, 3 months.	Mar. 17th.	Mar. 18th.	Mar. 19th.	Mar. 17th–26th.	
	McCloud River Drainage Basin.											
19	Johns Camp	......	1904	4.50	19.73	27.26	51.49					
	Pit River Drainage Basin.											
20	Cedarville	4 675	1904	1.12	4.87	4.61	10.60					
			1907	1.99	3.70	3.31	9.00	0.67	0.40	0.06	1.59	34.4°
21	Alturas	4 460	1907	1.35	2.87	4.13	8.35	0.55	0.75	0.14	2.45	35.6°
	Feather River Drainage Basin.											
22	Magalia	2 321	1904	3.43	23.39	30.13	56.95					
			1907	23.57	10.71	37.75	72.03	7.65	6.66	2.79	24.42	41.9°
23	Oroville	250	1904	1.60	7.99	10.86	20.45					
			1907	6.71	3.59	10.90	21.20	1.10	1.44	0.52	5.57	51.0°
24	Butte Valley	4 020	1904	4.20	22.90	22.10	49.20					
			1907	11.96	6.78	26.76	45.50	....	....	....	......	......
25	Greenville	3 600	1904	2.39	18.81	15.53	36.73					
			1907	9.57	4.48	24.51	38.56	4.25	6.17	2.91	19.89	37.8°
26	Meadow Valley	4 730	1904	4.13	29.10	29.90	63.13					
27	Quincy	3 400	1904	2.46	22.10	10.83	35.39					
			1907	11.89	4.96	30.15	47.00	5.30	6.50	4.40	25.55	35.8°
28	Inskip	4 850	1907	......	......	45.30	......	....	....	....	......	......
29	Biggs	98	1904	1.09	4.98	8.35	14.42					
			1907	4.55	1.85	6.57	12.97	0.20	0.00	0.00	3.65	52.0°
30	Brush Creek	2 140	1904	4.81	23.11	25.01	52.93					
			1907	16.21	11.49	33.02	60.72	5.70	5.40	3.40	23.96	42.8°
31	Marysville	67	1904	1.19	5.18	7.77	14.14					
			1907	4.52	4.30	10.59	19.41	0.30	1.30	2.00	6.44	53.8°
32	Palermo	213	1904	1.48	7.22	9.35	18.05					
			1907	5.36	3.34	8.80	17.50	0.97	0.86	0.25	4.37	50.8°
33	Sterling City	3 525	1904	3.96	26.51	25.22	55.69					
			1907	24.63	17.54	43.38	85.55	6.66	7.90	6.16	32.86	37.0°

TABLE 1.—(*Continued.*)

Rainfall Stations.				Precipitation, in Inches.								Mean temperature for March.
No.	Name.	Elevation.	Date.	Jan.	Feb.	Mar.	Total, 3 months.	Mar. 17th.	Mar. 18th.	Mar. 19th.	Mar. 17th-26th.	
	Yuba River Drainage Basin.											
34	Colgate	650	1904	3.79	9.92	8.19	21.90					
			1907	7.86	10.28	19.31	37.45	....	....	....	......	......
35	Dobbins	1 650	1904	3.79	14.04	13.65	31.48					
			1907	10.54	8.98	19.43	38.95	2.50	2.33	2.60	13.04	......
36	Nevada City	2 580	1904	2.76	19.17	18.64	40.57					
			1907	10.21	8.22	24.62	43.05	2.47	3.34	3.63	17.76	41.6°
37	No. Bloomfield	3 200	1904	3.85	16.44	21.89	42.18					
			1907	10.25	9.23	28.64	48.12	3.07	4.51	3.97	21.10	39.8°
38	Cisco	5 939	1904	5.20	30.80	26.87	62.87					
			1907	14.70	6.25	24.20	45.15	1.00	1.00	3.60	14.10	34.4°
39	Summit	7 017	1904	4.20	30.40	21.30	55.90					
			1907	13.50	4.38	27.36	45.24	1.42	2.42	2.32	16.06	28.8°
40	La Porte	5 000	1904	4.48	30.35	31.66	66.49					
			1907	17.75	16.40	42.62	76.27	6.58	6.19	5.42	33.12	32.2°
41	Comptonville	3 400	1907	......	16.78	36.12	......	....	....	....	......	......
42	Woodleaf	3 250	1907	18.75	12.87	37.38	69.00	....	....	....	......	......
43	Deer Creek	......	1907	......	......	36.93	......	....	....	....	......	......
44	Head Dam	......	1907	......	......	26.78	......	....	....	....	......	......
45	Fordyce Dam	6 500	1907	12.11	11.26	29.01	52.38	....	....	....	......	......
46	Bowman's Dam	5 500	1904	5.37	45.61	39.51	90.49					
			1907	13.82	12.68	31.46	57.96	....	....	....	......	......
	Bear River Drainage Basin.											
47	Bear Valley	4 600	1904	4.46	34.26	27.99	66.71					
			1907	14.59	11.10	35.50	61.19	....	....	....	......	......
48	Wheatland	84	1904	1.09	6.14	7.22	14.45					
			1907	4.67	3.06	9.64	17.37	1.18	1.23	0.81	6.19	50.6°
49	Grass Valley	2 090	1907	11.22	11.79	26.15	49.16	....	....	....	......	......
50	Gold Run	3 222	1907	10.47	9.61	21.61	41.69	....	....	....	......	39.4°

TABLE 1.—(*Continued.*)

Rainfall Stations.				Precipitation, in Inches.								Mean Temperature for March.
No.	Name.	Elevation.	Date.	Jan.	Feb.	Mar.	Total, 3 months.	Mar. 17th.	Mar. 18th.	Mar. 19th.	Mar. 17th-26th.	
	American River Drainage Basin.											
51	Colfax	2 421	1904	3.50	20.10	20.46	44.06					
			1907	9.45	9.75	19.46	38.66	2.40	1.55	2.85	12.36	48.2°
52	Emigrant Gap	5 230	1904	3.75	25.10	31.22	60.07					
			1907	14.35	14.45	30.20	59.00	2.00	2.50	4.50	19.45	30.0°
53	Georgetown	2 650	1904	4.79	26.02	21.17	51.96					
			1907	8.96	13.50	29.07	51.53	3.58	1.08	4.90	19.47	42.4°
54	Placerville	2 109	1904	2.96	15.59	13.48	32.03					
			1907	8.13	8.15	20.54	36.82	2.03	1.06	4.28	14.62	47.0°
55	Rocklin	249	1904	1.29	7.94	2.18	16.41					
			1907	5.51	5.71	12.46	23.68	1.70	0.15	2.20	8.65	51.2°
56	Represa	305	1904	1.15	8.33	8.55	18.03					
			1907	6.31	5.31	12.39	24.01	....	....	....	.....	......
57	Auburn	1 360	1904	2.73	13.34	11.83	27.90					
			1907	8.35	9.70	16.66	31.71	2.08	0.39	3.11	11.23	47.6°
58	Blue Canyon	4 695	1904	4.81	30.61	26.14	61.56					
			1907	13.18	17.95	35.11	66.24	4.18	4.35	6.45	27.33	36.6°
59	Iowa Hill	2 825	1904	4.58	20.20	16.97	41.75					
			1907	11.52	10.13	24.36	46.01	2.42	2.88	3.21	16.35	42.6°
60	New Castle	970	1904	1.93	10.79	11.61	24.33					
			1907	7.09	6.72	14.10	27.91	1.35	1.18	1.78	8.49	50.0°
61	Folsom	252	1904	1.12	7.19	7.70	16.01					
			1907	5.25	5.65	11.06	21.96	1.42	0.10	2.08	7.33	51.0°
62	Pilot Creek	4 000	1904	5.48	29.88	25.45	60.81					
			1907	14.40	11.79	32.88	59.07	....	....	....	......	......
63	Towle	3 704	1904	3.84	25.50	23.29	52.63					
			1907	9.45	12.24	21.05	45.74	2.40	2.43	2.69	15.83	37.7°
	Stony Creek Drainage Basin.											
64	Fouts Springs	1 650	1904	2.34	9.44	12.73	24.51					
			1907	14.85	4.60	15.63	35.08	....	....	....	......	......
65	Julian	750	1904	0.75	4.79	6.22	11.76					
66	Orland	254	1904	......	4.06	6.36	......					
			1907	......	......	3.97	......	0.57	0.15	0.12	2.11	48.8°

TABLE 1.—(*Continued.*)

Rainfall Stations.				Precipitation, in Inches.								
No.	Name.	Elevation.	Date.	Jan.	Feb.	Mar.	Total. 3 months.	Mar. 17th.	Mar. 18th.	Mar. 19th.	Mar. 17th-26th.	Mean temperature for March.
	Cache Creek Drainage Basin.											
67	Bartlett Springs..........	2 375	1904	2.48	19.96	16.75	39.19					
68	Kono Toyee...............	1 350	1904	1.64	8.78	7.62	18.04					
69	Lake Port.....	1 325	1904	1.65	13.37	12.72	27.74					
70	Upper Lake	1 350	1904	1.62	11.19	10.14	22.95					
			1907	5.30	4.60	10.63	20.53	2.40	1.73	0.70	7.98	47.6°
71	Guinda....................	350	1904	0.75	6.80	7.55	15.10					
			1907	9.30	1.30	8.84	19.44	1.30	1.00	1.70	7.20	47.9°
72	Woodland................	63	1904	0.69	4.60	7.15	12.44					
			1907	4.45	3.24	5.90	13.59	....	....	....	4.10	50.5°
	Puta Creek Drainage Basin.											
73	Middletown......	1 300	1904	2.52	16.99	27.57	47.08					
74	Davisville	51	1904	0.53	5.05	7.57	13.15					
			1907	4.81	2.28	6.69	13.78	1.25	0.07	2.00	5.24	56.1°
75	North Lake Port..........	1 450	1907	5.45	4.30	12.35	22.10	....	....	....	......	......
76	Calistoga.	363	1904	2.65	16.08	16.10	34.83					
			1907	10.89	7.95	19.50	38.34	0.00	5.60	3.85	16.60	52.6°
77	Helen Mine.	2 750	1904	4.52	34.22	31.48	70.22					
			1907	27.21	11.66	36.73	75.60	7.40	6.64	5.10	28.90	43.4°
78	Vacaville.................	175	1904	1.67	8.61	11.73						
			1907	6.54	3.08	8.48	18.10	0.13	2.02	0.29	4.81	49.7°
79	Mt. St. Helena	2 300	1904	3.37	28.34	26.14	57.85					
			1907	19.95	12.18	24.20	56.33	....	....	....	......	......

TABLE 1.—(*Continued.*)

Rainfall Stations.				Precipitation, in Inches.								Mean temperature for March.
No.	Name.	Elevation.	Date.	Jan.	Feb.	Mar.	Total, 3 months.	Mar. 17th.	Mar. 18th.	Mar. 19th.	Mar. 17th-26th.	
	San Joaquin Drainage Basin.											
80	Farmington	111	1904	0.54	4.71	4.10	9.35					
			1907	4.70	2.65	5.47	12.82	....	....	....	......	52.9°
81	Fresno	293	1904	0.57	2.49	2.75	5.81					
			1907	3.35	0.94	1.74	6.03	0.12	0.00	0.32	0.56	52.8°
82	Le Grand	255	1904	1.15	2.60	2.90	6.65					
			1907	4.40	0.77	5.56	10.73	0.00	0.45	0.00	2.96	47.3°
83	Las Banas	121	1904	0.25	1.23	1.28	2.76					
			1907	3.17	1.17	4.39	8.73	0.00	0.45	0.67	2.78	53.0°
84	Mendota	177	1904	0.20	1.70	1.26	3.16					
			1907	2.83	1.31	1.79	5.93	0.00	0.03	0.00	0.64	55.5°
85	Merced	173	1904	0.55	2.30	2.34	5.19					
			1907	4.25	3.16	3.68	11.00	0.00	0.00	0.22	2.73	51.7°
86	Newman	91	1904	0.23	1.51	2.33	4.07					
			1907	3.35	1.49	3.82	8.66	0.15	0.00	0.67	2.42	51.8°
87	Stockton	23	1904	0.54	4.09	3.67	8.30					
			1907	3.94	2.52	6.03	12.49	0.53	0.06	1.22	4.07	51.2°
88	Storey	296	1904	0.69	2.69	2.47	5.85					
			1907	2.70	0.48	1.35	4.53	0.00	0.00	0.01	0.47	49.8°
89	Tracy	64	1904	0.46	2.10	1.93	4.49					
			1907	3.22	1.70	5.04	9.96	0.00	0.00	0.70	2.75	48.0°
90	Westley	90	1904	0.41	1.53	3.07	5.01					
			1907	5.18	1.39	3.55	10.12	0.00	0.18	0.48	2.31	55.2°
91	No. Fork	3000	1904	......	......	10.73	......					
			1907	9.19	4.42	14.30	27.91	....	....	....	......	.....
92	Pollasky	345	1907	4.20	0.86	4.24	9.30	....	....	....	......	.....
93	Lathrop	25	1907	3.58	1.54	4.64	9.76	....	....	....	......	41.5°
94	Antioch	46	1904	0.42	2.65	4.65	7.72					
			1907	3.23	1.80	6.43	11.46	0.65	0.03	0.83	4.38	54.8°

TABLE 1.—(*Continued.*)

Rainfall Stations.				Precipitation, in Inches.								
No.	Name.	Elevation.	Date.	Jan.	Feb.	Mar.	Total, 3 months.	Mar. 17th.	Mar. 18th.	Mar. 19th.	Mar. 17th-26th.	Mean temperature for March.
	Cosumnes Drainage Basin.											
95	Shingle Springs	1 427	1904	2.80	15.91	12.58	31.29					
			1907	5.05	8.50	16.84	30.39	....	....	....	......	......
	Mokelumne River Drainage Basin.											
96	Mill Creek	3 500	1907	10.20	8.00	24.45	42.65	2.06	2.36	4.04	15.96	41.5°
97	Kennedy Mine	1 500	1904	2.08	13.79	9.22	25.09					
			1907	6.25	6.25	13.85	26.35	....	....	....	......	......
98	Electra	725	1904	2.61	13.92	9.50	26.03					
			1907	7.47	5.15	18.01	30.63	0.26	4.55	0.47	10.04	53.2°
99	Mokelumne Hill	1 560	1904	2.44	13.35	9.52	25.31					
			1907	7.61	6.29	15.66	29.56	....	....	....	......	48.0°
100	West Point	2 326	1904	4.62	16.36	13.49	34.47					
			1907	9.38	6.66	19.76	35.80	....	....	....	......	......
101	Galt	49	1904	0.60	6.24	5.27	12.11					
			1907	4.00	3.29	7.59	15.38	1.00	0.04	1.47	5.92	51.3°
102	Ione	287	1904	0.90	7.05	5.00	12.95					
			1907	4.87	3.95	10.39	19.21	1.10	0.11	2.40	6.27	44.9°
103	Lodi	35	1904	0.72	5.77	4.85	11.34					
			1907	3.94	2.82	6.76	13.52	0.42	0.21	1.18	4.26	50.7°
	Calaveras Drainage Basin.											
104	Milton	660	1904	0.93	6.78	5.30	13.01					
			1907	4.76	2.53	9.27	16.56	0.34	0.35	1.53	4.90	51.2°
105	Valley Springs	673	1904	1.42	10.56	7.81	19.79					
			1907	5.51	4.31	11.12	20.94	0.92	0.25	2.62	6.66	54.2°
106	Jenny Lind	300	1907	......	2.61	9.38	......	....	....	....	......	......

TABLE 1.—(*Continued.*)

Rainfall Stations.				Precipitation, in Inches.								
No.	Name.	Elevation.	Date.	Jan.	Feb.	Mar.	Total, 3 months.	Mar. 17th.	Mar. 18th.	Mar. 19th.	Mar. 17th–26th.	Mean temperature for March.
	Stanislaus River Drainage Basin.											
107	Oakdale	156	1904	0.70	5.00	3.44	9.14					
			1907	3.72	2.36	6.37	12.45	0.59	0.00	1.33	3.36	49.1°
108	Melones	760	1907	5.03	6.49	17.48	29.00	....	....	....	......	......
109	Relief Creek	7 500	1907	11.38	2.63	29.43	43.44	....	....	....	......	......
	Tuolumne River Drainage Basin.											
110	Jamestown	1 471	1904	1.96	12.96	8.18	23.10					
			1907	7.82	5.59	17.27	30.58	1.04	1.43	1.83	10.68	49.0°
111	Tuol. Camp	3 100	1907	9.89	4.92	20.79	35.60	....	....	....	......	......
112	Crocker's (Sequoia P. O.)	4 452	1904	1.87	17.10	19.56	38.53					
			1907	13.49	5.82	27.41	46.72	....	....	....	......	......
113	Groveland	3 100	1907	9.66	3.70	15.95	29.31	....	....	....	......	......
114	Jacksonville	850	1907	5.76	4.54	13.38	23.68	....	....	....	......	......
115	Modesto	90	1904	0.33	1.67	2.15	4.15					
			1907	4.11	3.00	4.70	11.81	0.34	1.05	0.76	3.64	57.3°
116	Sonora	1 900	1904	1.79	13.82	8.63	24.24					
			1907	7.38	5.40	19.09	31.87	1.41	1.46	1.93	12.17	47.4°
	Merced River Drainage Basin.											
117	Summerdale	5 270	1904	2.60	14.96	17.09	34.66					
			1907	14.95	6.81	27.06	48.82	2.04	0.71	1.90	13.90	35.2°
118	Yosemite	3 945	1904	2.99	13.95	12.53	29.47					
			1907	11.96	3.72	20.98	36.66	2.61	2.02	1.85	13.15	38.4°
119	Merced Falls	375	1907	4.04	2.08	5.85	11.97	....	....	....	......	......
120	Elmwood	126	1904	0.57	2.19	2.17	4.93					
			1907	3.60	0.60	3.26	7.46	0.00	0.00	0.22	1.64	52.6°

TABLE 2.—RAINFALL STATIONS, ACCORDING TO ELEVATION, FOR MARCH, 1907.

ELEVATION, IN FEET.	0–500 46 STATIONS.		500–1 000 10 STATIONS.		1 000–2 000 12 STATIONS.		2 000–3 000 16 STATIONS.		3 000–4 000 15 STATIONS.		4 000–5 000 7 STATIONS.		5 000 6 STATIONS.
Basin.	Total precipitation.	Mean temperature.	Total precipitation.	Mean temperature.	Total precipitation.	Mean temperature.	Total precipitation.	Mean temperature.	Total precipitation.	Mean temperature.	Total precipitation.	Mean temperature.	Total precipitation.
Sacramento	6.20	50.3°	5.98	49.6°	19.46	48.0°	23.16	44.6°	26.42	37.6°	.....	.....	.....
Feather	9.22	51.9°	.....	.....	.....	.....	35.38	42.4°	32.68	36.9°	36.03	.....	.....
Yuba	.....	.....	19.31	.....	19.43	.....	25.70	41.6°	34.77	39.8°	42.62	32.2°	28.01
Bear	9.64	50.6°	.....	.....	.....	.....	26.15	.....	21.61	39.4°	35.50	.....	.....
American	11.97	51.1°	14.10	50.0°	16.66	47.6°	23.36	45.0°	28.46	37.7°	35.11	36.6°	30.20
Cosumnes	.....	.....	.....	.....	16.84	.....	.....	.....	.....	.....	.....	.....	.....
Mokelumne	8.25	49.0°	18.01	53.2°	14.76	48.0°	19.76	.....	24.45	41.5°	.....	.....	.....
Calaveras	9.38	....	10.20	52.7°	.....	.....	.....	.....	.....	.....	.....	.....	.....
Stanislaus	6.37	49.1°	17.48	.....	.....	.....	.....	.....	.....	.....	.....	.....	29.43
Tuolumne	4.70	57.3°	13.38	.....	18.18	48.2°	20.79	.....	15.95	.....	27.41	.....	.....
Merced	4.56	52.6°	.....	.....	.....	.....	.....	.....	20.98	38.4°	27.06	35.2°	.....
San Joaquin	4.12	51.2°	.....	.....	.....	.....	14.30	.....	.....	.....	.....	.....	.....
Stony Cr.	3.97	48.8°	6.22	.....	15.63	.....	.....	.....	.....	.....	.....	.....	.....
Cache Cr.	7.37	49.2°	.....	.....	10.63	47.6°	.....	.....	.....	.....	.....	.....	.....
Puta Cr.	11.56	52.8°	.....	.....	12.35	.....	30.46	43.4°	.....	.....	.....	.....	....
Average of all Stations	6.65	51.0°	12.09	51.6°	16.36	48.2°	25.24	43.9°	28.66	38.3°	34.25	34.7°	28.61

A comparison of the precipitation from January to March, 1907, with that for the same period and stations in 1904, shows that, for the average of all stations in the water-shed, the precipitation was greater in 1907 than in 1904, and that the difference increases from the north toward the south, but the percentage in favor of the former is quite small. An examination of Table 1 shows that the precipitation for January, 1904, was quite light compared with that of January, 1907, while the precipitation for February, 1904, was much heavier than for February, 1907. The comparison for March, however, is of most importance, as regards the floods of 1904 and 1907. Such a comparison is made in Table 4, where it is seen that, with the exception of the Sacramento River Basin, the precipitation throughout the water-shed was much greater in 1907. In the basins of the tributaries of the Sacramento River from the east, the rainfall in March, 1907, was from 20 to 41% greater than in March, 1904, while for basins on the west it is only from 2 to 3% greater. For the San Joaquin River and its tributaries the percentage is much greater, ranging from about 50 to 80 per cent. The distribution of the precipitation during the

TABLE 3.—RESULTS AT TWENTY-FOUR STATIONS, RANGING IN ALTITUDE FROM 60 TO 5 270 FEET.

Station.	Drainage.	Length of record, in years.	Precipitation in March, 1907. Month. Total, in inches.	Month. Percentage above normal.	March 17th-26th. Percentage of total.	March 17th-19th. Percentage of total.	March 17th-19th. Percentage of normal monthly.	Temperature. Month. Mean.	March 17th-20th. Mean.	March 17th-20th. Maximum daily range.
Alturas.......	Pit...........	3	4.13	....	59	35	....	35.6°	43.0°	13°
Auburn......	American...	36	16.66	224	67	33	107	47.6°	45.8°	19°
Cedarville.....	Pit..........	13	3.31	124	48	16	37	34.4°	42.0°	19°
Chico.........	Sacramento.	37	8.03	197	60	32	95	49.2°	53.1°	16°
Colusa........	Sacramento.	24	3.80	28	58	21	27	49.9°	57.2°	18°
Fresno........	San Joaquin	20	1.74	32	32	25	33	52.8°	58.1°	18°
Greenville	Feather.....	13	21.51	371	81	54	254	37.8°	40.5°	14°
Merced........	San Joaquin	33	3.68	156	74	6	15	51.7°	52.5°	25°
Milton.........	Calaveras...	17	9.27	133	53	24	56	51.2°	58.3°	13°
Nevada City..	Yuba........	15	24.62	193	69	38	111	41.6°	48.2°	19°
No. Blmfld....	Yuba........	10	28.64	254	74	14	156	39.8°	45.5°	23°
Palermo......	Feather.....	16	8.80	247	50	23	80	50.8°	55.5°	17°
Quincy........	Feather.....	12	30.15	353	85	54	244	35.8°	41.4°	16°
Red Bluff.....	Sacramento.	30	5.92	81	50	15	27	48.4°	52.2°	10°
Redding	Sacramento.	32	7.28	53	63	32	49	49.0°	52.9°	10°
Sacramento...	Sacramento.	30	7.28	148	65	37	92	50.9°	56.6°	12°
Shasta..	Sacramento.	11	14.47	167	76	33	88	48.6°	47.4°	19°
Stockton......	San Joaquin	36	6.03	164	67	30	79	51.2°	57.4°	18°
Summerdale .	Merced......	11	27.06	194	51	17	50	35.2°	39.4°	9°
Upper Lake..	Cache.......	22	10.63	238	75	45	152	49.6°	50.8°	16°
Wheatland....	Bear.........	20	9.64	247	64	33	109	50.6°	56.7°	17°
Willows	Sacramento.	28	3.63	119	55	24	53	49.1°	54.7°	17°
Yosemite.....	Merced.	3	20.98	....	63	31	....	38.4°	39.5°	17°
Georgetown..	American ...	34	29.07	209	67	33	102	42.4°	....	......
Average of above 24 Stations....................		21	12.43	179	63	31	87	45.5°	50.7°	16°
Average of 71 Stations in Basin....................			12.96	185	....	....	... .	46.5°	...	......

NOTE:—These selected stations are fairly representative as regards both temperature and precipitation. It is observed that the average of the mean temperature, March 17th–20th, is 5.2° above that for the month.

month, however, is of most vital significance. For March, 1904, the precipitation is distributed quite evenly throughout the entire month, though the intensity is noticeably greater during the equinoctial week. For March, 1907, however, not only is the total precipitation for the month considerably greater than in 1904, but its periodic occurrence in a series of storms is more pronounced. During the 10-day period centering about the equinox, the intensity was so great that about 70% of the total precipitation for the month occurred in this time, while more than 30% of it was recorded on March 17th, 18th, and 19th.

TABLE 4.—Comparison of Precipitation in Sacramento and San Joaquin Basins, for March, 1904, and March, 1907.

River basin.	Number of precipitation stations.	Precipitation.		Percentage of Difference.	
		1904.	1907.	1904.	1907.
Sacramento	17	11.34	10.13	12	....
Feather	10	16.52	23.24	....	41
Yuba	8	22.71	27.20	....	20
Bear	2	17.60	22.57	....	28
American	11	20.46	25.67	....	25
Stony	2	9.54	9.80	....	3
Cache	3	8.28	8.46	....	2
Puta	5	18.60	19.12	....	3
San Joaquin	13	3.34	4.86	....	45
Cosumnes	1	12.58	16.84	....	34
Mokelumne	7	6.56	10.20	....	56
Calaveras	2	8.12	13.15	....	62
Stanislaus	1	3.44	6.37	....	85
Tuolumne	4	9.63	17.12	....	78
Merced	3	10.60	17.10	....	61

Flood Flow of Streams.

On the following pages is recorded the daily flow of the various streams in the Sacramento and San Joaquin Basins for the 11 days, March 16th-26th, also the mean daily flow for the 4-day period, March 18th-21st, at all places where gauging stations were maintained. The figures given herein are not the final figures as they may appear in the Annual Report of the United States Geological Survey, but they will not differ materially from them. In all cases it is believed that the estimates are quite conservative, and rather inclined to be too low than too high.

Pit River.—This river drains a long, comparatively narrow, and high mountainous area in the northeastern part of the Sacramento Basin. In this area are several large reservoir sites. Those surveyed, to June, 1905, have a capacity of 6 000 000 acre-ft., but Big Valley Reservoir, above Bieber, with a capacity of 3 196 000 acre-ft., is in all probability the only one that could be utilized for flood control. The precipitation in this basin above Bieber was comparatively light, and occurred mainly as snow, so that the run-off per square mile was small.

The gauging station is about 12 miles below Bieber and about 70 miles in a direct line above the mouth of the McCloud River. The area above this gauging station is 2 950 sq. miles. Table 5 contains data on the flood flow at this station during the flood of 1907.

TABLE 5.—FLOW OF PIT RIVER, NEAR BIEBER.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.
March 16th	6.5	2 770
" 17th	12.8	17 400
" 18th	15.5	25 000
" 19th	16.4	27 500
" 20th	15.5*	25 000
" 21st	14.0*	20 800
" 22d	11.5*	13 800
" 23d	9.7	8 810
" 24th	8.5	6 110
" 25th	7.4	4 160
" 26th	7.3*	4 000

Period.	Mean daily discharge, in cubic feet per second.	Total run-off, in acre-feet.
February	4 180	232 000
March	6 940	427 000
March 18th–21st	24 600	195 000

* Estimated.

The mean rate of flow for the 24 hours when it was greatest was 9.3 cu. ft. per sec. per sq. mile, and the mean rate for the 4 consecutive days, March 18th-21st, was 8.3 cu. ft. per sec. per sq. mile. This small run-off was due to light precipitation and to the slow melting of the snow.

McCloud River.—McCloud River, the principal tributary of the Pit River, drains a long, narrow, mountainous, timbered strip of about 676 sq. miles on the north side of the Pit River Basin, including the southern and eastern slopes of Mount Shasta. Its low-water flow is remarkably large, never having been less than 1 200 cu. ft. per sec. at the gauging station in 4 years.

The gauging station is 14 miles east of Baird Spur, on the Southern Pacific Railroad, at Gregory Post-Office, and the drainage area above it is 608 sq. miles. Table 6 contains data on the flow at this station during the flood of 1907.

The mean rate of flow at this station for the 24 hours when it was greatest was 50.0 cu. ft. per sec. per sq. mile, and the mean for the 4 consecutive days, March 18th-21st, was 35.5 cu. ft. per sec. per sq. mile.

Upper Sacramento River.—The gauging station on the Upper Sacramento is in the foot-hills near Iron Canyon, 4 miles above Red Bluff, at an elevation of about 310 ft. above sea level. The drainage area above it includes 9 300 sq. miles of mountains and foot-hills. About 38% of this area is above the stations on Pit and McCloud Rivers. Table 7 contains data on the flow of the river at this place during the floods of 1907 and 1904.

TABLE 6.—FLOW OF McCLOUD RIVER, NEAR GREGORY.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.
March 16th	2.8	2 460
" 17th	4.0	4 210
" 18th	9.4	19 200
" 19th	12.0	30 400
" 20th	10.65	24 200
" 21st	7.5	12 700
" 22d	5.9	8 360
" 23d	5.5	7 400
" 24th	4.9	6 060
" 25th	4.55	5 300
" 26th	3.95	4 120

Period.	Mean daily discharge, in cubic feet per second.	Total run-off, in acre-feet.
February	5 490	305 000
March	5 990	368 000
March 18th–21st	21 600	171 000

TABLE 7.—FLOW OF UPPER SACRAMENTO RIVER, NEAR RED BLUFF.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.	Date, 1904.	Gauge height.	Discharge, in cubic feet per second.
March 16th	6.6	23 600	March 7th	16.30	77 200
" 17th	10.0	39 100	" 8th	24.40†	147 180
" 18th	21.4	118 000	" 9th	18.95	97 200
" 19th	26.05	164 000	" 10th	17.90	88 940
" 20th	28.7*	192 000	" 11th	15.80	73 700
" 21st	22.85	132 000	" 12th	14.70	66 220
" 22d	18.4	92 900	" 13th	13.30	57 400
" 23d	21.65	120 000	" 14th	15.80	73 700
" 24th	16.8	80 800	" 15th	17.25	84 050
" 25th	14.3	64 000	" 16th	18.30	92 040
" 26th	13.25	57 100			

Period.	Mean daily discharge, in cubic feet per second.	Total run-off, in acre-feet.
February, 1904	46 300	2 670 000
March, 1904	73 300	4 510 000
February, 1907	45 700	2 540 000
March, 1907	55 700	3 420 000
March 18th–21st, 1907	152 000	1 200 000

* Maximum stage, 29.4 ft.; discharge, 204 000 cu. ft. per sec. at 2 P. M.

† February 16th, the stage was 28.00 ft.; maximum stage, 31.0 ft.; discharge, 224 000 cu. ft. per sec. in the evening. On the 15th the stage was 17.4 ft. and on the 17th, 15.2 ft.

The mean rate of flow at this station, for the 24 hours when it was greatest, was 20.7 cu. ft. per sec. per sq. mile, and the mean for the 4 days, March 18th-21st, was 16.3 cu. ft. per sec. per sq. mile.

Attention is directed to the fact, shown in Table 7, that, although the flow for 4 days of the 1907 flood was greater than for any 4 consecutive days of 1904, the total flow for March, 1907, is only 76% of that for March, 1904.

By comparing the discharges and drainage areas above the gauging stations on the Pit, McCloud, and Sacramento Rivers, it is seen that, although the drainage area above the stations on the Pit and McCloud Rivers is 38% of that above the station on the Sacramento, the combined flow at these two stations is only 21% of that of the Sacramento during February, and 23% during March. This condition is due largely to the slower melting of snow during these months in the higher parts of the basin.

Feather River.—The Feather, the largest tributary of the Sacramento, derives its water from melting snow in the high Sierras, the highest point in its basin being more than 10 000 ft. above sea level. The main river is formed by the union of three streams, the North, Middle, and South Forks, above Oroville. Its principal tributaries are the Yuba and Bear Rivers, which enter it below Oroville.

TABLE 8.—FLOW OF FEATHER RIVER, AT OROVILLE.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.	Date. 1904	Discharge, in cubic feet per second.
March 16th	19.7	64 000	March 17th	52 000
" 17th	22.2	79 200	" 18th	95 000
" 18th	26.75	107 900	" 19th	88 000
" 19th	30.2*†	129 600	" 20th	82 000
" 20th	23.1*	84 900	" 21st	59 550
" 21st	20.2*	66 740	" 22d	43 350
" 22d	17.3*	49 300	" 23d	35 950
" 23d	16.5*	41 750	" 24th	29 900
" 24th	14.3*	32 650	" 25th	25 100
" 25th	13.5*	28 500	" 26th	23 250
" 26th	12.9*	25 550		

Period.	Mean daily discharge, in cubic feet per second	Total run-off, in acre feet.
February, 1904	27 800	1 600 000
March, 1904	39 500	2 430 000
February, 1907	21 500	1 190 000
March, 1907	36 000	2 210 000
March 18th-21st, 1907	97 300	770 000

* Estimated. † Maximum, about 1 A. M., 185 000 cu. ft. per sec.

The gauging station is on the main stream, in the foot-hills at Oroville. The drainage area above it is 3 640 sq. miles. Table 8 contains data on the flood flow at this station during the floods of 1907 and 1904.

The mean rate of flow for the 24 hours when it was greatest was 35.6 cu. ft. per sec. per sq. mile, and the mean for the 4 days, March 18th-21st, was 26.7 cu. ft. per sec. per sq. mile.

It is seen that, although the maximum daily discharge in 1904 is only 73% of that in 1907, the total flow for February and March is greater in 1904 than in 1907 by 29% and 10%, respectively.

The other four largest floods in the stream on record, or even recalled by the oldest inhabitants living along it, occurred in 1849, 1853, 1861, and 1881. In none of these floods, however, was the water as high at Oroville as in March, 1907. This may have been due in part or entirely to the filling of the river channel at and below Oroville with mining débris. About one-half of the Town of Oroville was flooded for 3 days. The water was about 3 ft. deep on the floor of the Union Hotel. The highway bridge and the Northern Electric Railway bridge in Oroville were swept away, and also other bridges along this stream.

The great range of river stage and its rapid fluctuations are shown by Table 9, the gauge record at Big Bend, 15 miles above Oroville.

TABLE 9.—GAUGE RECORD ON FEATHER RIVER, AT BIG BEND.*

Day.	Hour.	Gauge height.
March 14th	7 A. M.	6.9
" 15th	8 "	6.6
" 16th	8 "	6.6
" 17th	9 "	10.0
" 17th	1 P. M.	13.0
" 17th	6 "	20.0
" 18th	3 A. M.	25.0
" 18th	10 "	31.0
" 18th	4 P. M.	32.5
" 19th	1 A. M.	36.0
" 19th	10 "	34.5
" 20th	10 "	28.0
" 21st	10 "	22.0
" 22d	5 P. M.	18.0
" 25th	10 A. M.	10.0

NOTE: Low-water reading, 2 ft.

* Data furnished by Great Western Power Company, through Mr. L. J. Bevan.

Indian Creek.—Indian Creek is a tributary of the North Fork of the Feather River, and its water-shed is at a high altitude. The gauging station is about 1½ miles below the Town of Crescent Mills. The drainage area above this station is 740 sq. miles, the larger part of it being at an elevation of more than 5 000 ft.

TABLE No. 10.—FLOW OF INDIAN CREEK, NEAR CRESCENT MILLS.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.
March 16th	4.4	905
" 17th	7.1	2 570
" 18th	17.0	9 500
" 19th	19.7*	11 500
" 20th	17.9	10 100
" 21st	14.7	7 890
" 22d	10.95	5 265
" 23d	9.0	3 900
" 24th	7.8	3 060
" 25th	7.7	2 990
" 26th	7.5	2 850

Period.	Mean daily discharge, in cubic feet per second.	Total run-off, in acre-feet.
February	2 210	123 000
March	2 940	181 000
March 18th-21st	9 750	77 400

* Maximum, 20.2 ft.

The mean rate of flow for the 24 hours when it was greatest was 15.5 cu. ft. per sec. per sq. mile.

It is seen that the maximum run-off per square mile during this flood is less than one-half of that from the water-shed of the Feather River above Oroville, due to the slower melting of the snow at high altitudes.

Yuba River.—The Yuba is the largest tributary of the Feather River, entering it at Marysville, 30 miles above the junction of the Feather and Sacramento Rivers and 26 miles below Oroville. The entire area drained by it is about 1 330 sq. miles, of which 1 220 sq. miles are above the gauging station near Smartsville. The basin is comparatively long and narrow, the highest point having an elevation of 9 000 ft., which is not as great as that of the Feather River, the highest point of which is more than 10 000 ft. A large part of the basin is more than 5 000 ft. above sea level. Table 11 contains data on the flow of this stream at Smartsville during the floods of 1907 and 1904.

TABLE 11.—FLOW OF YUBA RIVER, AT SMARTSVILLE.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.	Date, 1904.	Discharge, in cubic feet per second.
March 16th.........	14.3	6 600	February 16th.....	58 000
" 17th........	24.0	56 000	" 17th.....	41 000
" 18th.........	27.9	85 000	" 18th.....	17 880
" 19th.........	29.2*	100 000	" 19th.....	12 340
" 20th.........	24.0	60 000	" 20th.....	9 350
" 21st.........	18.5	27 000	" 21st	9 350
" 22d	15.9	14 000	" 22d......	59 800
" 23d.........	16.4	16 500	" 23d......	27 660
" 24th.........	15.0	11 000	" 24th.....	59 800
" 25th.........	14.5	9 900	" 25th.....	24 080
" 26th.........	14.1	8 900		

Period.	Mean daily discharge, in cubic feet per second.	Total run-off, in acre-feet.
February, 1904................	14 900	858 000
March, 1904	15 400	947 000
February, 1907.............	14 100	783 000
March, 1907	17 300	1 060 000
March 18th-21st, 1907..........	68 000	537 000

* Maximum stage, 29.5 ft. about 2 P. M.

The mean rate of flow for the 24 hours when it was greatest was 82.0 cu. ft. per sec. per sq. mile. The maximum daily discharge of this stream was 67% greater in 1907 than in 1904. The total discharge for March, 1907, is larger than for March, 1904, but the total for February and March combined is about the same for the two years. The effect of rapid melting of snow in the middle altitudes is clearly shown here by the large run-off per square mile.

Bear River.—The Bear is the most southern tributary of the Feather River, entering it about 12 miles above the mouth. It drains an area of about 290 sq. miles, of which 263 sq. miles are above the gauging station at Van Trent, 8 miles above Wheatland. Its headwaters do not reach back to the crest of the Sierras, and, as much of its drainage basin is deforested, it is more torrential than the main stream. The greatest altitude in the basin is about 5 500 ft. Table 12 contains data on its flow during the 1907 flood.

The mean rate of flow for March 19th is 106.5 cu. ft. per sec. per sq. mile, and the mean for March 17th-20th is 75.3 cu. ft. per sec. per sq. mile.

It will be noticed that the run-off per square mile was 106.5 cu. ft. per sec. on March 19th, and 102.7 cu. ft. per sec. on February 2d.

TABLE 12.—FLOW OF BEAR RIVER, AT VAN TRENT.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.
March 16th	4.9	900
" 17th	13.95	18 200
" 18th	12.75	15 500
" 19th	17.8	28 000
" 20th	13.6	17 400
" 21st	9.6	8 400
" 22d	8.8	6 600
" 23d	13.2	16 500
" 24th	9.7	8 600
" 25th	9.4	8 000
" 26th	8.1	5 000

Period.	Mean daily discharge, in cubic feet per second.	Total run-off, in acre-feet.
February	3 460	192 000
March	5 570	342 000
March 17th–20th	19 800	157 000

NOTE: On February 2d the discharge was 27 000 cu. ft. per sec.

American River.—The American River drains an area of about 2 000 sq. miles, directly south of Bear River Basin and north of Cosumnes River Basin. It has three main forks, two of which head at an elevation of about 9 000 ft. above sea level, while the South Fork reaches back to an elevation of more than 9 600 ft. The gauging station is at Fair Oaks, and the drainage area above it is 1 910 sq. miles, a large part of which has an altitude of more than 5 000 ft. Table 13 contains data on the flow of this stream during the flood of 1907.

TABLE 13.—FLOW OF AMERICAN RIVER, AT FAIR OAKS.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.
March 16th	6.1	6 800
" 17th	13.40	33 000
" 18th	20.60	63 200
" 19th	27.6*†	93 000
" 20th	23.9*	77 000
" 21st	21.0*	65 000
" 22d	18.4*	54 000
" 23d	13.5	33 400
" 24th	13.25	32 300
" 25th	12.30	28 400
" 26th	11.50	25 000

Period.	Mean daily discharge, in cubic feet per second	Total run-off, in acre-feet.
February	14 200	789 000
March	23 200	1 430 000
March 18th–21st	74 600	594 000

* Bridges and gauges washed away. Gauge height estimated.
† Maximum stage, 30.2 ft., about 5 A. M.

The greatest mean daily discharge at this station was 48.7 cu. ft. per sec. per sq. mile, and the greatest 4-day mean was 39.1 cu. ft. per sec. per sq. mile.

Attention is called to the fact that the run-off per square mile for the American River is about 50% greater than that of the Feather River, although the percentage of each basin with an altitude exceeding 5 000 ft. is about the same.

Stony, Cache, and Puta Creeks.—These three streams are the largest tributaries of the Sacramento River from the west. They drain the eastern slope of the Coast Range, and are torrential. Stony Creek is the only one of the three that flows directly into the Sacramento River; the other two empty into Yolo Basin. The gauging station on Stony Creek is in the foot-hills, near Fruto, and the drainage area above it comprises 601 sq. miles. The station on Cache Creek is near Yolo, and the area above it is 1 230 sq. miles. Puta Creek station is at Winters, and the area above it is 805 sq. miles. Table 14 contains data on the flow of these streams during the flood of 1907.

TABLE 14.—FLOW OF STONY, CACHE, AND PUTA CREEKS.

Date, 1907.	Stony Creek.		Cache Creek.		Puta Creek.	
	Gauge.	Discharge.	Gauge.	Discharge.	Gauge.	Discharge.
March 17th	9.45	6 100	6.80	2 950	15.30	8 800
" 18th	14.25	25 000	19.45	13 500	21.60	19 800
" 19th	13.15	20 000	25.90+	19 000	23.65	24 700
" 20th	11.8	13 450	18.20	12 500	16.15	10 000
" 21st	9.8	6 810	12.65	7 820	12.35	5 460
" 22d	7.75	3 350	12.00	7 300	11.90	5 000
" 23d	11.55	12 300	20.85	14 800	26.60*	31 500
" 24th	8.7	4 760	19.30	13 400	15.60	9 200
" 25th	8.15	3 910	16.15	10 800	14.75	8 100
" 26th	7.75	3 350	12.55	7 750	11.40	4 500

Period.	Mean daily discharge, in cubic feet per second.			Total run-off, in acre-feet.		
	Stony Cr.	Cache Cr.	Puta Cr.	Stony Cr.	Cache Cr.	Puta Cr.
February	3 330	2 320	1 740	185 000	129 000	96 600
March	4 450	5 310	5 030	273 000	326 000	309 000
March 18th–21st	16 300	13 200	15 000	129 000	105 000	119 000

* Maximum, 28.15 ft., about noon. + Maximum, about 26.4 ft., during night.

The greatest daily rate of flow per square mile was, for Stony Creek, 41.6 cu. ft. per sec.; for Cache Creek, 15.5 cu. ft. per sec.; for Puta Creek, 39.1 cu. ft. per sec. The small run-off from Cache Basin is attributed to the storage and regulation effects of Clear Lake.

San Joaquin River.—Prior to the flood of March, 1907, no gauging station had been established on the San Joaquin River, because of inability to find a satisfactory section. In the fall of 1907, however, a station was established in the foot-hills near Pollasky, about 20 miles northeast of Fresno, where fair conditions obtain. The drainage area above this station is 1 640 sq. miles. In making an estimate of the run-off for March 18th-21st, a rate of 10 cu. ft. per sec. per sq. mile has been used. This rate is based on the rates in the basins to the north and south, and is believed to be quite conservative.

Mokelumne River.—Mokelumne River has a very narrow and very long drainage basin which extends eastward to the summit of the Sierras. From the low foot-hills to the junction of the three branches, about 30 miles above, its basin is a broad canyon with a minimum width of 1.1 miles. A large percentage of the upper basin ranges from 7 000 to 9 000 ft. in elevation, and several peaks are more than 10 000 ft. high.

TABLE 15.—Flow of Mokelumne River, near Clements.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.
March 16th	7.15	2 000
" 17th	13.60	8 600
" 18th	17.00	12 200
" 19th	21 00	17 000
" 20th	17.9	13 000
" 21st	15.9	11 200
" 22d	13.0	8 000
" 23d	13.3	8 300
" 24th	13.0	8 000
" 25th	11.6	6 500
" 26th	11.2	6 400

Period.	Mean daily discharge, in cubic feet per second.	Total run-off, in acre-feet.
February	2 920	162 000
March	5 320	327 000
March 18th–21st	13 350	106 000

This river drains an area of about 660 sq. miles between the American and Stanislaus Rivers, and empties into the San Joaquin through Georgiana Slough at the west end of Bouldin Island. It heads in the Sierras at an elevation of about 10 000 ft. The area above the gauging station, near Clements, is 642 sq. miles, fully two-thirds of which exceeds an altitude of 5 000 ft. Its principal tributary is Cosumnes River, which enters from the north, about 8 miles from Walnut Grove, and about 25 miles below the gauging station. Table 15 shows the flow of this stream during the flood of 1907.

The greatest run-off at this station in 24 hours during this flood was 26.5 cu. ft. per sec. per sq. mile, and the greatest 4-day mean was 20.8 cu. ft. per sec. per sq. mile.

Attention is called to the fact that the maximum daily discharge was not much larger than the mean for 4 days, showing quite clearly the effect of the configuration of the water-shed. The small run-off per square mile resulted from the fact that such a large percentage of the area is at a high altitude.

Cosumnes River.—Cosumnes River, the chief tributary of the Mokelumne, rises at an elevation of about 7 700 ft. above sea level, and drains an area of about 580 sq. miles. The discharge at Michigan Bar, 30 miles above the mouth, where the drainage area is 524 sq. miles, for the period, March 18th-22d, is given in Table 16.

TABLE 16.—FLOW OF COSUMNES RIVER, AT MICHIGAN BAR.

Date, 1907.	Discharge, in cubic feet per second.
March 18th	7 600
" 19th*	32 600
" 20th	9 300
" 21st	3 900
" 22d	3 300
March 18th–21st	Mean......13 350

* Maximum at 6 A. M.

The greatest run-off in 24 hours during this flood was 62.2 cu. ft. per sec. per sq. mile, and the greatest 4-day mean was 25.5 cu. ft. per sec. per sq. mile. The discharge for this stream has been computed from data on the cross-section and slopes of the bed and flood plane obtained after the flood. A regular gauging station has been established at Michigan Bar since the March flood.

Calaveras River.—The Calaveras is a comparatively small stream, draining an area of 490 sq. miles on the western slope of the Sierras between Mokelumne and Stanislaus Rivers. It empties into the San Joaquin River about 5 miles northwest of Stockton. Its headwaters have an elevation of 6 000 ft., which is greater than those of Bear River, but only a very small portion of the basin exceeds 4 000 ft. The data in Table 17 were obtained during the 1907 flood, near Jenny Lind, above which point the area of the drainage basin is 395 sq. miles.

TABLE 17.—FLOW OF CALAVERAS RIVER, NEAR JENNY LIND.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.
March 18th	5.4	3 800
" 19th	11.4*	26 100
" 20th	5.4	3 800
" 21st	5.0	3 300
" 22d	5.0	3 300
March 18th–21st		Mean....9 250

* Maximum stage at 8 A. M.

The maximum run-off in 24 hours was 66.2 cu. ft. per sec. per sq. mile, and the mean for the 4-day period was 23.4 cu. ft. per sec. per sq. mile.

As in the case of the Cosumnes, the discharge of this stream has been computed from meter measurements after the flood, and from slope and cross-section data. A gauging station has been established at Jenny Lind since the March flood. The small maximum daily run-off of the Mokelumne, as compared with that of the Cosumnes and Calaveras, is quite fully accounted for by the difference in the topography of the basins.

Stanislaus River.—Stanislaus River, which drains the area between Mokelumne and Tuolumne Rivers, empties into the San Joaquin River about 11 miles south of Lathrop. Its headwaters have an elevation of from 10 000 to 12 000 ft. The area drained includes about 1 050 sq. miles, of which 935 sq. miles are above the gauging station at Knights Ferry. About 500 sq. miles have an altitude of more than 5 000 ft. Table 18 contains data on the flow at this station during the flood of 1907.

The greatest run-off in 24 hours at this station during the 1907 flood was 58.1 cu. ft. per sec. per sq. mile, and in 1904 it was 32.3 cu. ft. per sec. per sq. mile. The greatest 4-day mean in 1907 was 34.5

cu. ft. per sec. per sq. mile. It is seen that the maximum run-off at this station in 1904 was only 57% of that in 1907.

TABLE 18.—FLOW OF STANISLAUS RIVER, AT KNIGHTS FERRY.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.
March 16th	8.85	2 730
" 17th	14.50	15 580
" 18th	17.35	24 100
" 19th	25.30	54 300
" 20th	19.10	31 400
" 21st	15.60	19 200
" 22d	14.55	15 740
" 23d	14.15	14 470
" 24th	13.80	13 430
" 25th	14.20	14 630
" 26th	12.75	10 420

Period.	Mean daily discharge, in cubic feet per second.	Total run-off, in acre-feet.
February	3 440	191 000
March	9 880	608 000
March 18th–21st	32 250	256 000

TABLE 19.—FLOW OF TUOLUMNE RIVER, AT LAGRANGE.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.
March 16th	6.55	3 420
" 17th	11.20	20 200
" 18th	13.50	33 400
" 19th	15.75	51 800
" 20th	13.00	30 500
" 21st	11.50	21 500
" 22d	10.50	16 700
" 23d	9.80	13 500
" 24th	10.65	17 000
" 25th	10.65	17 000
" 26th	9.30	11 500

Period.	Mean daily discharge, in cubic feet per second.	Total run-off, in acre-feet.
February	3 910	217 000
March	11 100	682 000
March 18th–21st	34 300	271 000

Tuolumne River.—Tuolumne River, which drains an area immediately south of the Stanislaus River, heads in the high peaks of the Sierras above Yosemite National Park, at an elevation of about 13 000 ft., and empties into the San Joaquin River about 10 miles west of

Modesto. The area above the gauging station at LaGrange is 1 500 sq. miles. Table 19 contains data on the flow at this station during the 1907 flood.

The greatest daily rate of run-off at this station during this flood was 34.5 cu. ft. per sec. per sq. mile, and the greatest 4-day mean rate was 22.9 cu. ft. per sec. per sq. mile.

Merced River.—Merced River drains the area between Tuolumne River and the Upper San Joaquin, and empties into the latter about 26 miles northwest of Merced. It heads at the summit of Mt. Lyell, at an elevation of 13 090 ft., and drains the southern and western slopes of this mountain, while the Tuolumne drains the northern slope. In this basin is the famous Yosemite Valley, with its great waterfalls and barren domes. The gauging station on this stream is at Merced Falls, above which the drainage area is 1 090 sq. miles. Table 20 contains data on the flow of this stream at the station during the flood of 1907.

TABLE 20.—FLOW OF MERCED RIVER, AT MERCED FALLS.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.
March 16th	10.85	2 200
" 17th	15.2	14 400
" 18th	14.8	13 000
" 19th	18.0	23 000
" 20th	16.05	17 400
" 21st	14.8	13 000
" 22d	13.95	10 200
" 23d	13.55	8 800
" 24th	16.55	19 200
" 25th	15.60	15 800
" 26th	13.65	9 200
Period.	**Mean daily discharge, in cubic feet per second.**	**Total run-off, in acre-feet.**
February	1 920	107 000
March	7 170	441 000
March 18th–21st	16 600	132 000

The greatest daily rate of flow during this flood was 21.1 cu. ft. per sec. per sq. mile, and the greatest 4-day mean was 15.7 cu. ft. per sec. per sq. mile. The small run-off per square mile arises from the fact that much of the basin has a high altitude, and that the precipitation was not as heavy as in the basins to the north.

FLOW THROUGH SACRAMENTO AND SAN JOAQUIN VALLEYS.

The rate of inflow into the Sacramento and San Joaquin Valleys from the metered mountain and foot-hill areas during this flood can be seen from the preceding pages. For ready reference, however, these rates of inflow at gauging stations for the 4-day period, March 18th-21st, are given in Table 21.

TABLE 21.—RUN-OFF FROM SACRAMENTO AND SAN JOAQUIN BASINS, IN CUBIC FEET PER SECOND, FOR MARCH 18TH–21ST, 1907.

Stream.	Place.	Drainage, in square miles.	Date, March, 1907. 18th.	19th.	20th.	21st.	Mean for March 18th–21st.
Sacramento....	Red Bluff...	9 300	118 000	164 000	192 000	132 000	151 500
Stony	Fruto..................	601	25 000	20 000	13 450	6 800	16 310
Feather	Oroville...............	3 640	107 900	129 600	84 900	66 740	97 290
Yuba..........	Smartsville..........	1 220	85 000	100 000	60 000	27 000	68 000
Bear...........	Van Trent...........	263	15 500	28 000	17 400	8 400	17 300
American	Fair Oaks...........	1 910	63 200	93 000	77 000	65 000	74 600
Cache..........	Yolo...................	1 230	13 500	19 000	12 500	7 820	13 200
Puta...., ...	Winters	805	19 800	24 700	10 000	5 460	15 000
Unmetered mountain and foot-hills.		3 907	....	....	....	....	76 000*
Sacramento Valley..................		4 250	....	....	....	....	25 500‡
Total, Sacramento Basin......		27 126	....	....	....	....	554 700
Cosumnes.....	Michigan Bar........	524	7 600	32 600	9 300	3 900	13 350
Mokelumne ...	Clements............	642	12 200	17 000	13 000	11 200	13 350
Calaveras	Jenny Lind..........	395	3 800	26 100	3 800	3 300	9 250
Stanislaus.....	Knights Ferry......	935	24 100	54 300	31 400	19 200	32 250
Tuolumne.....	LaGrange.	1 500	33 400	51 800	30 500	21 500	34 300
Merced.	Merced Falls........	1 090	13 000	23 000	17 400	13 000	16 600
San Joaquin...	Pollasky............	1 640	....	....	....	....	16 400‖
Unmetered mountain and foot-hills.		5 656	....	....	....	....	67 900†
San Joaquin Valley....		5 890	....	....	....	....	23 560§
Total, San Joaquin Basin......		18 272	....	....	....	....	226 960

* Run-off per square mile assumed as 50% of precipitation for period, March 17th–20th, or 20 cu. ft. per sec.
† Run-off per square mile assumed as 50% of precipitation for period, March 17th–20th, or 12 cu. ft. per sec.
‡ Run-off per square mile assumed as 40% of rainfall for period, March 17th–20th, or 6 cu. ft. per sec.
§ Run-off per square mile assumed as 40% of rainfall for period, March 17th–20th, or 4 cu. ft. per sec.
‖ Run-off per square mile assumed as 10 cu. ft. per sec.

From Table 21 it is seen that the mean rate of run-off from the metered area of the Sacramento Basin (83% of all mountains and foot-hills) for the 4-day period, March 18th-21st, was about 453 000 cu. ft. per sec. The estimated run-off for this period was 76 000 cu. ft. per sec. from the unmetered mountains and foot-hills, and 25 500 cu. ft. per sec. from Sacramento Valley, making a mean rate of run-off from the Sacramento Basin of about 555 000 cu. ft. per sec. for 4 consecutive days.

It is not possible to trace the movement of this water through the valley, on account of overflow into flood basins and breaks in the levee system. The levees failed at many places on both sides of the Sacramento River, and also on some of its tributaries, and it is impossible to compute the flow through any of these breaks. Such an estimate, if correctly made, would have practically no value, as it would give little idea of the distribution of flow through the valley during any other flood when failure of levees occurred at other places.

While an estimate of the volume passing specified places in the valley at a given time cannot be made, the points where large volumes left the channel and returned to it again or crossed it can be indicated, as well as the time of failure of important levees. On the evening of March 20th the water was overtopping the levees for almost the entire distance between Princeton and Jacinto, and also above and below Colusa. On March 21st, eleven breaks in the levees occurred between Colusa and Grimes, and during that night several breaks occurred in the levees on the east side of Sacramento River between Clarksburg and Courtland, allowing water from the Sacramento to pass into Mokelumne River and thence into the San Joaquin. On March 22d several other breaks occurred in Colusa County, and also in the Island District, where large areas of reclaimed land were submerged. On March 23d the levees of Ryer, Tyler, Brannan, Andrus, and Bouldin Islands and the Lisbon District failed, flooding 65 000 acres of land. Besides the failures already mentioned, there were numerous others of more or less seriousness in different places in the Sacramento Valley.

At Knights Landing, on March 21st, the Sacramento was 1 ft. higher than recorded at any previous time. Below this point, a large part of the water from the Feather River was flowing across the Sacramento Channel into Yolo Basin. Through the Kripp crevasse of February 8th, opposite the City of Sacramento, a large part of the waters of the Sacramento and American Rivers also passed into Yolo Basin, and the water level of this basin was several feet higher than ever known before. On February 24th the Sacramento at Rio Vista reached its greatest height during the flood, being 3 ft. higher than indicated by previous records. The failure of the levees of Brannan, Twitchell, and Andrus Islands, near the mouth of Cache Slough, permitted a part of the water of Yolo Basin to flow across the Sacramento Channel into the San Joaquin River, submerging large areas in the San Joaquin Delta. In all, it is estimated that about 300 000 acres of reclaimed land were submerged during this flood. Below the City of Sacramento, the only reclaimed districts having levees that withstood the high waters are: Reclamation District No. 744; Merritt Island, Grand Island, and Randall Island Reclamation Districts; Geo. W. Locke, private reclamation; Reclamation District No. 545; Sutter and Sherman Islands; and the northern portion of Union Island.

Referring again to Table 21, it is seen that, in all the streams of the San Joaquin Basin, the greatest rate of flow occurred on March 19th. On this date the mean rate of run-off from the metered area (41% of all mountains and foot-hills) was about 205 000 cu. ft. per sec. The rate from the unmetered area must have been at least 84 000 cu. ft. per sec. from mountains and foot-hills and 24 000 cu. ft. per sec. from the valley, making a maximum run-off of about 313 000 cu. ft. per sec. from the San Joaquin Basin. The mean rate for 4 days, March 18th-21st, was about 227 000 cu. ft. per sec. It is impossible to indicate the volume of flow at different points in this valley owing to the failure of levees on both the San Joaquin and Sacramento Rivers, and the passage of a large volume from the latter into the former, producing back-water and retardation of flow.

It is also seen from Table 21 that the mean flow from the mountains and foot-hills of the Sacramento and San Joaquin Basins combined, for the 4 days, March 18th-21st, was about 732 000 cu. ft. per sec. It is seen, too, that the mean rate of discharge into Suisun Bay for these 4 days, if storage in the valleys had not been permitted, would have been about 782 000 cu. ft. per sec., a volume for these 4 days of 6 200 000 acre-ft., or 9 690 mile-ft., enough to cover both basins to a depth of 2.56 in., if spread over them evenly.

Table 22 shows the run-off, expressed as depth, in inches, over the drainage basin, together with the precipitation for the March flood. Of course, there is the very regrettable condition of too few and poorly placed precipitation stations, but it is believed that the records here given are quite representative for the different basins. This table gives some idea of the effects of altitude and of melting snow in the various drainage areas.

TABLE 22.—RUN-OFF, AS DEPTH, IN INCHES.

Stream.	Place of gauging.	Drainage area above station.	Altitude of source, in feet.	Run-off per Square Mile, March, 1907. Maximum, in cubic feet per second.	Mean for March 18th-21st. Cubic feet per second.	Depth, in inches.	Precipitation, in inches.	Percentage of precipitation.
Pit	Bieber	2 950	9 900	9.3	8.3	1.23	....	....
McCloud	Gregory	608	14 400	50.0	35.5	5.28	....	....
Sacramento	Red Bluff	9 300	14 400	20.7	16.3	2.43	6.56	37
Feather	Oroville	3 640	10 000	35.6	26.7	3.97	10.40	29
Indian Cr.	Crescent Mills	740	7 000	15.5	13.2	1.96	10.00	20
Yuba	Smartsville	1 220	9 000	82.0	55.7	8.29	10.33	80
Bear	Van Trent	263	5 500	106.5	75.3	11.30	8.13	139
American	Fair Oaks	1 910	9 000	48.7	39.1	5.81	8.63	67
Stony Cr.	Fruto	601	....	41.6	27.1	4.03	5.27	76
Cache Cr.	Yolo	1 230	....	15.5	9.3	1.38	5.00	28
Puta Cr.	Winters	805	....	39.1	18.6	2.77	5.10	54
Cosumnes	Michigan Bar	524	7 700	62.2	25.5	3.80	7.50	51
Mokelumne	Clements	642	10 000	26.5	20.8	3.08	6.42	48
Calaveras	Jenny Lind	395	6 000	66.2	23.4	3.48	5.06	69
Stanislaus	Knights Ferry	935	11 500	58.1	34.5	5.14	6.26	82
Tuolumne	LaGrange	1 500	13 000	34.5	22.9	3.40	6.65	51
Merced	Merced Falls	1 090	13 000	21.1	15.7	2.34	5.92	40
San Joaquin	Pollasky	1 640	13 000	(Est.)	10.0	1.49	5.00	30

RATE OF FLOW IN SACRAMENTO VALLEY.

It will be instructive to compute the probable rate of flow of the Sacramento River during this flood at the four places where it receives large volumes of water from tributaries, namely, just below the mouths of Stony Creek, Feather and American Rivers and Cache Slough, taking into account the time required for the water to pass from the gauging stations to the Sacramento and the time to pass between the above-mentioned places. No great degree of refinement will be attempted, as the data will not warrant it.

As a flood wave travels down a channel there is a gradual diminution of its height, due to the filling of the channel and the flattening of the wave. Such diminution would have been small for this flood, and is neglected in the computations, for the following reasons:

(1).—The flood wave was a long one, the water at some of the stations continuing to rise for 4 days;

(2).—The streams had reached a comparatively high stage on March 17th, and consequently their channels were from more than half to two-thirds full at the date when the computations begin;

(3).—The rates of flow computed at gauging stations are 24-hour means, not maxima for a few hours.

It can be shown that the speed of a flood wave, M, in a stream channel, is given by the equation, $\frac{d\,Q}{d\,h} = M\,W$, in which $d\,Q$ is the increment of discharge corresponding to the increment of stage, $d\,h$, and W is the channel width. The value of M has been computed at each gauging station for intervals of 1 ft. in gauge height during the flood stages, and a mean value obtained for the distance, in hours, from the gauging station to places along the Sacramento River. These results are given in Table 23:

TABLE 23.—Data on Rate of Progress of Flood Wave, in Streams, if Water Were Confined in Channels.

Place to place.	Distance, in miles.	Rate of travel, in miles per hour.	Time of travel, in hours.
Gauging Station, Sacramento River to mouth of Stony Creek........	40	9	5
Gauging Station, Stony Creek to mouth of Stony Creek...............	35	6	6
Gauging Station, Feather River to mouth of Feather River..........	60	7	9
Gauging Station, Yuba River to mouth of Feather River............	50	8	6
Gauging Station, Bear River to mouth of Feather River.............	15	5	3
Mouth of Stony Creek to mouth of Feather River..	100	7	14
Gauging Station, American River to mouth of American River..,....	15	5	3
Mouth of Feather River to mouth of American River.................	20	7	3
Mouth of American River to mouth of Cache Slough.................	46	7	7
Gauging Station, Cache Creek to mouth of Cache Slough..........	45	4	11
Gauging Station, Puta Creek to mouth of Cache Slough.	45	4	11

Note: The rate of travel for flood waves, as given above, is the mean of the computed rates on each of the days, March 17th-21st, reduced, in most instances, by a considerable percentage.

A study of the daily rate of discharge of the streams in the Sacramento Basin, for March 18th-21st, Table 21, shows that the discharge at places along the Sacramento River was undoubtedly at a maximum when the crest of the wave from the Feather River reached them. This wave crested at Oroville about 1 A. M., March 19th. As Oroville is about 9 hours above the mouth of Feather River, the crest would reach the Sacramento River at about 10 A. M., March 19th, with a discharge of about 258 000 cu. ft. per sec., including the Yuba and

Bear Rivers. This amount, combined with the flow in the Sacramento at that time, would give the maximum discharge just below the mouth of the Feather River. The flow in the Sacramento at this time, however, was the flow at the gauging station above, about 19 hours before, combined with the flow at the gauging station on Stony Creek, about 20 hours before, or the flow of the two at, say, 2 P. M., March 18th. This flow was 143 000 cu. ft. per sec., which, added to the 258 000 cu. ft. per sec. from the Feather River, would give a discharge of 401 000 cu. ft. per sec. in the Sacramento. This volume would reach the mouth of American River 3 hours later, and be augmented by 93 000 cu. ft. per sec. passing the gauging station 3 hours before, so that the maximum discharge in the Sacramento below the mouth of the American River would be about 494 000 cu. ft. per sec., and would occur at about 1 P. M., March 19th. This volume would reach the mouth of Cache Slough at about 8 P. M., March 19th, to be increased by the flow of the Cache and Puta Creeks at the gauging stations 11 hours before, which amounted to about 44 000 cu. ft. per sec. Below the mouth of Cache Slough, therefore, the discharge would have been about 538 000 cu. ft. per sec. It is to be noted that the maximum flow in the Sacramento below the mouth of Stony Creek was about 205 000 cu. ft. per sec., and did not occur until some time on March 20th.

The figures just given do not include the unmetered flow of 76 000 cu. ft. per sec. from the mountains and hills below the metered basins, nor the 25 500 cu. ft. per sec. from the valley. It is evident that, unless stored in the flood basins, it must have appeared in the Sacramento below Cache Slough. It is impossible to compute the increase in discharge at the different places on the Sacramento River due to these two rates of inflow, because it is not known at what points all these waters were delivered; but it is quite clear that there must have been a very decided increase above the mouth of Stony Creek from each side of the river. On the east side there are 1 600 sq. miles of mountains and foot-hills lying between the Feather and Upper Sacramento Basins, which are drained by numerous creeks, the most important of which are Mill and Deer Creeks, the headwaters of which come from Lassen Peak, more than 10 000 ft. in altitude. Several of the stations reporting the greatest precipitation in March, 1907, are in this area or very near it. Taking into consideration its position between two basins in which the rate of run-off is known, together with its heavy precipitation and generally lower altitude, it is believed that the mean rate of run-off may be safely placed at 25 cu. ft. per sec. per sq. mile for the period, March 18th-21st. This means 40 000 cu. ft. per sec. from this side. On the west, above the Stony Creek Basin, are 1 080 sq. miles of mountains and foot-hills, for which it is safe to put the run-off at 15 cu. ft. per sec. per sq. mile, or a mean of 16 000 cu. ft. per sec. for March 18th-21st. This would mean an increase in the discharge below Stony Creek of about 56 000 cu. ft. per sec.

A considerable area of mountains and foot-hills between the Feather and Bear Basins must have contributed a large volume to the Sacramento through the Feather River, so that, all told, the maximum discharge below the mouth of the Feather River was probably at least 65 000 cu. ft. per sec. greater than that computed above. As the rates of run-off for the unmetered area of mountains and valley are 4-day means, the maximum discharge below Cache Slough must have been about 640 000 cu. ft. per sec. This maximum, however, is only 15% greater than the 4-day mean flow of 555 000 cu. ft. per sec. for March 18th-21st.

It will be noticed that the maximum discharge just below the mouth of Cache Slough would probably occur at 8 P. M., March 19th, if the water were confined in channels. But the maximum stage at Rio Vista, a few miles below the mouth of this slough, actually occurred at 11 P. M., March 23d. Overflow and storage in the flood basins, therefore, delayed the arrival of the flood crest at Cache Slough about 4 days.

Table 24 is a comparison of maximum rates of flow of the Sacramento River during this flood with those assumed by the 1904 Engineering Commission, provided that the total run-off is confined between the levees and not allowed to collect in the flood basins.

TABLE 24.

Place.	Maximum rate assumed by 1904 Engineering Commission. Cubic feet per second.	Maximum rate computed from March, 1907, flood. Cubic feet per second.
Below mouth, Stony Creek.....	180 000	261 000
" " Feather River..	190 000	466 000
" " American River.	230 000	559 000
" " Cache Slough...	250 000	640 000

These computed rates are from 45 to 156% larger than the assumed rates.

Profile of Flood Wave in Sacramento River.

Fig. 2 is a profile of the flood wave in Sacramento River during March, 1907. This profile merely shows the greatest elevation of the flood plane above mean sea level at different points along the course of the river. In other words, the maximum height attained by the flood at various points is platted with reference to the distance from the mouth of the river and the elevation above mean sea level. An inspection of this profile shows that the mean gradient of the flood plane, in feet per mile, between observed points, decreases quite rapidly from Red Bluff toward the mouth of the river, actually changing sign below Walnut Grove. This gradient varies from — 2.41 between Red Bluff and Munroeville, near the mouth of Stony Creek, to + 0.01 below Walnut Grove. Such a reversal of slope would seem to indicate a constricted condition of the channel near the mouth of the river.

A profile of the flood wave of 1905, made under the direction of the Commissioner of Public Works of California, is also shown on Fig. 2 for the purpose of comparison. This profile may be considered as typical of the usual flood wave in the spring of each year.

Losses Due to the Flood of March, 1907.

The losses resulting from this flood consisted mainly in the destruction of the crops then growing on about 300 000 acres of land completely inundated, together with the damage done to a portion of the prospective yield for the season of 1907. In addition to this, many miles of costly levees had to be rebuilt and many miles more extensively repaired on account of overtopping and wind action. The railroads suffered heavily, in bridges and culverts washed out, in injury to miles of roadbed, and in loss of traffic. The line from Marysville to Knights Landing was closed from March 19th to May 13th. Among the larger bridges swept away or badly damaged were the highway and the Northern Electric Railway bridges across the Feather River at Oroville, the highway bridge across the American River at Fair Oaks, the highway bridge on the Mokelumne River near Clements, and the bridge on the Cosumnes River at Bridge House. Three costly dredges for mining gold-bearing gravel in the Feather River near Oroville were

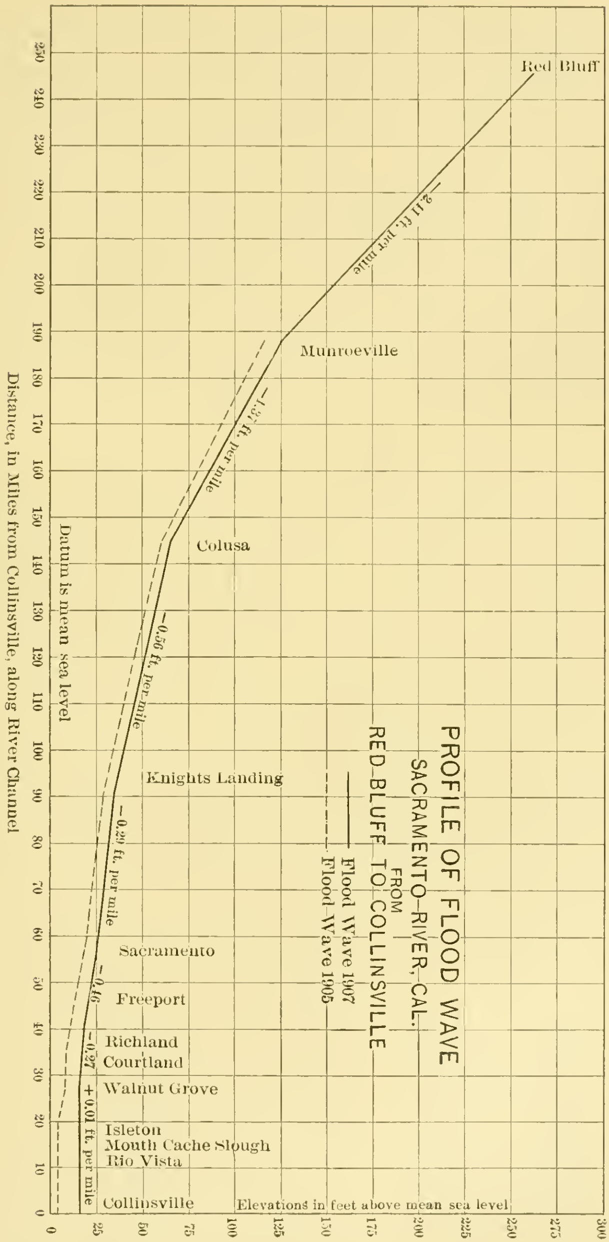
PROFILE OF FLOOD WAVE
SACRAMENTO RIVER, CAL.
FROM
RED BLUFF TO COLLINSVILLE
Flood Wave 1907
Flood Wave 1905
Red Bluff
2.11 ft. per mile
Munroeville
1.37 ft. per mile
Colusa
0.56 ft. per mile
Knights Landing
0.29 ft. per mile
Sacramento
0.46
Freeport
Richland
Courtland
0.27
Walnut Grove
+0.01 ft. per mile
Isleton
Mouth Cache Slough
Rio Vista
Collinsville
Elevations in feet above mean sea level
Datum is mean sea level
Distance, in Miles from Collinsville, along River Channel

Fig. 2.

destroyed. Many towns and villages were partially inundated, subjecting the inhabitants to serious inconvenience at the time and to heavy expense in repairs later. The greater part of Stockton was flooded for nearly a week, because of the failure of the levees along Mormon Slough and Jackson Creek. About half of Oroville was flooded for three days, and one hundred and twenty-five families were driven from their homes. The restraining dam on the Yuba River, 14 miles above the mouth, known as Barrier No. 1,* was destroyed on the night of March 18th. This dam was built to hold back the mining débris in the channel above. With its destruction, practically all the débris restrained by it (probably amounting to more than 1 000 000 cu. yd.), was transferred to the channel below. It is estimated that the total damage resulting from this flood exceeded $5 000 000.

Effect of Mining Débris on Floods.

From 1849 to 1880 enormous quantities of débris—sand, gravel, and cobbles, the tailings from hydraulic mining—were deposited in the upper course of several of the streams on the eastern slope of the Sacramento Basin. The volume of this débris in the Yuba River alone has been variously estimated at from 71 000 000 to 700 000 000 cu. yd. At the mouth of the river, near Marysville, it has a depth of 7½ ft.; at Dugnens Point, 11 miles above the mouth, it has a depth of 26 ft., and at The Narrows, 18 miles above the mouth, it has a depth of 84 ft. The gradual elevation of the flood plane at Marysville, due to the accumulation of débris in the channel at this place, is shown by the maximum gauge readings at Marysville (Table 26). The zero of the gauge is the elevation of low water in 1872.

TABLE 26.—Maximum Gauge Readings at Marysville.

Date.	Gauge height.†
January 11, 1862	11 ft. 6 in.
March 6, 1869	15 " 11 "
January 19, 1875	15 " 2 "
April 22, 1880	16 " 2 "
February 24, 1881	18 " 2 "
December 23, 1884	17 " 1 "
January 18, 1896	18 " 5 "
March 25, 1899	18 " 5 "
February 21, 1901	19 " 0 "
February 25, 1904	20 " 0 "
January 19, 1906	21 " 8 "
February 2, 1907	21 " 3 "
March 19, 1907	23 " 4 "

* The failure of this structure is described in *Engineering News*, Aug. 8th, 1907.

† Data furnished by W. T. Ellis, Levee Commissioner.

The low-water reading on this gauge, in the summer of 1906, was 9.0 ft. The flood of 1907, however, changed the low-water channel from the right to the left side of the river, so that in August the elevation of the water surface could not be read at all, the débris around the gauge being 3 ft. higher than the water.

The failure of Barrier Dam No. 1, on the Yuba River 14 miles above the mouth, liberated about 1 300 000 cu. yd. of débris which was deposited in the bed of the stream at varying distances below the dam, depending upon the size of the material. The deposition of this enormous volume of material in the stream bed, and the gradual elevation of the flood plane due to it, require frequent raising and widening of the levees along the river. Such a condition is fraught with growing peril to the valley land and to all interests adjoining the river.

Effect of Storage Reservoirs on Floods.

Any rational system of reclamation for the overflow lands in the Sacramento and San Joaquin Valleys must make provision for passing the peak of the floods rapidly to Suisun Bay. The volume of flood water to be passed in Sacramento Valley, as determined by actual gaugings of the flood of March, 1907, largely exceeds all estimates previously used as a basis for the computation of proper channel capacity to carry safely the flood waters of the Sacramento River. Indeed, it may be that the task of rectification and enlargement of channel necessary to pass such floods as that of March, 1907, is so great as to make it economically impossible. In such event, some auxiliary system of flood control would have to be devised. Probably no more effective and easily executed auxiliary system could be found than that of large, regulating storage reservoirs in the mountains. Such reservoirs could be utilized to store water during floods, thereby reducing the peak of the flood in the valley sufficiently to allow the main channel to carry it safely to Suisun Bay.

The United States Reclamation Service has located the principal reservoir sites in the Sacramento Basin, and has made surveys to determine the capacity and probable cost of most of them. Of the reservoirs surveyed to date, four are in Stony Creek Basin, with a total capacity of 124 100 acre-ft.; two are in Cache Creek Basin, with a total capacity of 176 500 acre-ft.; two are in Puta Creek Basin, with a total capacity of 318 000 acre-ft.; seven are in Feather River Basin, with a total capacity of 775 600 acre-ft.; four are in Pit River Basin, one of which has a capacity of 3 196 000 acre-ft.; and one is on the Upper Sacramento River at Iron Canyon, with a capacity of 226 900 acre-ft. In the San Joaquin Basin no reservoir sites have been located and surveyed yet, although it is probable that the area contains some good ones.

TABLE 27.—RESERVOIR DATA.

Name of Reservoir.	Capacity, in acre-feet.	Drainage, in square miles.	Volume Available for Storage, in Acre-Feet.*				
			Mar. 18th.	Mar. 19th.	Mar. 20th.	Mar. 21st.	Mar. 18th–21st.
Stony Creek Basin.							
East Park.....	26 000	114	9 390	7 520	5 060	2 560	24 530
Stony Ford........	40 000	110	9 060	7 250	4 880	2 470	23 660
Briscoe........................	14 400	58	4 770	3 820	2 570	1 300	12 460
Mill Site......................	43 700	323	26 600	21 300	14 300	7 250	69 450
Cache Creek Basin.							
Clear Lake....................	100 000	486	10 600	14 900	9 790	6 120	41 410
Little Indian....	76 500	123	2 680	3 770	2 480	1 550	10 480
Below reservoirs and above gauging station.............	....	621	13 500	24 600	12 500	7 820	52 820
Puta Creek Basin.							
Guenoc	188 000	148	7 210	8 990	3 460	1 990	21 830
Puta Creek....................	130 000	603	29 400	36 600	14 800	8 110	88 910
Below reservoirs and above gauging station.............	....	54	1 980	2 460	998	546	5 984
Feather River Basin.							
Grizzly Valley................	61 800	44	2 580	3 100	2 040	1 600	9 320
Mohawk Valley...............	12 600	682	40 000	48 100	31 700	24 800	144 600
Big Meadow...................	500 000	506	29 700	35 600	23 500	18 400	107 200
Buck's Valley and Spanish Ranch......................	46 270	29	1 700	2 040	1 350	1 050	6 140
American Valley..............	86 100	172	10 100	12 100	8 000	6 260	36 460
Indian Valley.................	68 800	1 010	59 300	71 200	47 000	36 700	214 200
Below reservoirs and above gauging station.............	....	1 200	70 400	84 500	55 800	43 6[illegible]0	254 300
Pit River Basin.							
Big Valley....................	3 196 000	2 950	49 600	54 500	49 600	41 200	194 900
Sacramento River.							
Iron Canyon..................	226 900	6 350	184 400	270 500	331 400	220 800	1 007 100

* The daily run-off per square mile is assumed to be constant over the basin above the gauging station.

In Table 27 are shown the reservoir sites in the Sacramento Basin which could be used for flood control, together with the drainage area tributary to each and its capacity in acre-feet. Assuming the run-off per square mile to be constant in any particular basin, the quantity of water available for storage at each reservoir is given for each of the days, March 18th-21st, and also the total for the 4 days. It will be noted that some of these reservoirs would be only partially filled by the flood flow of March 18th-21st, while others would store but a small percentage of the run-off for this period.

A study of Table 27 will show that the four reservoirs in Stony Creek Basin would have stored the run-off from 481 sq. miles, or 80% of the area above the gauging station, and would have reduced the maximum daily flow from 25 000 to 5 000 cu. ft. per sec. The two reservoirs in Cache Creek Basin would have stored the flow from 609 sq. miles, or 50% of the area above the gauging station, and would have reduced the maximum daily flow from 19 000 to 9 500 cu. ft. per sec. The two reservoirs in Puta Creek Basin would have stored the flow from 751 sq. miles, or 93% of the area above the gauging station, and would have reduced the maximum daily flow from 24 700 to 1 700 cu. ft. per sec.

The seven reservoirs in Feather River Basin would have stored the flow from about 1 134 sq. miles, or 31% of the area above the gauging station at Oroville, leaving 2 506 sq. miles uncontrolled. Of this uncontrolled area, 623 sq. miles are above Mohawk Valley Reservoir, 683 sq. miles are above Indian Valley Reservoir, and 1 200 sq. miles are below the reservoirs and above the gauging station. This storage would have reduced the daily flow at Oroville as follows:

From 107 900 to 74 300 cu. ft. per sec. on March 18th; from 129 600 to 89 200 cu. ft. per sec. on March 19th; from 84 900 to 58 500 cu. ft. per sec. on March 20th; and from 66 740 to 45 900 cu. ft. per sec. on March 21st. Big Valley Reservoir, on Pit River, would have stored the entire flow at that place and reduced the daily flow of the Sacramento River at Red Bluff about 25 000 cu. ft. per sec. The storage at Iron Canyon, together with that on Pit River, would have reduced the greatest daily flow of the Sacramento River at Red Bluff from 192 000 to 106 000 cu. ft. per sec.

The combined effect of all these reservoirs in operation at the same time would have been to reduce the maximum flow in the Sacramento River by about 86 000 cu. ft. per sec. above the mouth of Stony Creek, 106 000 cu. ft. per sec. above the mouth of the Feather River, and 179 000 cu. ft. per sec. below the mouth of Cache Slough.

It would seem that the ultimate solution of the flood problem in the lower portions of the Sacramento Valley is closely interwoven with the reclamation of the higher portions by irrigation. Reservoirs which would impound flood waters and reduce the peak of floods, so as to save the lowlands from overflow in the early spring, would serve later as storage reservoirs from which to draw for irrigation purposes. The flood problem in this valley is indeed a very serious one, and merits the most careful and thoughtful consideration.

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PAPERS AND DISCUSSIONS.

THE REINFORCED CONCRETE WORK OF THE McGRAW BUILDING.

Discussion.*

By Messrs. T. L. Condron and F. F. Sinks, E. W. Stern, L. J. Mensch, and P. E. Stevens.

T. L. Condron and F. F. Sinks, Members, Am. Soc. C. E. (by letter).—No good reason can be offered for not exercising the same common sense in designing reinforced concrete structures as that expected and demanded in designing steel or timber structures. Too much is heard regarding "systems" of reinforced concrete, and too little regarding the simple application of the well-known laws relating to the strength of materials and the distribution of stresses in such structures. It is not many years ago that iron bridges were built according to one or another special, and generally patented, type. To-day the "patented bridges" are limited to draw bridges, which, after all, are machines as well as structures. It is doubtless true that, in a large measure, the present wide use of reinforced concrete is due to the energetic promotion of various so-called "systems," together with the equally energetic promotion of concrete construction by makers of various forms of reinforcing materials. While, in some cases, capable and conscientious engineers have done splendid work in developing new and better designs for reinforced concrete as a substitute for designs of steel or masonry structures, in other cases, less capable, and in some instances ignorant, men have produced "systems" which would be ridiculous if they were not dangerous. Messrs. Condron and Sinks.

*Continued from January, 1908, *Proceedings*.

Messrs. Condron and Sinks.

In designing reinforced concrete, the writers have endeavored to follow the same methods of analysis of stresses and proportioning of parts as they use in designing steel structures. They have studied carefully all the experimental and research work done by the leading technical schools and universities, and believe that more can be gained by such study than by simply developing any refined theoretical analysis of the strength of concrete reinforced with steel.

The writers present herewith illustrations of what they believe to be rational designs of reinforced concrete construction. Care has been taken to have these designs free from every unnecessary complication, the whole aim being to gain great strength and everlasting durability with the most simple construction possible.

The author's description of the McGraw Building is of especial interest, as there are several features in its design which are similar to those used by the writers; therefore, they present the following description of one building, and some notes regarding two others, designed by them.

The Manufacturers' Furniture Exchange Building, in Chicago, the reinforced concrete features of which were designed by the writers, as Consulting Engineers for the Architect, Mr. William Earnest Walker, was designed in the spring of 1906 and completed near the close of that year. In the McGraw Building, as well as in the buildings designed by the writers, the columns have been reinforced with latticed steel angles. As far as the writers are aware, the first reinforced concrete building in which columns of this form were used was the Watson Building, in Chicago, built in 1905, for which Messrs. Huehl and Schmidt were the Architects. The writers' original recommendation for the columns of this building was that the angles be latticed, but they were actually built with horizontal tie-plates, as shown by Fig. 1, Plate XXIV. At the time this photograph was taken the view, Fig. 2, Plate XXIV, was also taken on the first floor, where concreting was going on, the forms for the floors above being supported so that they did not interfere with the placing of concrete on this floor.

The general plans for the Manufacturers' Furniture Exchange Building were completed in June, 1906, and the contracts were let about July 1st. The building is near the business center of Chicago, and has a frontage of 70 ft. on Wabash Avenue, running back 170 ft. on Fourteenth Street to an alley. The general appearance of the building is shown by Fig. 1, Plate XXV. It is an eight-story and basement building, designed for furniture show rooms, warehouse purposes, or light manufacturing. The floors are designed to carry live loads of 150 lb. per sq. ft. on the lower floors, and 100 lb. per sq. ft. on the upper floors. Fig. 2 is a plan and Fig. 3 a cross-section of the building, showing the general arrangement of the columns and beams.

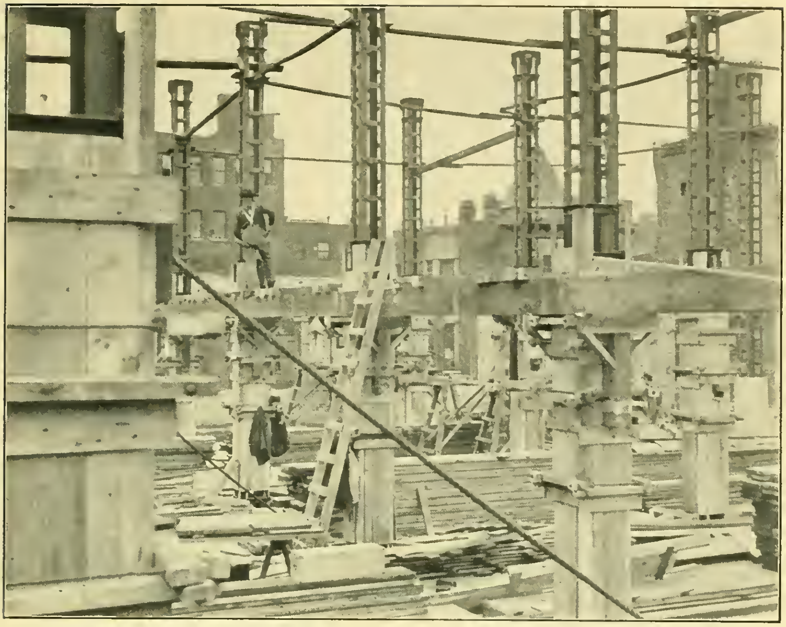


Fig. 1.—Second and Third Story Column Reinforcing, Watson Building.

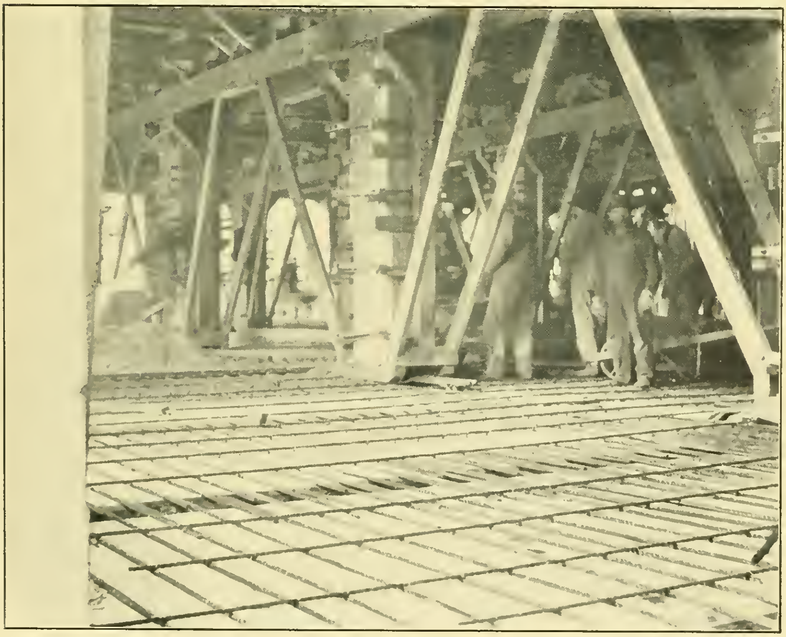
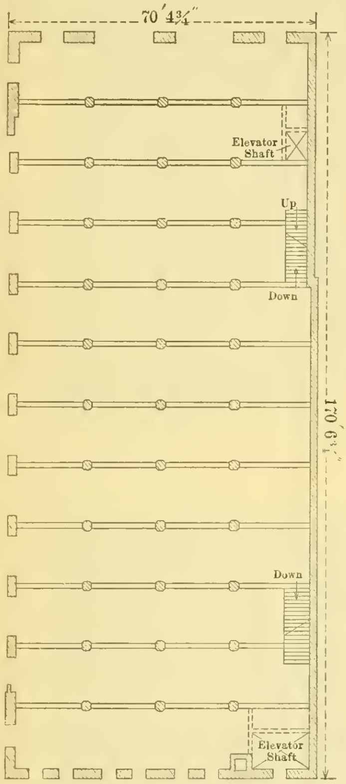


Fig. 2.—Concreting on First Floor, Watson Building.

Messrs. Condron and Sinks

On the first floor a bulkhead is carried around on two sides, supporting platforms for the show windows and permitting half windows for lighting the basement. The second to eighth floors, inclusive, are exactly alike. The roof is of reinforced concrete, and has two rows of saw-tooth skylights.



PLAN OF 2ND TO 8TH FLOOR

Fig. 2.

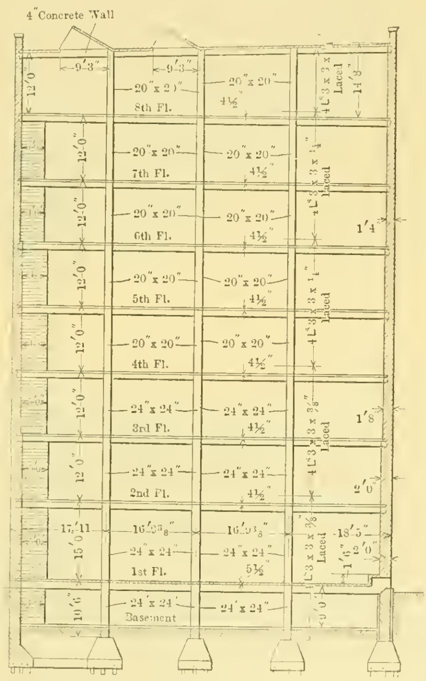
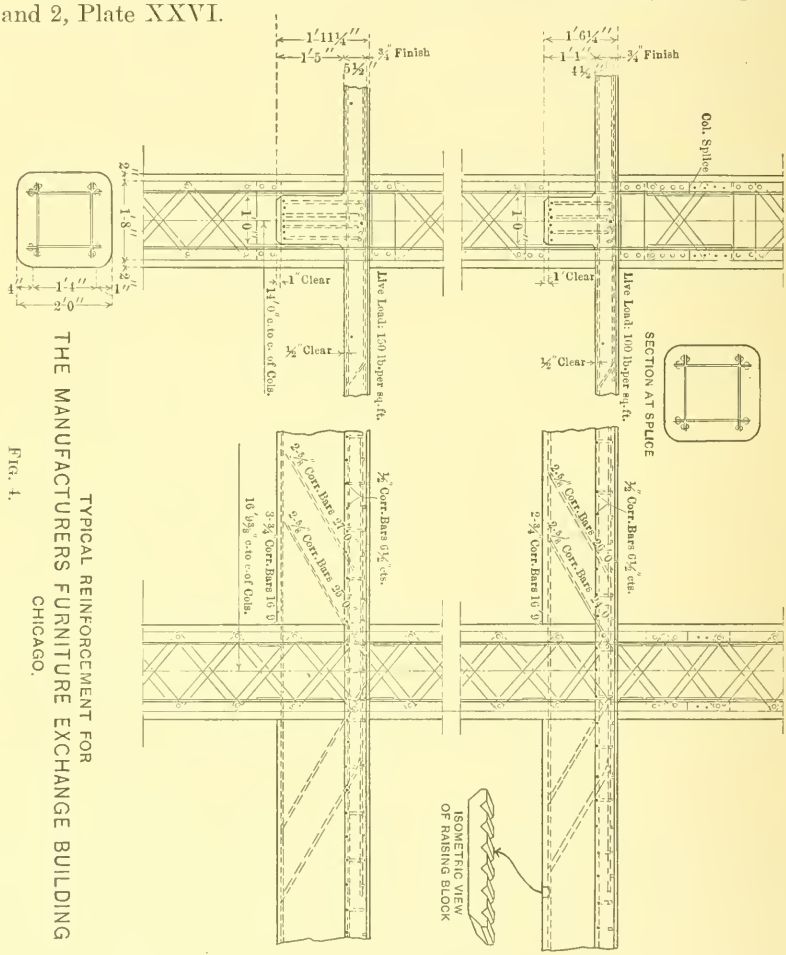


Fig. 3.

In designing the columns, the ratio of the moduli of elasticity of steel and concrete was assumed as 15 to 1. The columns were not considered as hooped concrete, only 500 lb. per sq. in. being allowed for the working stress on the concrete and 7 500 lb. per sq. in. on the steel. Only one change was made in the size of the concrete columns. From the basement to the third story the columns were 24 in. square, and above that they were 20 in. square. The corners of the columns were rounded to a radius of 4 in., except in the basement.

Fig. 4 shows the typical reinforcement of the columns, girders, and slabs. Temporary cross-angles were bolted to the steel column rein-

Messrs. Condron and Sinks. forcement to support the column forms, and, in turn, the floor forms above. After the concrete for one floor was finished, the weight of the form work of the floor above was supported by shores in the usual manner, resting directly on the finished concrete floor. The temporary angles were then removed from the columns, and the column boxing was closed, preparatory to casting the concrete in the column section above the finished floor. This is all shown quite clearly in Figs. 1 and 2, Plate XXVI.



TYPICAL REINFORCEMENT FOR THE MANUFACTURERS FURNITURE EXCHANGE BUILDING CHICAGO.

Fig. 4.

Fig. 1, Plate XXVI, is a photograph taken at the beginning of the concreting work on the first floor (October 10th, 1906), and when taken, the forms were completed for the first floor, the reinforcement of this floor was in place, the column reinforcement, extending from the

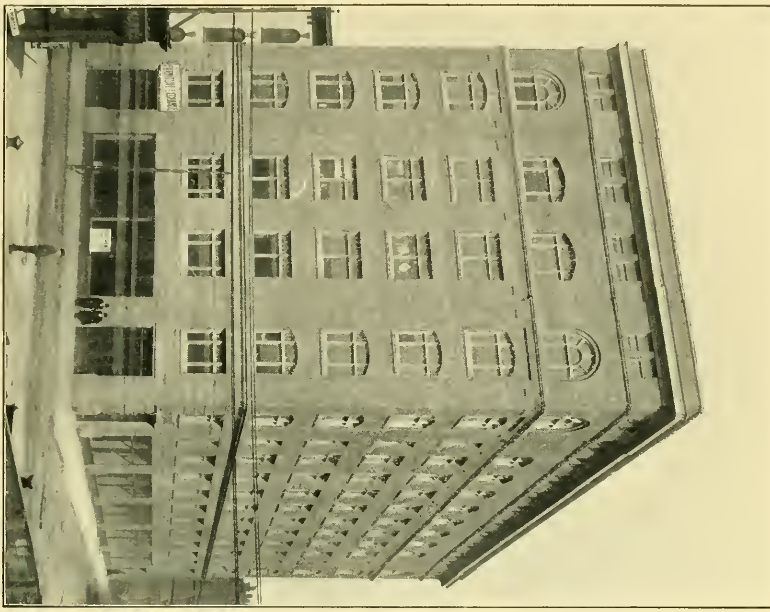


FIG. 1.—MANUFACTURERS' FURNITURE EXCHANGE BUILDING,
CHICAGO, ILL.

FIG. 2.—MANUFACTURERS' FURNITURE EXCHANGE BUILDING,
DEC. 15TH, 1906.

footings to the level of the second floor, was also in place, and the basement columns were cast. The character of the column reinforcement is shown very clearly in this photograph. The points where the lacing of the columns is omitted near the top are the openings left for the reinforcement for the second-floor beams to pass through. Messrs. Condron and Sinks.

In Fig. 2, Plate XXVI, the temporary angles may be seen near the bottom of the column in the foreground. This photograph was taken on November 6th, and shows the concreting in progress on the second floor. After the concrete work on the first floor was finished, no more concreting was done until after the forms for the second and third floors were both completed. The carpenters then worked on the third floor, building the forms for the fourth floor. At this stage of the work the reinforcing material for the second floor was placed, and the concreting of this floor proceeded. The photograph, Fig. 1, Plate XXVII, was taken at the same time as Fig. 2, Plate XXVI, from which it will be seen that the exterior walls had been run up practically to the level of the fourth floor, and the carpenters are seen working on the fourth-floor forms, and, as stated previously, concrete was being placed on the second floor. From this time forward, both the concrete gang and the carpenters were kept constantly at work, the carpenters being two stories ahead of the concrete gang. In order to prevent freezing, the window openings were closed, and coke fires were kept burning in salamanders in the story directly under the floor on which concrete was being placed. As a consequence, the concreting went on at a temperature that was considerably above the freezing point, even in the coldest weather. The cement finish was put on the concrete floors as soon as the first concrete had taken its initial set. Owing to the prevalence of rainy weather during this construction, considerable trouble was caused by water dripping from the forms on the newly finished cement floors, and great care had to be exercised to protect these floors from injury.

Fig. 2, Plate XXV, is a photograph, taken on December 15th, when the exterior walls were finished. It shows the concrete hoist on the side of the building, with a dumping bucket just below the level of the fourth floor. The scaffold at the rear of the building carried the elevator used for raising brick. The concrete mixer is shown on the ground at the base of the concrete hoist.

As already stated, concreting on the first floor began on October 10th, and the concrete roof over the eighth story was completed on December 27th, only 66 working days intervening between the time of starting the first floor and the completion of the roof. On December 31st the tenants began moving into the practically finished building.

Fig. 2, Plate XXVII, is a photograph of the third floor, taken on December 29th, and is typical of the upper floors.

Messrs. Condron and Sinks.

The details of the reinforced-concrete construction are shown quite clearly in Fig. 4. It will be seen, both in the beams and in the slabs, that some of the reinforcing bars run straight through on the bottom, and some are turned up to form the reinforcement for the upper face of the beams and slabs in such manner as to provide as much steel in the top of the beams and slabs over the supports as in the bottom of the beams and slabs between the supports.

Fig. 5 is a bill of bars for the slabs of one floor, and Fig. 6 is the bill of bars for one girder, taken from the working plans. These bills of bars were prepared carefully and were shown on the working plans, together with sketches of each different beam or girder, so that the

Mkd.	Total No. of Bars	Size	Length	Shape	Location
A-A	1324	½″	17′6″	12″; 10′6″; 6′0″; 3″	P_{16} to P_{24} incl.
B-B	224	"	14′0″	Straight	$P_{19\text{-}20\text{-}22\text{-}23\text{-}24}$
C-C	9	"	6′0″	"	P_{21}
D-D	20	"	8′0″	3″; 12″; 4′0″; 3′0″	P_{25}
E-E	235	"	2′6″	6″; 2′0″	Anchors
F-F	5	"	14′0″	3″; 12″; 5′0″; 8′0″	P_{24}
H-H	5	"	9′6″	Straight	P_{24}
X-X	224	"	23′0″	"	Longitudinal
Y-Y	18	"	18′0″	"	"
H-H	3	"	9′6″	"	"
J-J	4	"	17′6″	10′6″; 2′0″; 5′0″; 3″	P_{18}

BILL OF BARS IN SLABS FOR ONE FLOOR

Fig. 5.

contractor was able to get out the correct number of bars, and bend them to the shape required. By following the plans, it was a simple matter to select the right bars, and place them properly in the beams and panels. The specifications required that no concrete should be put in until the inspector had checked and approved the placing of the reinforcing material. The floor bars were held the proper distance above the forms by 2-in. lengths of round iron of the proper diameter, while the bars in the beams were supported by two cement blocks in each beam like those shown in Fig. 4. About 1 300 of these cement blocks were required for the entire building, and each block was reinforced with two No. 8 wires, so that they could be handled safely. These blocks were found to work perfectly, the bars resting in the notches, thus being held in their proper places while the concrete was poured into the beams.

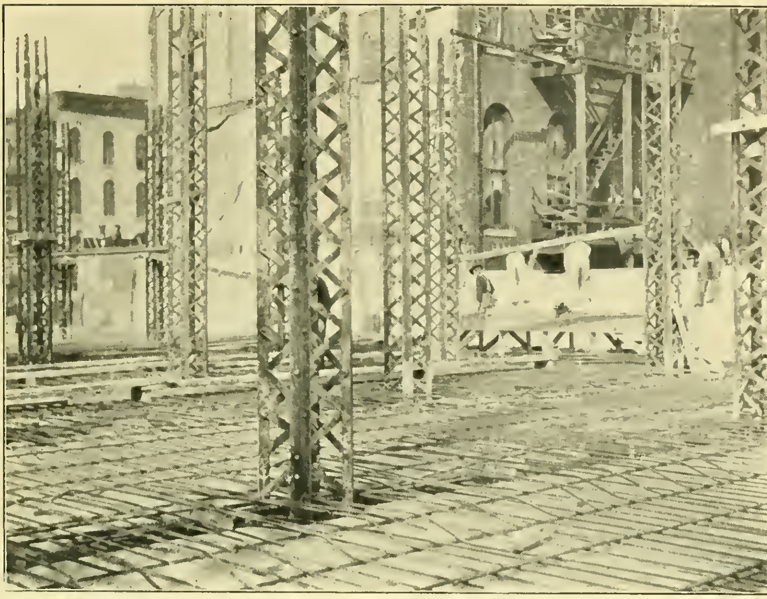


FIG. 1.—REINFORCING OF COLUMNS AND FLOOR, MANUFACTURERS' FURNITURE EXCHANGE BUILDING.

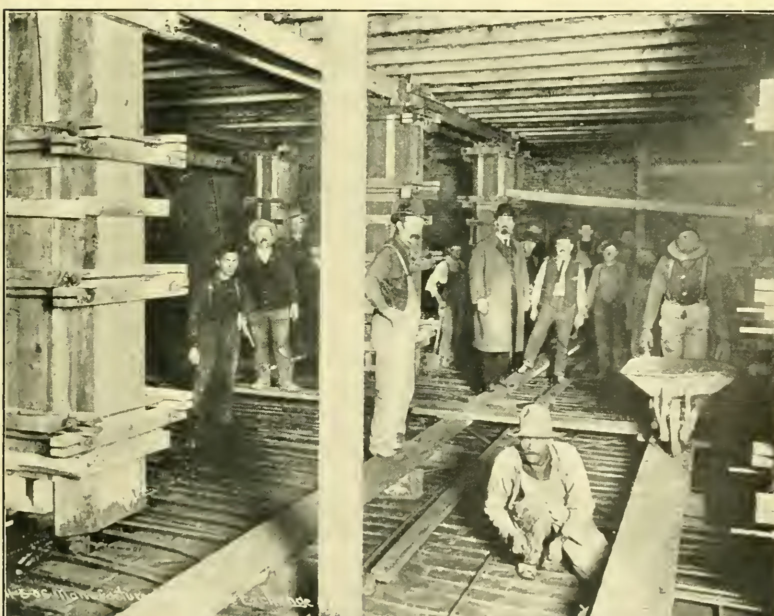


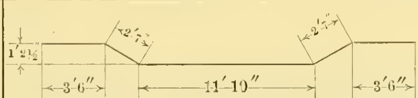
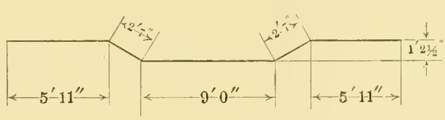
FIG. 2.—CONCRETING ON SECOND FLOOR, MANUFACTURERS' FURNITURE EXCHANGE BUILDING.

Messrs. Condron and Sinks.

Fig. 7 shows the reinforcement of the stairways, of which there were two flights running from the basement to the eighth floor.

The extreme simplicity of the reinforcement of this building is evident, and, as corrugated bars were used, it was not necessary to make bends at the ends of the bars or use other means of insuring bond, the form of the bar giving in all cases an absolute bond between the concrete and the steel.

The total cost of the reinforcing bars delivered in Chicago was almost exactly 5% of the cost of the building, and, while the special bar used cost more than plain or other forms of reinforcing bars, the saving which would have been made by using a less expensive one would have been insignificant as compared with the cost of the building.

Mk.	No. of Beams	No. of Bars in each Beam	Shape
G_6	154	$2-\frac{3}{4}''-16'0''$	Straight
		$2-\frac{5}{8}''-21'0''$	
		$2-\frac{5}{8}''-26'0''$	

BARS REQUIRED FOR BEAMS G_6

FIG. 6.

Under the Chicago Building Ordinances, it is necessary for reinforced-concrete floors to be tested with a load at least double that for which they are designed, and this ordinance requires that the floors thus tested shall show no evidence of failure and shall not deflect more than $\frac{1}{700}$ of the span, or $\frac{1}{4}$ in. for a 14-ft. span.

Under this ordinance, these floors were tested with a load of 350 lb. per sq. ft., covering an entire panel of 14 by 17 ft., under which test load a deflection of less than $\frac{1}{16}$ in. was measured.

Later, the writers followed this method of design for the warehouse of the Advance Thresher Company, at Kansas City (Mr. J. C. Llewellyn, Architect), the typical reinforcement of the columns, girders, beams, and floor slabs of which is shown in Fig. 8. These floors were designed to carry a working load of 250 lb. per sq. ft. in addition to the dead load. In this case, floor slabs of 8 ft. span were

Messrs. Condron and Sinks.

adopted, and these were carried by concrete joists framing into concrete girders. The spacing of the columns was very irregular in the building, but the maximum spans were 24 ft. The type of column reinforcement was about the same as in the Manufacturers' Furniture Exchange Building, except that the angles for this building were placed with the corners out, instead of in.

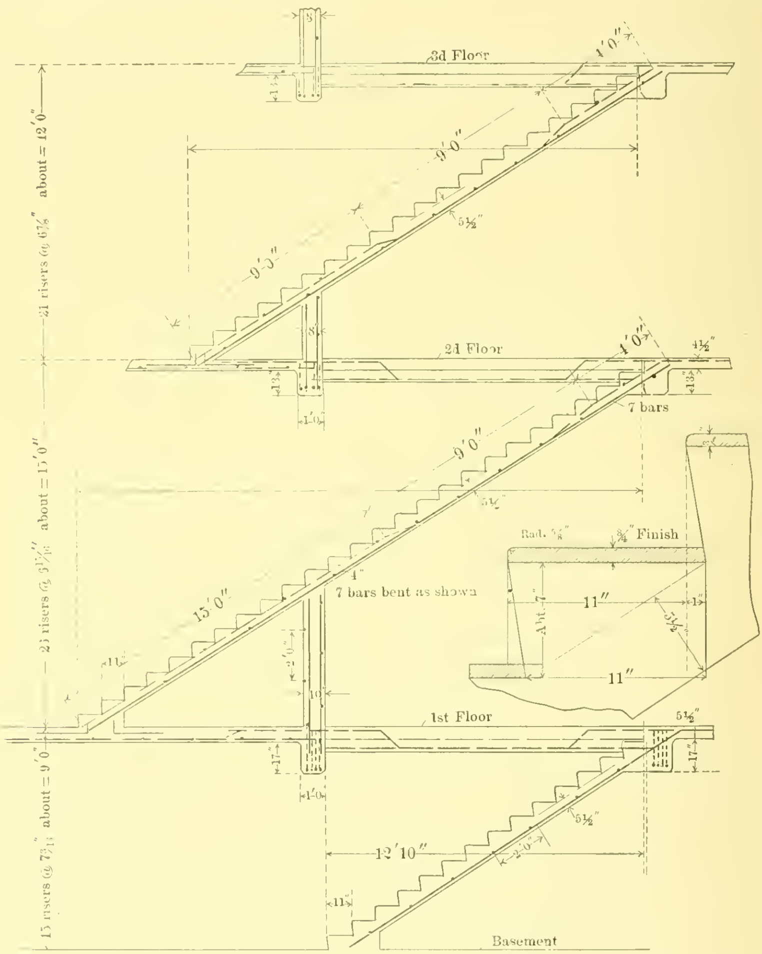


Fig. 7.

Owing to the fact that this building is to be used for exceedingly heavy concentrated loads, that is, the weight of traction engines having 12 000 lb. concentration on a wheel, the owners desired to satisfy themselves of the effect of concentrated loads on these floors, and therefore, the following test was made:

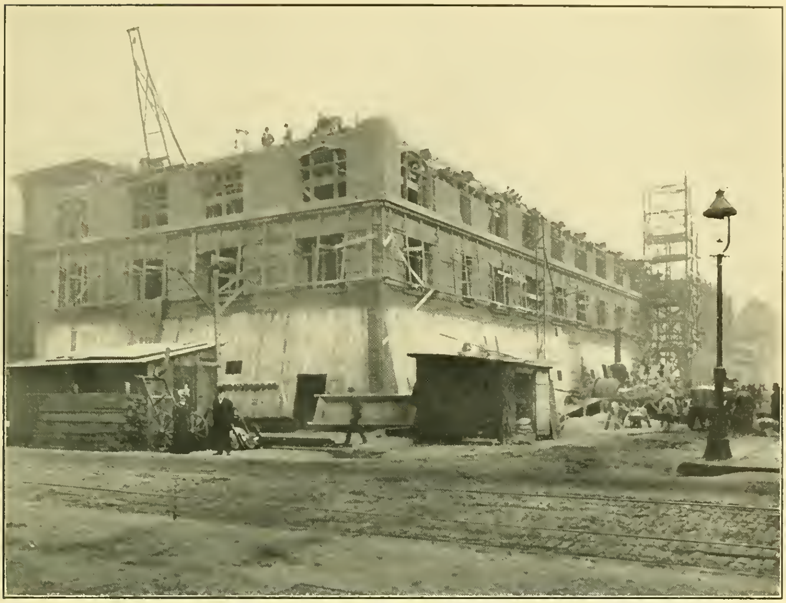


FIG. 1.—EXTERIOR OF MANUFACTURERS' FURNITURE EXCHANGE BUILDING, NOV. 6TH, 1906.

FIG. 2.—TYPICAL INTERIOR, SECOND TO SEVENTH STORIES, MANUFACTURERS' FURNITURE EXCHANGE BUILDING.

Messrs. Condron and Sinks.

Two strips of 2 by 4-in. wood, each 4 ft. long, were laid along the centers of two adjoining 8-ft. slab spans parallel with the supporting joists. On these two strips rested a platform on which a load of 40 850 lb. was placed, giving a concentration of 20 425 lb. on each strip, or a concentrated load, at the center of each of these two slabs, of 5 106 lb. per lin. ft., thus producing the same moment in the slab

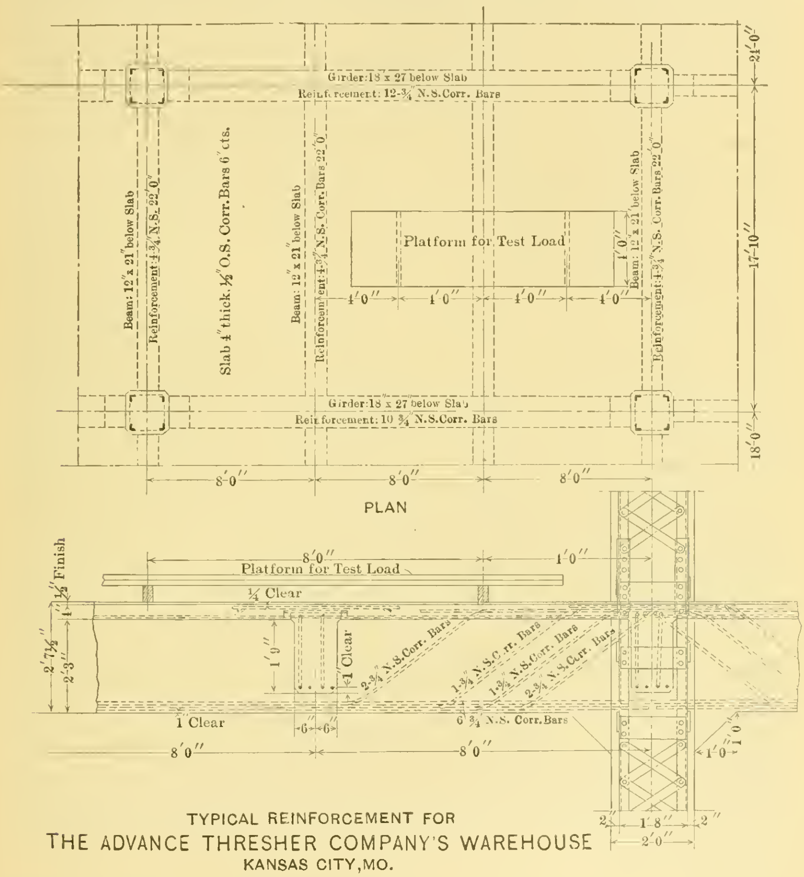


FIG. 8.

as a uniformly distributed load over a 4-ft. width of the two panels of 1 274 lb. per sq. ft. Under this test a deflection of $\frac{1}{16}$ in. was measured. Of course, the entire floor assisted in carrying such a test load, and this is only mentioned as illustrative of the remarkable carrying capacity of such floor slabs. Notwithstanding the fact that such floor tests give astonishing results, the writers believe that floors should be designed, not on the basis of such tests, but in accordance with con-

Messrs. Condron and Sinks. servative practice, and in all their work they have considered that, for floor slabs and intermediate beams, where the reinforcement passes over the top of the slabs and beams at the ends and well into the next panel, the moment is equal to $w\ l^2 \div 12$, and that, for end beams and end panels, where the reinforcement can only pass in this manner over the top at one support, $w\ l^2 \div 10$. The writers consider that the dead load plus the assumed live load will stress the reinforcing material to one-third of its elastic limit, and they use from 0.8 of 1% to 1% of steel reinforcement having an elastic limit of 50 000 lb. per sq. in. They have not calculated the beams as T-beams, even in such floor construction as illustrated, but have considered them as rectangular beams with a depth equal to the distance from the top of the floor to the bottom of the beam, and in them have used reinforcement as great as 1½%, but usually it does not exceed 1¼%, of the area of the beam, not including any portion of the floor slab except that which is a part of the beam section.

In designing beams and girders, the writers have used the empirical formula, $M = (450\ P + 55)\ bd^2$. This formula was adopted as a result of the study of all the tests of reinforced concrete beams which had been made in the various engineering laboratories of the technical schools prior to June, 1905. Up to that time, 202 beam tests had been reported, of which 72 were of beams reinforced with Johnson bars having an elastic limit of about 50 000 lb. per sq. in. Of these 72 tests, 80% showed ultimate strengths exceeding that given by the formula, while only 8% developed less than 90% of the formula strength, and the lowest test developed 78 per cent. This formula is only used where P, the percentage of reinforcement, is more than half of 1%, and not more than 1½%, and where steel having an elastic limit of 50 000 lb. per sq. in. is used, and with a positive mechanical bond. For 1% of reinforcement, the ultimate moment, $M = 505\ bd^2$, and, in general that percentage has been used. The writers have considered that three times the dead-load moment plus three times the live-load moment is equal to the ultimate moment. Therefore, if the dead load is equal to one-half of the live load, it would require, theoretically, an application of four times the working live load to reach the ultimate load. The tests made indicate that this practice is on the safe side.

Mr. Stern. E. W. Stern, M. Am. Soc. C. E.—The author states that, in view of the uses to which the McGraw Building was to be devoted, it was imperatively necessary that it should be designed to afford the greatest possible resistance to the vibration of heavy machinery. Now, is enough known about the action of reinforced concrete under vibratory loads to make certain its suitability for this purpose?

In reinforced concrete buildings, cracks occur when there are practically no vibratory loads; under the influence of vibrations continued for a number of years, due to running machinery in the building, is it certain that cracks will not develop, and that the reinforcing rods will not work loose in the concrete? Mr. Stern.

Considering the design of the columns, the author assumes that stress is transmitted into the concrete filling through the rivet heads and lattice bars of these columns, so that both steel and concrete act together as a unit. The speaker cannot accept this assumption. It seems to him far-fetched and entirely problematical. If there are any experiments to fortify the contention of the author, it would be of benefit to this discussion to have these results.

The author likewise adopts a working stress of 750 lb. per sq. in. on the concrete filling of the columns, equivalent to 45 tons per sq. ft. Such a very high unit stress is so much more than has been considered good practice (being more than double that allowed in the Building Code of Manhattan), that the author should give the Profession the benefit of the experiments upon which he bases his conclusions.

It is not clear to the writer, in examining the details of the columns, how the splices of the columns were arranged at the various joints to take care of the reduction in dimensions. For instance, the columns in the ninth story are 17 by 17 in., back to back of angles, whereas the next section of columns, supported on these, is decreased suddenly to 10 by 10 in., back to back of angles. It would be of interest to know how this change in size was taken care of in the details.

The speaker believes the type of column used to be neither as economical nor as efficient as a box steel column made of plates and channels. A column of this type, to take the same load, would be made of 15-in. channels with 17-in. cover-plates, and would build up about 21 in. square, if surrounded with 2 in. of fire-proof covering. The columns in the McGraw Building are 29 in. square in the basement and first floor, or about 40% larger in outside dimensions, and occupy nearly twice as much space; in fact, in the lower stories, these columns are actually about as large in outside dimensions as the steel columns in the thirty-two story City Investing Building, at Broadway and Cortlandt Street, designed by the firm of which the speaker is a member.

There is actually more steel in the type of column adopted than there would be in the channel and plate box column above mentioned, assuming that the entire load were carried by it, without any regard to concrete filling; and, if it were intended to fill these columns solid with concrete and surround them with a fire-proof covering of that material, there would be less concrete used, so that the column adopted was extravagant both in material and in space occupied.

The author claims that the use of the type of steel column adopted was a great convenience in erection, as it enabled the steelwork to be

Mr. Stern. erected ahead of the concrete work, and afforded convenient supporting members for the adjoining forms or for other erection work. This argument would be equally applicable to the more economical type of column suggested by the speaker.

The columns are spaced too closely together for a building adapted to loft purposes. The interior columns, parallel to 39th Street, are 15 ft. 9 in. to 14 ft. 8 in. apart. A better arrangement would have been to have these columns spaced about 18 ft. apart.

The author also states that the construction of the concrete work in this building during the winter of 1906-07 was entirely successful, thus demonstrating that reinforced concrete work may be conducted during a New York winter without material interruption. He states that this was accomplished by covering window openings with canvas, using salamanders, and covering the fresh concrete, as fast as poured, with tarpaulins or hay, or both. Now, while it may be possible to obtain first-class concrete work in this way, it is undoubtedly a great expense thus to do the work, and likewise risky, as the work may freeze at any time, especially the thin floor slabs, from underneath.

The speaker knows of a number of cases of collapse due to this cause, and he believes that the erection of reinforced concrete work, in which there are thin floor slabs, undertaken during freezing weather, carries with it grave responsibility and uncertainty.

It might be interesting to compare the quantity of steel required in reinforcing the concrete work of the McGraw Building and in that of a steel skeleton structure. A complete steel frame structure for the McGraw Building, computed for the same loads that were used by the author, would weigh approximately 1 500 tons. In the McGraw Building the steel columns weigh 655 tons, the reinforcing rods 507, making a total of 1 162 tons, equivalent to a saving in the McGraw type of 340 tons in the steelwork. This would amount to about $21 000, assuming the price of steel to be about $62 a ton. This difference, however, would most likely be more than offset in other ways in the steel skeleton type, as there would be much less concrete required, and the erection methods would be less expensive. Perhaps the author made comparisons as to the cost of these different types of construction; if so, it would be interesting to have his figures.

In an experience covering more than seventeen years in the construction of buildings, the speaker has had to deal with practically all kinds of materials, and has had charge of a number of reinforced concrete structures. In his opinion, nothing, thus far, has been devised which is comparable to the modern steel skeleton type of construction for high buildings, not only for safety, but for economy, speed in construction, and ability to make the frame as thoroughly fire-resisting as possible.

Every condition of loading can be intelligently taken care of in a steel structure, the stress in each member of the frame being capable of complete analysis, and the knowledge at hand to-day as to what the unit stresses should be has been so thoroughly tried out that it is safe to say there is practically no element of uncertainty in the design of a steel building. In a reinforced concrete building, however, the case is otherwise. The factor of ignorance is much greater. Most of the work is done on the premises by labor more or less unskilled. Mr. Stern.

The supervision of the work during construction is of the most exacting nature, and requires high intelligence and unremitting vigilance. The difficulty of getting good workmanship, and of making the construction correspond with the plans, is very great, and, finally, after the work is finished, grave defects of workmanship may exist in spite of all the care exercised.

L. J. Mensch, M. Am. Soc. C. E. (by letter).—This paper has been read with great interest by the writer, and, while he does not doubt that the owners are more than pleased with the strength of the building, he has to take exception to many statements made. Mr. Mensch.

The structure cannot be called a true reinforced concrete building, the columns being of steel, fire-proofed by concrete, although ostentatiously calculated as reinforced concrete columns. Neither is it the latest type of reinforced concrete building construction; it is, in fact, the oldest type of high building construction in which reinforced concrete was used. After the introduction of reinforced girder and slab construction, many years elapsed before owners and architects could be persuaded to allow the use of reinforced concrete columns, and, in most cases, latticed steel columns, fire-proofed by concrete, were used. Of the numerous buildings of this type, the writer will mention only the ten-story Audit Office of the French Government at the Cours de la Reine, Paris. The statement, that the McGraw Building is higher than heretofore considered practicable, must be contradicted. The height of the Ingalls Building, in Cincinnati, is about 220 ft. above the basement, and the height of the Pugh Power Building, in the same city, designed by the writer, is about 180 ft. above the basement. The latter is used for the same purpose as the McGraw Building, and has also the same spacing of columns; and the first section, 70 by 335 ft., proved such a success that the owner built an addition to it, making it now about 150 by 335 ft. and ten stories high. Mr. Douglas has shown clearly the waste of steel in the columns.

It is true that very few tests of structural steel columns strengthened by concrete filling have been made, and the writer is pleased to be able to mention at least one test which was made by Dr. F. von Emperger, and published in the July number of *Beton und Eisen*, 1907. Two I-beams, about $5\frac{1}{2}$ in. deep and $6\frac{3}{8}$ in. from center to center, were connected by eight flat irons $2\frac{1}{2}$ by $\frac{1}{4}$ in. in a length of 13 ft. This column failed at 100 000 lb., the I-beams buckling separately. The

Mr. Mensch. column was straightened out, and the space between the I-beams filled with concrete, and tested after six weeks. The composite column failed at 265 000 lb. The iron section contained 6.3 sq. in., the concrete section contained 31½ sq. in., the radius of gyration of the two I-beams was 2.6 in., and, from this, Dr. von Emperger demonstrates that the carrying capacity of this column is to be considered as the sum of the carrying capacities of the iron and of the concrete. He also mentions that the concrete completely separated from the iron, and crushed into pieces from 1½ to 3 ft. long. Test cubes, cut from such pieces, gave an ultimate resistance of 1 120 lb. per sq. in.

Although this test may not be entirely convincing, it shows that, in such a column, the highest working stress on the steel section may be allowed safely, disregarding the concrete, which may be considered as acting only as a stiffener. From this it follows that it would have been safe to reduce the size of the columns of the McGraw Building.

The author is correct in his statement that the form work represents the most difficult part of reinforced concrete construction; and the success of a contractor, and also the speed of erection, depend entirely on his ability to organize his carpenter force, and to give his foremen complete working drawings, omitting not the smallest detail, even specifying the number and kind of nails; in fact, do the work on the same basis as structural steelwork. But it is also the duty of the engineer to design the building so that the form work is reduced to a minimum. For example, lumber comes only in certain sizes—a so-called 2 by 10-in. plank, is generally only 1⅝ by 9½ in.—and, if a column 20 in. square is specified, the forms can only be made by ripping the planks. On the other hand, it will be found that a column 19¼ by 19½ in. can be formed in by using commercial lumber, and it is absolutely necessary that the designing engineer should know the commercial sizes of lumber, as they vary with different localities. The same applies to girders and beams, which, as a rule, cannot be obtained in even dimensions without considerable waste of labor and material.

The use of brackets should be carefully considered. It seems that in most cases they are adopted for good luck, with no regard to statical considerations. The writer has seen many brackets, the under side of which formed an angle of 60° and more with the horizontal, which were generally not more than 8 or 12 in. in length. Such brackets add greatly to the cost, but very little to the strength, of the structure. A little consideration would show that it would be cheaper to use deeper girders and omit the brackets. A bracket is of importance only in case the underside forms an angle of not more than 25° with the horizontal.

The layout of girders and beams should be made as simple as possible. The writer cannot say that the distribution of girders and beams in the McGraw Building is the most economical, or the most favorable, for the form work. In the Pugh Building, the girders were adopted in the direction of the 21-ft. spans, and the beams in the direction of the 14-ft. spans, and were spaced about 11 ft. apart. This reduced the number of beams, and made the centering of the girders much simpler, and the little excess of concrete used in the slabs, which were reinforced in both directions—a necessity in every good design—was more than counterbalanced by the saving in the form work and time. The fact is that, although the floor area of this building was more than 20 000 sq. ft., and all the walls and main partitions were also of reinforced concrete above the third floor, the rate of progress was a story every 16 days, with a comparatively small gang of men. Mr. Mensch.

In regard to the use of a derrick tower with four swinging booms, the writer's experience proves that the cost of the handling of the concrete and the installation of such an outfit is considerably more expensive than the use of a small elevator and concrete bucket, which empties into a hopper and is hauled in two-wheeled buggies to the place where needed.

P. E. Stevens, Assoc. M. Am. Soc. C. E. (by letter).—In the design of the McGraw Building, careful attention has been given to those details intended to secure continuity in the beams, even though the New York Building Code does not permit the designer to take full advantage of the increased strength resulting from such continuity. This feature of the design has been made the subject of some adverse criticism, on account of the abundant reinforcement provided. The writer believes that these criticisms are not well founded. The description and drawings of the reinforcing frames for the girders show the steel reinforcement over the supports to be the same as that at the center of the span, and this has been regarded by some as a waste of material. The usual formulas for stresses in continuous beams apply in the case of concrete only when such reinforcement is provided. In the derivation of such formulas, three conditions are imposed: Mr. Stevens.

1.—Unyielding supports, conforming to the unstrained outline of the beam—usually styled supports all on a level;

2.—Spans all equal;

3.—Uniform moment of inertia throughout the length of the beam.

The first of these conditions it is impossible to fulfill, the second seldom prevails, and the third is commonly ignored. This is not intended for cynicism, but is a simple statement of fact. Imperfect workmanship, uneven shrinkage of concrete, and elasticity in the material, make any assumption of "supports on a level" untenable. This alone is sufficient reason for placing small dependence on the increased strength due to continuity, when designing the beam.

Mr. Stevens. An example of a very common and erroneous interpretation of the third condition is found in another discussion of this subject. Mr. Noble has said that the formulas for stresses in continuous beams apply only when the bending moments are

"adequately met by moments of resistance, and then only when the unit stresses in the material furnishing this amount of resistance are the same at the center span and the points of support."

This is decidedly at variance with the third condition imposed by the formulas. Uniformity of stress is far from identical with uniformity of moment of inertia.

The amount of the bending moment over the support added to that at the center of the span will give a sum equal to $\frac{w\,l^2}{8}$ in an infinite series of uniform spans uniformly loaded. If, further, the moments of inertia of the sections of the beam are the same throughout its length, then, and then only, the bending moment over the support is $\frac{2}{3}\left(\frac{w\,l^2}{8}\right)$, and that at the center is $\frac{1}{3}\left(\frac{w\,l^2}{8}\right)$. In a series of eight or more spans complying with the three conditions before mentioned, and uniformly loaded, the span at the middle would have approximately—within 1%—the above distribution of bending moments.

A design which assumes some fraction of this total, $\frac{w\,l^2}{8}$, as the moment at the support, and the remainder as the moment at the center of the span, simply because moments of resistance have arbitrarily been provided at those points to resist such moments, cannot be justified by any sound theory.

In order to show how erroneous any conclusions drawn from such assumptions may be, the writer has derived the correct moments at the supports and center of the span corresponding with various ratios between the moment of inertia for the cantilever portion and that for the suspended portion of the span.

The beam to be considered will be assumed to be one of an infinite series of uniform spans, uniformly loaded—approximated by a span at the middle of a series of eight or more spans—and with supports "all on a level." In Fig. 9 let the following nomenclature and conditions govern:

$A\ E$ = the undeformed neutral axis of the beam, with supports at A and E;

$A\ B\ C\ D\ E$ = the deformed neutral axis;

B and D = the points of contraflexure;

w = the uniform load per unit length;

I = the moment of inertia of any section of $A\ B$, or $D\ E$;

K = the moment of inertia of any section of $B\ C\ D$.

All conditions are symmetrical, therefore the elastic curve will be symmetrical. Mr. Stevens.

a = the distance, $A\ B$, or $D\ E$;

b = the distance, $B\ D$;

M_0 = the bending moment at A or E;

M_1 = the bending moment at the center of the span, C;

$B\ D$ will be parallel with $A\ E$.

$B\ F$ is tangent to $A\ B$ at B, making the angle, α, with $A\ E$ and $B\ D$;

$B\ G$ is tangent to $B\ C\ D$ at B, making the angle, β, with $B\ D$;

n and m are current co-ordinates of points in $B\ C\ D$, referred to B;

x and y are current co-ordinates of points in $A\ B$, referred to B.

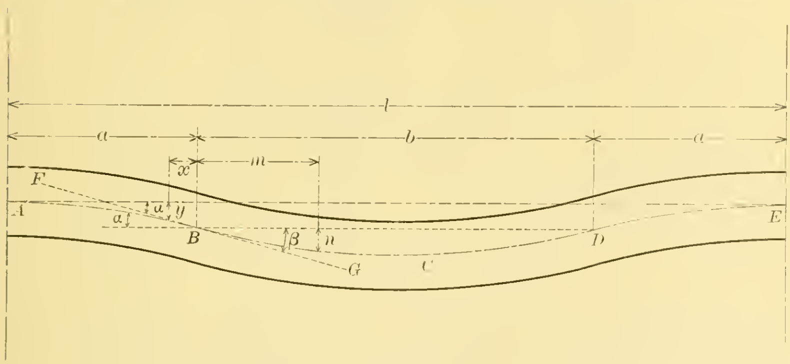


FIG. 9.

Since B and D are points of contraflexure, and bending moments at these points are zero, the beam may be regarded as made of three beams: two cantilevers, $A\ B$ and $E\ D$, and a simple span, $B\ D$. Consider first the span, $B\ D$, loaded with w per unit of length. By the well-known theory of flexure,

$$n = \frac{w\ m}{24\ E\ K}\,(b^3 - 2\ b\ m^2 + m^3) \ldots\ldots\ldots\ldots (1)$$

Differentiate and obtain the first derivative,

$$\frac{d\ n}{d\ m} = \frac{w\ (b^3 - 6\ b\ m^2 + 4\ m^3)}{24\ E\ K} \ldots\ldots\ldots\ldots (2)$$

Make $m = 0$, then,

$$\tan.\ \beta = \frac{w\ b^3}{24\ E\ K} \ldots\ldots\ldots\ldots (3)$$

Consider now the cantilever, $A\ B$: It is loaded with the uniform load, w, per unit length and the end reaction from $B\ D$ at B. This end reaction is equal to $\frac{w\ b}{2}$. From uniform load:

Mr. Stevens.

$$\text{deflection} = \frac{w}{42\,E\,I}(x^4 - 4\,a^3\,x + 3\,a^4) \ldots\ldots\ldots\ldots (4)$$

and, from load, $\frac{w\,b}{2}$ at B:

$$\text{deflection} = \frac{\frac{w\,b}{2}}{6\,E\,I}(2\,a^3 - 3\,a^2\,x + x^3) \ldots\ldots\ldots\ldots (5)$$

and y, the sum of these deflections, is

$$y = \frac{2\,w\,b\,(2\,a^3 - 3\,a^2\,x + x^3) + w\,(x^4 - 4\,a^3\,x + 3\,a^4)}{24\,E\,I} \ldots (6)$$

obtain the first derivative:

$$\frac{d\,y}{d\,x} = \frac{2\,w\,b\,(3\,x^2 - 3\,a^2) + w\,(4\,x^3 - 4\,a^3)}{24\,E\,I} \ldots\ldots\ldots (7)$$

make $x = 0$, then,

$$\tan.\ \alpha = -\frac{6\,w\,b\,a^2 - 4\,w\,a^3}{24\,E\,I} \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Since B is a point of contraflexure, $\alpha = \beta$, and

$$\tan.\ \alpha = \tan.\ \beta \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

and, from Equations 3 and 8,

$$\frac{w\,b^3}{24\,E\,K} = -\frac{6\,w\,b\,a^2 - 4\,w\,a^3}{24\,E\,I} \ldots\ldots\ldots\ldots\ldots (10)$$

$$\text{whence,}\ \frac{b^3}{6\,b\,a^2 + 4\,a^3} = -\frac{K}{I} \ldots\ldots\ldots\ldots\ldots (11)$$

The negative sign governing the second term of this equation is due to the fact that the moments in the cantilever and those in the suspended portion are of opposite sign. It will be dropped hereafter, as it is immaterial to this discussion.

By assigning to a and b consistent values, fractions of the total span, $A\ D = l$, corresponding values of $\frac{K}{I}$ may be obtained. Such values have been platted and a curve drawn through them (Fig. 10). A curve has also been drawn for the values of M_o in terms of $\frac{w\,l^2}{8}$, and one for the ratio, $\frac{M_1}{M_o}$, for comparison with $\frac{K}{I}$. Attention is called to the values of the various functions given by these curves corresponding with $a = l\,(1 - \frac{1}{3}\sqrt{3}) = 0.211\ l$. The curves show $\frac{K}{I} = 1$: hence

$$K = I,$$

$$M_o = 0.66\left(\frac{w\,l^2}{8}\right)$$

$$M_1 = \tfrac{1}{2}\,M_o$$

which values correspond exactly with the formula for beams with uniform moment of inertia. It will be noted that the curves for $\frac{K}{I}$

Mr. Stevens.

FIG. 10.

Mr. Stevens

and $\frac{M_1}{M_0}$ intersect at a common value of 0.17, indicating that for these values, if the depths are uniform, the stresses at the center and at the supports will be equal. If, as is often done, $\frac{K}{I}$ is made equal to $\frac{1}{2}$ and a bending moment of $\frac{2}{3}\left(\frac{w\,l^2}{8}\right)$ is provided for at the supports, an overstress of $12\frac{1}{2}\%$ will result. If, as is said to be the French practice, $\frac{4}{5}\left(\frac{w\,l^2}{8}\right)$ is provided for at the center of the span, and $\frac{1}{5}\left(\frac{w\,l^2}{8}\right)$ is provided for at the supports, then $\frac{K}{I} = 4$, and $M_0 = 0.475\left(\frac{w\,l^2}{8}\right)$. Therefore, if the conditions of continuity and loading assumed are realized, the result will be an actual theoretical stress $2\frac{3}{8}$ times as great as that calculated. This appears to the writer to be extremely bad designing.

It must be borne in mind that the live load to be sustained by the beams and columns of a building is not a uniformly distributed one, but a constantly varying set of unequal loads, sometimes concentrated and sometimes distributed over varying areas. In a steel structure, where each beam is an independent span, a uniform live load, with a proper "scaling down formula" can be specified which will enable the designer to work very close to actual conditions. In a reinforced concrete structure, where the floor slabs, the floor beams, and the girders are built as continuous beams, the stress in every beam, girder, and column is influenced by every live-load concentration, whether it is assumed so or not. In such a case, it is manifestly impossible to make close calculations, and the most conservative estimate must be placed upon the value of continuity as a factor in saving material. Also, if concentrated loads are to be provided for, the reinforcement over the supports must be made equal to that at the center of the span, as has been done in the design of the McGraw Building.

In the design of steel structures, experience has taught that forms in which the stresses in each member can be determined accurately are preferable. Many of the statically indeterminate forms, such as multiple intersection trusses and continuous girders, have almost entirely disappeared.

The workers in the younger art of reinforced concrete would do well to give most respectful consideration to this idea of using statically determinate forms, which has become so general in the design of structures in steel—a material the properties of which may be far more accurately determined or controlled than those of concrete.

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PAPERS AND DISCUSSIONS.

THE USE OF REINFORCED CONCRETE IN ENGINEERING STRUCTURES.

An Informal Discussion.*

By Messrs. E. P. Goodrich, Edwin Thacher, Sanford E. Thompson, William H. Burr, T. Kennard Thomson, D. W. Krellwitz, Guy B. Waite, and C. L. Slocum.

Presented at the Meeting of January 8th, 1908.

E. P. Goodrich, M. Am. Soc. C. E.—The use of reinforced concrete in engineering structures has had a phenomenal development, both as to the amount built in each succeeding year and as to the variety of applications made. Its field of usefulness is rapidly broadening, and its exploitation is believed to have been overdone in a few lines. Mr. Goodrich.

The theory concerning the mode of action of the two materials involved, is constantly undergoing modification, making it more perfect through deduction from experiment. This is the scientific method of development of any art; and in this particular branch of the building art, it is believed that by experiment alone can proper working stresses be determined, upon which to base all designs of structures. Such stresses should be deduced primarily from fatigue experiments, and not be chosen as arbitrary fractions of ultimate strengths. It is believed, further, that every careful designer should take proper account of the secondary stresses induced in structures like buildings and arches, by the increasing permanent set caused by repeated loading.

* The discussion of this subject, for which no formal paper was presented, is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion.

Mr. Goodrich. Experimental research is yet much needed along several different lines connected with this subject. More light is desired as to the cause and cure of the retrogression in the tensile strength of cement briquettes, often disclosed. More extended compression tests are also needed to determine the presence and amount of any retrogression in the compressive strength of concrete. When tests, published in engineering periodicals, show, at the end of 2 years, values hardly greater than those at the end of 7 days, it would seem as if this subject needed most careful investigation.

If possible, a cement of higher compressive strength should be developed, especially for use in concrete columns. Perhaps this is impossible with the materials involved, because of their very nature, and because the strength of the aggregate has been practically reached; but unless some considerable increase can be secured, it would seem as if concrete columns would have to be excluded from consideration in high building design; that is, in structures higher than perhaps six stories. In their stead, structural steel columns would seem necessary, but they should be heavily fire-proofed and entirely filled with a concrete of cheap quality. It may seem excessive to some engineers, but it is believed that experience has shown the necessity of fully 3 in. of good concrete fire-proofing over all extreme edges of such columns.

The best design for the steelwork of columns of this variety, is believed to be of angles latticed or battened, of channels similarly fabricated, or perhaps wide-flanged I-beams, or the usual Z-bar types.

Columns of the Considère variety are believed to be proper, if a suitable relation exists between the spiral and the longitudinal reinforcement, and if a sufficient quantity of each is used. In such columns, a lower limit should be set on the quantity of each kind of reinforcement, and an upper limit on the size of the opening between the parts. It may not be out of place to state that the inventor of spiral reinforcement himself uses spirals of very thick material with a comparatively small pitch, and it is believed that a large majority of the columns being erected at the present time, and considered of high carrying capacity, would not disclose any appreciable excess of carrying power if tested to failure.

In reinforced concrete columns, with longitudinal rods as the principal reinforcement, an upper limit should be set on the percentage which may be allowed. In addition to the fact that many laboratory tests show lower efficiencies for rods of large diameter in concrete columns, it would seem as if the use of rods more than 1½ in. in diameter, or aggregating more than 5% of the total area of the column were of more than doubtful value, simply from the impossibility of being certain that enough adhesion is developed to secure the theoretical compressive stress in the steel itself. It might seem as if more nearly practical conditions would be secured, in laboratory tests, if all reinforcement were kept away from the ends of the concrete columns a distance equal to at least one-fourth of the diameter of the column. Mr. Goodrich.

It is believed that too little attention is given to the design of the footings under such columns, especially with regard to a proper transmission of the stress in the longitudinal rods into the foundation concrete, and that in most work not more than half the proper number of ties are used to prevent buckling in the vertical rods. The disposition of the latter, so as to prevent easy depositing of the concrete, is imperative, and those varieties of columns in which the steel is distributed very uniformly through the whole column section are viewed with distrust, however superior they may be considered from a theoretical standpoint.

Great care should also be exercised in the design of the beam and girder reinforcement, to prevent a congestion of steel in the column sections at the floor levels. Much ingenuity can profitably be expended in obviating this trouble.

It is well known that the addition of steel to increase the compressive strength of concrete columns is not on the side of economy of first cost, but only of economy of floor area occupied. It would seem, therefore, as if the best practice would be to introduce steel only to carry bending stresses, and to use a cement of higher quality (if obtainable), or a richer mixture of the commercial product, and thus secure higher working stresses with correspondingly smaller sections.

The subject of impervious concrete is of vital importance for those who are interested in the construction of dams, reservoirs, conduits, sewers, and water pipes; of hardly less interest in connection with sea walls, retaining walls, bridge abutments, and building foundations; and even of considerable interest in regard to the superstructures of buildings, arch bridges, etc. Many experiments have been made as to the perviousness of different mixtures of different aggregates of different sizes, but, apparently, something further is necessary. Great things are claimed for the several patented wet and dry compounds now on the market, designed to render concrete impervious, and, until further progress can be made in this line, the use of the best of these in all concrete work is strongly recommended. Perhaps an impervious cement will soon be evolved, produced either by the addition of one of the present products to the practically finished cement product, somewhat as gypsum is now added, or by some other substance which the inventor may work out. Such a cement is greatly to be desired, if for no other reason than to prevent the unsightly discoloration from efflorescence which now defaces almost all exterior cement work. This may be partially cured by the use of a so-called "non-staining" cement, but all those now on the

Mr. Goodrich. market are unsatisfactory, either as to effect or first cost. A better and cheaper one is essential.

Although other elements than moisture are involved in the rusting of steel, the use of an impervious cement would have gone far toward preventing the absolute disintegration of the metal backing of some stucco work examined by the speaker on several occasions, and would make one feel much safer as to the probable life of some of the reinforcement which has been plainly visible on the bottoms of floor slabs when the latter were slightly scratched with a knife or other sharp implement. It will not be very long before some of such floors will show signs of failure, especially those in which wires of small diameter, or sheet material of small thickness have been used for the reinforcement.

Impervious concrete is also of vital importance in foundations, and walls of reservoirs, conduits, etc., through which water will percolate slowly. It is known that at least one heavy retaining wall became honeycombed to such an extent that failure resulted; and in one high building on lower Broadway, in New York City, the sub-cellar walls have been screened, apparently to hide the process which may possibly be slowly producing a similar effect in that structure.

Again, the few experiments which have been made with regard to electrolysis of embedded steel in wet concrete, together with the astonishing phenomena observed in one reinforced concrete street car barn, in which hot metal was sometimes encountered when a trolley pole left the trolley wire, seem to be convincing evidence of the necessity of using impervious concrete in all reinforced foundations which may be in the line of electric earth currents.

It would also seem wise to use only such concrete in reinforced concrete piles, because they are relatively slender members, and any disintegration of either the reinforcement or the concrete in them would be of grave moment. As to the general subject of concrete piles, not enough is yet known. While a strong difference of opinion may exist, it would seem as if fewer objections could be raised against those piles which are moulded in plain sight and driven as is a wooden pile, than against those piles which are moulded in place. The latter rarely are properly reinforced, and it is extremely likely that the fresh concrete will be displaced before it has properly set, by operations in their vicinity.

A very heavy hammer should be used for driving such piles, one weighing at least as much as the pile being very important.

It is probable that impervious concrete will partially solve the problems incident to the use of cement in sea water, whether the disintegration caused by the latter be chemical or mechanical in its nature. It has already been demonstrated, at the New York Navy Yard, for example, that a rich face mixture, rendered more impervious by careful surface treatment as soon as the forms are removed, is the best preventive of disintegration. It is hoped that someone, therefore, will produce a water-proof cement. Mr. Goodrich.

More care than is often taken, should be exacted with regard to the placing of reinforcement. Some hints have been given of the evils incident to poorly designed columns, placing floor reinforcement too close to the surface, and the dangers of electrolysis. All these may be obviated to a great extent by the exercise of care in design and execution. The compulsory use of reinforcement fabricated in units, in place of separate bars in beams and girders, is advocated. It is believed that the small possible saving which is claimed for the latter method is more than offset by the insurance that a rod or two will not be accidentally omitted from an important member, or a short one thrown in to take the place of a lost longer one. Such cases have been actually observed, even with the most perfectly organized forces, and one superintendent of a company which still advocates separate bars, once said the company could have some of his salary if they would use units, because of the immeasurable lessening of responsibility on his shoulders.

More attention should be paid to the subject of shear or diagonal tension in reinforced concrete beams and girders. The fact that certain empirical systems have produced many buildings which have not collapsed under load, is no proof of the adequacy of their reinforcement in this respect. It is believed that, in much work now under way, while the factor of safety against failure through tension or compression is four or more, the margin of safety, with regard to diagonal tension, is much smaller. The care taken with this point of design by foreign engineers is far above that common in America. Many more experiments, covering various ages and arrangements, are urgently needed.

The ideas incident to the use of discrete structural members are not applicable to the design of monolithic concrete structures. In the latter, the continuity of the members should be recognized, and the reinforcement of columns, beams, girders, and floors, should be arranged so as to make the parts act as rigidly connected elements. The design of columns simply as compression members, entirely ignoring the bending produced by unbalanced loads on rigidly connected girders, is not considered the best practice; and the use of the factor, $\frac{1}{8}$, in the moment formula is an inheritance from the older methods used in timber and steel. Even the use of so much reinforcement at supports as corresponds with the factor, $\frac{1}{10}$, is believed to be entirely inadequate; and the speaker ventures the prophecy that progressive failure is taking place in many structures designed with only that quantity of steel at the points in question.

It is further advocated, that designs be made so that tests of se-

Mr. Goodrich. curity can be carried out on the very day the centering is allowed to be removed. It is also recommended that the tests be exacted on those dates. Such specifications, rigidly adhered to, would reduce to a minimum the danger and number of premature failures.

Deformed rods may be better in theory, but almost no practical proofs of their superiority have been produced, as far as known. Laboratory tests are hardly conclusive, since many experiments on beams actually seem to show some kinds of rods to be really detrimental to the best results.

More experiments are very desirable concerning the effect of the proportion of water used in the original mixture and the effects of continued and intermittent saturation of the concrete, upon the adhesion between it and smooth rods embedded therein. Perhaps the use of impervious concrete would solve this difficulty, irrespective of the actual effects produced by excessive moisture. More fatigue experiments, also, are essential to a full knowledge of this subject, and the very few so far reported along all lines are worthy of the highest commendation and the most careful study.

The compression experiments of this kind, in conjunction with those carried to rupture on columns of the Considère type, seem really to show the justice of allowing high stresses on such columns. As long as the elastic limit of the concrete is not reached, since columns reinforced in this way show very large deformation before final failure (thus reducing the danger of the latter), there would seem to be no good reason for restricting the working stress to the low figures at present usually exacted for plain concrete or longitudinally reinforced columns.

Nor do rods of high elastic limit appear to be advantageous, under ordinary conditions. Since all varieties of steel have practically the same modulus of elasticity, and since the first tension cracks in the concrete appear at approximately the same strain in all specimens, and consequently at the same stress, irrespective of how much higher the elastic limit may be, the relative amount of the latter is of no importance provided it is beyond the usual working stress.

Perhaps such rods may be of value in column work, where high stresses are used, and they may be advantageous in the reinforcement of long walls against shrinkage, but, even in these positions, the advantage is not evident. Reports as to actual structures of the last mentioned kind, where no cracks have appeared, together with the amount of steel introduced, are greatly to be desired. It is possible that the distribution of the reinforcement is also influential to some extent.

The character and size of the aggregate does not receive half the attention it deserves, and the quantity of water being used, especially in the manufacture of much cement brick, concrete blocks, and ornamental cement work, is entirely inadequate. Very few persons are interesting themselves in the artistic phase of the subject, and the results attained in most part are still considered rather crude. There are some beautiful exceptions to this statement, however. Mr. Goodrich.

Experiments should be made as to means of securing more uniformity of color of stucco, and the application of color to cement surfaces should receive more study. In Europe there are some beautiful examples of such work. While some progress has been made in devising effective and pleasing results in surface treatment of concrete work, there is still ample opportunity for improvement. All are familiar with the terazzo effect of good granolithic work, and most have seen surfaces which have been picked, axed, hammered, or treated with a sand blast. Some of the effects produced by washing, with heavy scrubbing while quite fresh, and of etching with weak acid are fairly pleasing, but probably the use of stucco in all its several varieties will eventually predominate. Colored tile can also be used, either in connection with stucco or in combination with selected aggregates, and treated with water or acid to bring out the color.

With the wider use of stucco, the necessity of securing a perfect bond between it and the foundation material will be more apparent. Several patented and secret processes are now in use, but none is beyond reproach, and in this there is a wide field for improvement. When eventually produced, such a bonding process should be used, even between parts of work done on succeeding days.

The engineer should pay more attention to the subject of forms. If specifications, hitherto, had not been so indefinite in regard to this item, fewer premature failures would have taken place. The practice to be followed in the erection of at least one important arch, of designing and specifying in detail all points as to the centering, can be followed profitably in lesser structures. With this element of risk removed, wherein the contractor has an opportunity to involve seriously the safety of the work by his faulty design and erection of falsework, and with the use of reinforcement in units designed by an engineer of long and wide experience, there is no reason why reinforced concrete work, eventually, should not become absolutely safe and fairly economical. Only one other point remains: the process of manufacture of concrete should be inspected as carefully as the production of structural steel and the grading of timber. Then the ideal will have been reached. Meanwhile, a careful study of the problem of forms is exceedingly profitable, because, in the cost of finished work, that of the labor and material thus involved often exceeds 40%, and sometimes approaches 75%, of the total cost; and, when carefully done, it may be reduced to 25%, where conditions are favorable. The rapid deterioration of all form material, because of wear and tear from repeated use, makes this item of cost high, even when the forms are

Mr. Goodrich. used a great many times. Doubtless, metal will eventually be used to a great extent, although wood will continue to be necessary for many parts. Staff is being used to excellent advantage, even for comparatively simple work, but it is not probable that its use will ever be very extensive. Some device which will remain a permanent part of the structure will probably be used, because these parts themselves can be moulded in shops where few forms are necessary, and the latter can be used a great number of times. In a similar way, the manufacture of structural members in a factory, by machine, or in such manner that few forms are necessary, will also be more widely developed where conditions make it possible.

In the labor element, a reduction can often be made by handling the forms in large units by derricks, and many devices are constantly being invented to do away with the costly work involved in the use of the saw, hammer, and nails. Bolts and a wrench, and work cut to length in a mill, are more nearly ideal. In all probability, less attention will soon be given to the finish of the work as it comes from the forms, because, for most classes of work, a better quality of surface finish is desirable, and more than enough money can be saved by using cruder forms, to cover the cost of such surface treatment.

Perhaps it is yet too early to discuss the subject of standardizing the sizes of beams, percentages of reinforcement, etc., but such a step will doubtless be taken just as soon as the art has outgrown its present really experimental stage.

Finally, a plea is made for more rational municipal building regulations and architects' specifications, in the framing of which engineers should have a hand. When the designing engineer and the man in charge of the furnishing of materials and erection of the work, are distinct individuals, better results will be attained; and owner, architect, engineer, and contractor will then all be striving for the most economical and artistic structure possible.

Mr. Thacher. Edwin Thacher, M. Am. Soc. C. E.—The effect of sea water upon Portland cement mortar and concrete, and upon steel embedded therein, is a subject which has received considerable study from American and foreign engineers and chemists, for several years past; but the investigations thus far made appear to have resulted in very little positive knowledge on the subject. There is considerable conflict of opinion between foreign experts themselves, and between foreign and American experts. What is most desired is to know why certain works have failed, and why other works have stood the tests of many years without any signs of decomposition or injury. When this is known it will be possible to write specifications for future work in which the chemical composition of the Portland cement used, and the mixture, manipulation, and placing of the concrete shall be such as will insure uniformly safe and satisfactory results. According to the best known

European writers on the subject, the use of Portland cement concrete in sea water is attended with great risk of chemical decomposition, and it is difficult and expensive to carry out their recommendations, in the way of precautions to be observed to overcome partially the risk of such a result, and their conclusions do not appear to be justified by experience in America during the past twenty years or more. M. Feret states that no cement has yet been found which will give absolute security against the decomposing action of sea water, that sulphuric acid is the principal cause of decomposition, that the cement should be low in alumina, and as low as possible in lime, that puzzolanic material is a valuable addition to the cement, that gypsum should be used sparingly, that fine sand used in mixing is injurious, and finally that the mortar must be such as will give a dense and impervious concrete. Mr. Thacher.

Dr. W. Michaëlis also recommends a completely impervious mixture, but differs from M. Feret in recommending that at least one-third of the sand used in mixing must be very fine. If the whole body of the concrete is not impervious, he says, this impervious layer should surround the porous kernel on all sides, and even underneath. He advises a cement rich in silica and as poor as possible in alumina and ferric oxide, also the addition of puzzolanic material to the cement.

M. Le Chatelier considers that the aluminous compounds in Portland cement are the direct cause of its disintegration in sea water, and advises that the alumina be replaced by oxide of iron. These foreign authorities do not give the chemical composition of a practical Portland cement, such as they would recommend for work in sea water, but satisfy themselves by condemning to a greater or less extent every constituent of Portland cement, except silica, and no manufacturer has yet succeeded in producing a satisfactory Portland cement containing this material only.

The writer has communicated with quite a number of American engineers who have had extensive experience in the use of concrete in sea water, and, almost without exception, the results have been highly satisfactory, notwithstanding the fact that very little precaution has been observed regarding the chemical composition of the cement, or the impermeability of the mixture; and the damage sustained has been confined mostly to points between high and low water, apparently due to mechanical causes more than to chemical decomposition. Joseph E. Kuhn, Major, Corps of Engineers, U. S. A., Norfolk, Va., is of the opinion that little apprehension of chemical action need be felt when standard and well-proved brands of seasoned cement are used. He mentions a sea wall built at Fort Monroe, just outside low water, fifteen years ago, of 1 : 4 : 8 concrete, with two-man stone incorporated. It has been exposed to wave action from storms, in which the beach sand was stirred up, and hurled against the wall with great

Mr. Thacher. force, also to tides and heavy swells from steamers. This mixture would naturally give a very porous concrete, but it is hard and tough, and no indications of chemical action or damage of any kind are noticeable except between high and low water, where the wall has in places been reduced in thickness as much as 4 in. This face has been repaired by 1 : 2½ : 4 concrete. Major Kuhn concludes that, by using a Portland cement of good quality, and a dense and strong facing layer when exposed to the action of the water, concrete-steel structures are as safe in salt as in fresh water.

C. W. Staniford, M. Am. Soc. C. E., Engineer in Chief, Department of Docks and Ferries, New York City, says:

"In the work of constructing the bulkhead or river walls around Manhattan, which has been in progress for the past 30 years, and is now being continued, no extra precautions are taken on account of the concrete being laid in sea water, except the use of first-class material and careful work."

Practically all the river wall, from low water up, has a granite face, backed by concrete in place, and heavy concrete blocks set in place with derricks from low water down, and the work is in perfect condition, after, in many cases, a period of 30 years. This applies also to concrete blocks laid above water at points not readily visible, and to concrete laid *en masse* above low water during the past 8 years, except in one location where, between low water and 2½ ft. above, the concrete shows some signs of pitting, and slight disintegration, which indicates a wear occasioned by the extreme pressure of ice during the long low-water slack.

S. W. Hoag, Jr., M. Am. Soc. C. E., Assistant Engineer, Department of Docks and Ferries, says:

"As regards chemical action, the experience in New York Harbor ought to be valuable, as our waters carry sewage probably not equalled in any smaller city. If chemical action counts for anything, I think it would in the harbor of New York along the North and East River waterfronts. I do not think that the possible deterioration from chemical action is likely to amount to much, unless the exposure is in close proximity to some chemical works. The above remarks are predicated on first-class material and workmanship."

A committee of the Association of Railway Superintendents of Bridges and Buildings made some investigation on the subject of concrete in sea water, and some of the replies to its inquiries are of interest and may be noted as follows:

A. Where there is no ice, concrete made in air with fresh water and sunk in sea water, works well. We would not deposit concrete direct into sea water. Disintegration more rapid than if deposited in blocks. Where there is large ice formation, concrete between high and low water will disintegrate from ⅛ to ¾ in. annually. Stone facing recommended.

B. Mix dry, no water, and deposit through chutes; depositing in blocks preferable; tides and frost have no appreciable effect on blocks. Mr. Thacher.

C. Concrete deposited direct into sea water gives perfectly satisfactory results if the materials and work are right. The cement should contain not more than 2% sulphuric tri-oxide. Concrete should not be leaner than 1 : 2 : 4. Stone facing preferred between high and low water.

D. A concrete pier at Warren, R. I., built about 25 years ago, of 1:3 mortar, is sound except between high and low tide, where it has worn away in places from 4 to 8 in., due to ice and tide. Current about 8 miles an hour.

The committee reports in favor of depositing concrete direct into sea water. It considers this method the cheapest and best, and is of the opinion that, with good material, properly mixed and handled, and with a granite face above low water, it will do good service.

Louis C. Sabin, M. Am. Soc. C. E., says:

"Many of the most eminent and conservative engineers consider that most failures are due to improper specifications, proportions, and manipulation, rather than to any defect in the cement."

William B. Mackenzie, Chief Engineer, Intercolonial Railway of Canada,* has used concrete in eight different places in clear sea water, and in every case disintegration has taken place between high and low tide, from ½ in. to 6 in. in depth. The concrete was generally 1 : 2 : 4. He learned that, where sea water carries sediment, the sediment penetrates into the pores and coats the surface, and no disintegration takes place.

Martin Murphy, Provincial Government Engineer, Nova Scotia,† has used concrete extensively for bridge piers since 1883. Some of the bridges were within the influence of the turbulent tides of the Bay of Fundy, most of them exposed to heavy drift ice, and all of them to extremes of temperature, yet but one failure can be recorded, and that, in his opinion, was due to careless workmanship.

J. G. Theban, Assoc. M. Am. Soc. C. E., Engineer in Charge of the Department of Bridges, Borough of the Bronx, New York City, has made an interesting experiment relating to the preservation of steel embedded in concrete in sea water. On August 24th, 1904, or somewhat more than three years ago, he sank in Pelham Bay, in 20 ft. of water, a shallow wooden box, in which ten steel Thacher bars, spaced at equal intervals, had been spiked to wooden cross-pieces. A bucket of 1:2:4 concrete was then lowered and dumped on and around these bars. After one month the box was raised and placed at low tide, where it was covered with sea water twice every 24 hours. The bars have been removed from time to time, and all have been found free

* *Engineering News*, October 31st, 1907.

† *Transactions*, Am. Soc. C. E., 1893.

Mr. Thacher. from rust. The speaker saw the last bar removed on January 1st, 1908, and it and also the spikes with which it was fastened were free from rust. Only a thin film of grout at most could find its way under the bars at points where they were in contact with the wood, but no rust could be discovered at these points.

Mr. S. E. Thompson. SANFORD E. THOMPSON, M. AM. SOC. C. E.—Columns represent the most vital part of a building, since the failure of one may cause the fall of the entire structure. The extreme variations in the fundamental assumptions in different private specifications, and also in city ordinances, make it imperative that the subject should receive more accurate and scientific treatment. As an illustration of the variety of ideas as to what constitutes safety, the extremes may be cited of certain city ordinances which permit a load not greater than 350 lb. per sq. in. on the column, and the value which is sometimes used in private practice of 1 000 lb. per sq. in. based on the entire cross-section of the column without appreciable reinforcement. The convincing argument, once addressed to the speaker by a prominent architect, for the adoption of the latter value in an important structure was that buildings in the Middle West had been designed and constructed with this unit compressive stress and were still standing.

The owners of a building frequently bring great pressure to bear upon the designer to reduce the size of the columns in the lower stories. This is not to be wondered at when it is considered that their dimensions may be 30 or 36 in. square, and thus require an appreciable amount of floor space.

It is well to recognize at the start that reinforced concrete columns, of a section which will compare favorably with steel, cannot yet be safely and economically constructed. A design after the principles followed by Professor Burr in the McGraw Building perhaps approaches a minimum section as closely as is possible, but, even here, only a low unit stress can be allowed on the steel without over-compressing the concrete. It may be laid down as a general principle that, not only is it cheaper to resist compressive stress with concrete than with steel, but also that concrete is cheaper than any combination which may be made of steel and concrete.

In order to reduce the size of concrete columns, four distinct methods have been used:

(1).—Rich proportions,

(2).—Vertical reinforcing steel,

(3).—Structural steel reinforcement,

(4).—Hooping or banding.

The use of a very rich mixture has much to commend it. The ultimate strength, by using a 1 : 1 mortar, may reach 5 000 lb. per sq. in.,* and the modulus of elasticity will also be so high that the deformation will be slight.

* "Tests of Metals," U. S. A., 1904, p. 386.

The introduction of vertical steel rods is indicated by the majority of tests* to be a satisfactory manner of increasing the strength, but the low stress which can be taken by the steel without permitting too great deformation of the concrete, makes this an expensive method, and the percentage of steel is limited, not only by economical considerations, but also because of the difficulty, especially when deformed rods are used, of placing the concrete around them properly. Mr. S. E. Thompson.

The use of structural-steel shapes for reinforcement has already been so fully considered in previous discussions that no further mention need be made of it here.

Hooping or banding, first introduced by Considère in France, perhaps more than any other method of reinforcement, has caught the popular eye, with a resulting tendency to great extremes of loading. For this reason, it behooves engineers to examine very carefully the underlying principles involved in this method of reinforcement and the results of experiments thus far made.

To illustrate the position taken by many conservative engineers on the subject of hooped columns, it may be worth while to study for a moment the real action which takes place under loading, as shown both by theory and tests.

When a load is placed upon the top of any column, it causes vertical compression or deformation with, at the same time, a lateral expansion. The lateral expansion in concrete columns, as shown by tests upon plain and upon reinforced columns by Mr. J. E. Howard at the Watertown Arsenal,† and by A. N. Talbot, M. Am. Soc. C. E., at the University of Illinois,‡ is at first very small. Any stress produced in the steel hooping must be proportional to its deformation or stretching; hence, with small lateral expansion of the concrete, there can be but slight stress in the hoops. For this reason, and also because of the initial shrinkage of the concrete, which the lateral expansion must first overcome, scarcely any stress or pull comes upon the hoops until the concrete within them has reached a loading equal to the breaking load in plain concrete. As this load is approached, the modulus of elasticity of the concrete becomes very much lower, and consequently both the vertical and lateral deformations become much greater. Then, and not until then, does the lateral pressure begin to act appreciably upon the hoops. In other words, up to the very crushing strength of plain concrete, the value of the hooping is actually negligible. From then on, the reinforcement takes practically all the load, and a high ultimate strength may be attained, although coincident with great shortening of the column.

It is evident that, if concrete is confined in a tube, advantage can be taken of the added strength due to the tube. On the other hand,

* "Tests of Metals," U. S. A., 1904, p. 386; 1905, p. 377.

† "Tests of Metals," U. S. A., 1905, pp. 293–336.

‡ *Proceedings*, American Society for Testing Materials, Vol. VII, 1907, p. 382.

Mr. S. E. Thompson. if hoops are very far apart, it is evident that the concrete, when it reaches a stress equal to the strength of plain concrete, will be thrust out between the hoops. Professor Talbot's tests,* using a gradually increasing load, indicate that, with ordinary spacing (the effect of different hoop spacing is not definitely discussed in the advance report of the tests thus far made), the hoops will effectually restrain the concrete within them. The effect of repeated and continued loading was not investigated by him.

Even with the concrete restrained within the hoops, the shell of concrete outside of them, which is necessary for fire-proofing and for the protection of the steel, begins to crack and peel at about the same load as that which will cause complete failure in unreinforced concrete. Professor Talbot, in fact, states as a general proposition that: "Cracking and peeling of the concrete appear at loads corresponding to the ultimate strength of the concrete."

This applies to hoops held rigidly. If the hooping is in short spiral sections, with the ends of the wire or rods simply lapped or insecurely fastened together, it follows, inevitably, that the spiral must give way and unwind as soon as it is exposed by the stripping of the concrete from the steel. Consequently, the breaking strength of a column hooped in this way will only be effectively equal to that of an unreinforced column.

The modulus of elasticity of the concrete within any hooping, after the point of exterior cracking is reached, drops very rapidly, reaching, in the two diagrams shown in Professor Talbot's paper, less than 300 000 lb. per sq. in., even at 2 000 lb. per sq. in. stress in the column, the deformation becoming so great, in fact, that any vertical reinforcing steel, unless in such quantity as to take the full load, would pass its elastic limit soon after the point of first crack,† and by its buckling increase the surface peeling. Furthermore, from the appearance of the deformation curve, the concrete itself would seem to be in somewhat the same condition as is steel after it has passed its elastic limit.

When it is considered that the usual practice in concrete column design takes no definite account of eccentric loading, or of bending caused by expansion and contraction of floor and wall areas, and that inferior spots may occur in any concrete, through careless mixing or placing, it appears that the greatest care should be exercised in fixing the unit stresses in hooped columns.

Tentative conclusions with regard to hooped column design at the present stage of tests may be summarized as follows:

(1).—Hooping, if properly applied, increases the ultimate breaking strength under a single loading to double or treble the breaking strength of a plain column.

* *Proceedings*, American Society for Testing Materials, Vol. VII, 1907, p. 382.

† See also Mr. Howard's tests, in "Tests of Metals," U. S. A.

(2).—The surface of concrete outside of the hooping will begin to crack at a loading corresponding to the breaking load of an unhooped concrete column. Mr. S. E. Thompson.

(3).—Hooping, if not continuous or rigid, will peel off with surface concrete, so that the effective strength of the column will be no greater than a similar one of plain concrete.

(4).—The total vertical deformation of a hooped column is so great at the period of first external crack that any vertical steel, unless designed to carry the entire load, is stressed beyond its safe strength.

(5).—The ultimate breaking strength of a hooped column is no measure of its safe strength, and formulas based on the ultimate strength are useless.

(6).—With the present knowledge, based on tests in America and abroad, the safe load allowed on hooped columns should be but slightly, if any, greater than on similar columns without hooping.

In spite of the favorable reports which have resulted from the European experiments upon hooped concrete, it seems impossible to ignore the additional facts brought out by American tests. Before the hooping acts, the concrete has begun to crush, and any structural material which has begun to crush is unsafe.

William H. Burr, M. Am. Soc. C. E.—Statements made in the course of this discussion appear to indicate that, in such a general treatment of the entire concrete-steel question as this, some features at least of the use of concrete-steel should receive a more careful consideration than would otherwise seem necessary, in view of recent successful constructions. Mr. Burr.

Caution has been urged against using a unit working strees in the concrete-steel combination exceeding one-tenth of the ultimate resistance of plain concrete, such caution being based upon some of the results obtained in the tests of 12-in. cubes of 1 : 2 : 4 concrete at the Watertown Arsenal. In the consideration of experimental results attained by testing concrete cubes, it is of the utmost importance to know completely all the circumstances of such tests, including the preliminary tests of the cement used and the gradations of the sand and gravel or broken stone aggregate. If a 1 : 2 : 4 concrete should be mixed relatively dry, and allowed to set in air and remain in a dry building, from the time of its mixture until testing, the results at the end of any usual test period might and probably would be quite different from those found at the end of the same period with a comparatively wet mixture kept constantly moist by sprinkling for a month or longer subsequent to mixing. Other conditions equally productive of varying results can be named, besides the quality of the cement.

As a matter of fact, there are numerous tests of 12-in. cubes of 1 : 2 : 4 concrete in the records of the Watertown Arsenal which show an ultimate compressive resistance of from 3 000 to 3 600 lb. per sq. in., Mr. Burr. and even more, at the end of three months, with increasing resistances for longer periods. It is a conservative statement to say that well-balanced 1 : 2 : 4 concrete, made with a good quality of Portland cement, may give from 2 700 to 3 000 lb. per sq. in., at the end of three months, with ultimate resistance continually increasing with age. Such concrete may properly and safely be expected to reach ultimate resistances of from 4 000 to 4 500 lb. per sq. in. at the end of a year, results which are justified by extended experience both in America and in Europe.

It is difficult to assign any satisfactory reason for the use of a working stress as low as one-tenth the ultimate resistance of concrete. It is true that there are occasional cases of retrogression, but, with the high grade of Portland cement available from the most reputable producers both in America and abroad, it is reasonable to state that, with the usual engineering inspection to which the best classes of public work are now subjected, cement with retrogressive qualities may confidently be excluded. No engineer at the present time need apprehend sensible difficulty in securing Portland cement the resistance or strength of which will go on increasing indefinitely, and, having reached its maximum, hold it. Under such conditions, a working resistance or permissible intensity of compression in concrete of one-fifth to one-sixth of its ultimate, certainly affords all margin of safety required for engineering works of the best class. Indeed, probably a somewhat higher working stress than that is justified in large structures of reinforced concrete, especially where the reinforcement is of such a character as to give material lateral support to the concrete. This subject is illustrated effectively by the report of a French Government Commission bearing upon the use of reinforced concrete in France. In that report the limit of compressive stresses allowed in reinforced concrete is two-sevenths of the ultimate crushing resistance of the same concrete as determined by tests of plain cubes at the age of 90 days, with the further provision that this two-sevenths may be increased to three-fifths if the longitudinal and transverse reinforcements comply with certain prescribed conditions. This French provision would yield a safe working stress with first-class reinforced concrete work but little if any under 900 lb. per sq. in. The Bureau of Buildings of the Borough of Manhattan, New York City, therefore, has taken a safe and satisfactory course in allowing 750 lb. per sq. in. in such reinforced concrete work as the Thirty-ninth Street Building in the City of New York. In fact, this latter working resistance is conservative for the best class of reinforced concrete work of the present time.

The apprehension regarding the reliability and durability of reinforced concrete work as shown by timorous expressions reminds one strongly of the attitude which some engineers and others used to take toward structural steel when it first came into use, twenty-five or more

years ago. It is remarkable, when one reflects that structural steel is Mr. Burr.
practically the only structural metal which we now possess, that at the period to which allusion is made it was frequently argued out of any future possibility of use, as compared with such a reliable material as wrought iron, in consequence of the erratic behavior which some structural steel members exhibited at that time. Fine cracks, started at a punched rivet hole or sheared edge, would sometimes extend far enough to destroy the reliable carrying power of a channel or angle or other member. Such disclosures, with other erratic experiences, were sources of keen apprehension to many; others, however, believed them to be merely passing phases of difficulty, which attend the introduction of all new materials and processes, and careful study, with intelligent shop manipulations, has shown them to be such. Experience, of course, has more than justified the advocates of structural steel, and that metal has now proved to be, not only reliable, but by far the best structural material ever yet made available to the engineer for a wide range of purposes; indeed, wrought iron is no longer available for structural purposes, nor has it been for a number of years.

Reinforced concrete is passing through a similar phase. It is admirably adapted to a great range of structural purposes. Much has already been learned in regard to it, but extending experience will disclose a widening fund of information of value to the engineer in its intelligent application. As a matter of fact, more is actually known about the carrying capacity or the ultimate resistance of concrete-steel members than about the carrying capacity of steel columns, as determined by actual tests. There has already been accumulated a great mass of well-considered and well-digested experimental data regarding the design and construction of both concrete-steel beams and columns, although there is need of many additional tests of some of the latest and best forms of concrete-steel columns. On the other hand, there are almost no tests of full-sized steel-built columns, made in such a way as to disclose some of the most important fundamental principles of design. In the present condition of actual tests of the two classes of members, it is reasonable to believe that there may be at least as much confidence attached to the computed ultimate carrying capacity of both reinforced concrete beams and columns as now built under the best design as can be attached to the computed ultimate carrying capacity of steel columns. Engineers have been so accustomed to design and construct built-steel columns in their every-day work that few ever reflect on the paucity, or even absence, of experimental data on which to base a rational and competent design of such members.

All that reinforced concrete construction needs for reliable results is good cement, good inspection, and intelligent design, which, up to the present time, it has not always had. It is one of the most useful build-

Mr. Burr. ing materials which the engineer has yet had at his command, but it must be dealt with in a manner suitable to any first-class engineering work. There must be rational design, intelligent and effective handling, and good inspection, precisely as with structural steel; and, under such conditions, reliable and durable results may confidently be expected.

Mr. T. K. Thomson. T. Kennard Thomson, M. Am. Soc. C. E. (by letter).—Reinforced concrete, like all other good things, should be protected from its friends. Many young men, having very little knowledge of steel or concrete, have formed companies to build reinforced concrete structures, and one of the first things with which they come in contact is the fact that to obtain a contract they must bid low, another is the necessity of showing the advantages of reinforced concrete over structural steel, and, as the question of cost is the one that appeals most forcibly to the majority of purchasers, they try to design their structure so that the cost will be as low as, or not much higher than, plain steel. One of the methods of doing this is to use fiber strains which are higher than a good bridge or building designer is accustomed to allow.

Many who design reinforced concrete strain their steel bars up to 20 000 or 22 000 lb. per sq. in.—strains which bridge engineers have countenanced only for very long spans, that is, those where the dead loads are large compared with the live load. The recent collapse at Quebec, where it was intended to allow a possible strain of 24 000 lb., and where, owing to faulty detailing, the structure failed at about 18 000 lb., has made many doubt the wisdom of allowing such high combinations of strains (even if only possible), which are hardly likely to occur on any span.

It is practically impossible to ascertain the exact elastic limit of the built-up members of a bridge—due to imperfections of workmanship, material, etc., etc., and therefore it is decidedly unsafe to approach too close to the elastic limit, in estimating the stresses, or to assume that the elastic limit of the test bar is the elastic limit of the full-sized member. There is no reason for allowing higher fiber strains in reinforced concrete than in plain steel, as there are many elements of uncertainty in the former which do not occur in the latter, because far more care is required in the field work and inspection of concrete.

One source of danger, "dry concrete," is rapidly disappearing, for dry concrete practically required an inspector for each laborer, in order to ensure proper ramming, whereas wet concrete will almost ram itself—the only danger being the risk of letting the water escape, thus carrying the cement with it. A 4-in. reinforced concrete wall in New York City was recently removed, when it was found that there was no bond between the steel and the concrete. Not knowing the conditions under which the wall was built, it can only be assumed that the concrete must have been put in too dry.

After the design for a reinforced concrete structure has been made, the three most important considerations are the proper handling of the material, protection from rust, and—more important still—protection from electrolysis. Mr. T. K. Thomson.

In ordinary structures where large masses of concrete are used, buckets containing 2 cu. yd. can be dumped in place, and, if wet, require almost no handling, but, in most reinforced concrete structures comparatively little material is used, and the utmost care is required in handling. In many cases the extra sum paid for labor plus the reinforcement makes the work cost as much as, or more than, a good plain concrete structure containing twice as much concrete, in which case it is better to put one's money into the material rather than into the labor.

Much difference of opinion exists as to whether or not concrete can be made water-tight. The writer's experience has been that it can be, but may not always be, owing to carelessness, and that the mixture should always be rich, that is 1 part of cement to 2 parts of sand, with as much stone as can be covered.

The writer has seen 24-in. I-beams, which had been buried in concrete under the city streets for five or six years, taken out cleaner than they were put in, and in many places showing the original blue shop scale—no paint or oil having been used. In a few isolated places, however, these beams were pitted with rust, showing where the water had found its way to them. It is well known that paint and oil interfere with the bond between steel and concrete. Steel caissons and coffer-dams have been sunk in quicksand in New York City, which, when exposed some seven years later, showed not the slightest evidence of rust.

The writer has removed old steel and cast-iron columns, which had been bedded in concrete and brickwork for years, which showed absolutely no sign of rust. Therefore, in large buildings, carefully constructed, it would seem that there is almost no danger of rust, but it is doubtful if this is true of reinforced concrete bridges, where thin layers of concrete are used, for it has been found very difficult to put in a roadway floor which will not allow any water to percolate through.

The danger from electrolysis is probably very much greater than from rust, and its action is more rapid. There have been cases in New York City where a certain amount of current has been grounded through the steel in foundations buried in concrete, and the steel has been absolutely destroyed. For foundations, it would seem to be safer, in many cases, to rely on mass concrete rather than on thin slabs of reinforced concrete, which cost almost as much in the first place. Of course, in cases where water can reach the embedded steel and carry an electric current with it, the danger is very great, and very certain in its action.

Mr. T. K. Thomson. It is probably true that steel in reinforced concrete is much less likely to rust than in a steel structure covered with the best paint, but the latter can be inspected and the former cannot.

In short, the best friends of reinforced concrete should restrict its use to its legitimate spheres, which are many.

Mr. Krellwitz. D. W. KRELLWITZ, JUN. AM. SOC. C. E. (by letter).—Probably the most novel form in which reinforced concrete has been used is in transmission-line structures.

One case is the 12-mile transmission, for many thousand horsepower at high voltage, from Decew Falls to Welland, Ont., Canada, for which a line with reinforced concrete towers was completed in 1907. Another example is the line of towers* carrying transmission circuits of high voltage to St. Catharines, Ont. These towers are at present the highest monoliths that have ever been erected, being considerably more than twice the height of any of the famous Cleopatra needles.

For the elevations above ground at which it is common to support the conductors of transmission lines (from 25 to 45 ft.), a reinforced concrete tower, in various parts of the United States and Canada, will cost from one to five times as much as a wooden pole. It follows at once from this fact that there must be cogent reasons, apart from the matter of first cost, if the substitution of reinforced concrete towers for wooden poles on transmission lines is to be justified on economical grounds. The electric transmission of energy from distant water-powers to important centers of population has grown from the most humble beginnings to the delivery of hundreds of thousands of horsepower in the service of millions of people, and the lines for some of this work are supported on reinforced concrete towers. Electrical supply in Buffalo, N. Y., to the amount of 30 000 h.p., depends entirely on the circuits from Niagara Falls which operate at 22 000 volts and, at Tonawanda, N. Y., are supported on reinforced concrete.

In the operation of high-voltage transmissions, during the past, some difficulties have been met, but they have not been so serious as to prevent satisfactory service. Nevertheless, it is being urged that certain impediments, met in the operation of transmission systems, would be much reduced by the substitution of reinforced concrete for wooden poles, and it is even suggested that perhaps the first cost, and probably the last cost, of a transmission line of this kind would be less than with wood for supports. The argument for reinforced concrete in the matter of costs is that, while a tower requires a larger investment than a wooden pole, yet the smaller number of towers may reduce the entire outlay to about the same as for wood. More than this, it is said that the lower depreciation and maintenance charges on rein-

* Described by the writer in his paper on "Reinforced Concrete Towers," *Proceedings*, Am. Soc. C. E., Vol. XXXIII, p. 572.

forced concrete supports will make their final cost less than that of wooden poles. Mr. Krellwitz.

One advantage of reinforced concrete over wood is that it will not burn, and is probably not subject to destruction by lightning. The fact that reinforced concrete will not burn may make it desirable in places where a long line passes over a territory covered with brush or timber. In tropical countries where insects rapidly destroy wood the use of reinforced concrete, even at a much greater cost, might be highly desirable.

Guy B. Waite, M. Am. Soc. C. E. (by letter).—Reinforced concrete has its uses, and, up to the present, there are few things to which it has not been found to apply. Mr. Waite.

Public opinion has changed within a very few years from serious doubt about concrete being good for anything to that now held, that it is good for everything.

Friends of concrete can do much damage to the cause by insisting on pointing out personal achievements where actually failures should have been recorded.

It is not possible for one man to formulate a statement as to the universal adaptability of concrete for a given purpose, in all localities from New York to California, without a knowledge of all the conditions in each locality. The popular idea seems to be in most places that concrete should be used for buildings because it is so much cheaper than wood, and that in concrete construction the cost of almost anything is very trifling. This view has recently been strengthened by one of our most distinguished and respected prophets, who promises to see that a two-family house, if it is desired, is turned out complete in a few hours. It is to be regretted that the necessary details to enable others to benefit by his discovery are not disclosed.

Concrete has its *pros* and *cons* which could be stretched in long columns, thus, for example:

Against concrete:	For concrete:
Not good in tension;	Good in compression;
Requires forms;	Good for limited amount of shear;
Requires time to set;	Strength improves with age;
Difficult to tear down—or to fall down;	Economical where forms are simple;
Etc.	Is monolithic;
	Etc.

Stone concrete, mixed in the proportion of 1 : 2 : 4, can be laid down in almost any part of a fair-sized building, with profit, at 30 cents per cu. ft., not including forms.

An average steel column, for a corresponding building, could be erected, at a profit, for $90 per ton.

Mr. Waite. Average steel floor beams and girders, of standard sections, will cost $60 per ton.

Beginning with the supporting columns of a building, a properly reinforced concrete column (conservatively estimated) will carry an average of 750 lb. per sq. in. On the other hand, suppose the corresponding steel column to be strengthened so that it carries an average of 16 000 lb. per sq. in. Then the required amount of materials in the two cases will be as 750 to 16 000, or about as 1 to 21.

The costs of corresponding sections of the two materials, on the foregoing assumption, will be 30 cents and $21.96, or as 1 to 73. Therefore the relative costs of the sections of each material to carry any unit loading will be as 21 to 73, or about 1 to 3½ in favor of the concrete column.

From here on, practical experiences will become useful to decide whether the percentage of 1 to 3½ in favor of concrete is the ultimate ratio of cost, when everything is considered.

Even engineers prejudiced in favor of steel will perhaps concede that for this steel column about 12 to 15% will have to be added to the carrying shaft for fittings, etc. (and in the case of latticed columns much more than this), which added amount of steel will be sufficient to reinforce the concrete column—according to the accepted theory of hooping. Further, if the steel column is to be protected from rust as well as fire, the forms and the concrete material for such fire-proofing will be substantially the same in each case.

Without taking time to go further into details, it would appear that concrete properly used in the form of columns would certainly have the better of the argument, when comparing costs.

The speed of erection sometimes becomes important, and, where the reinforcement to the concrete column is made in the form of an independent carrier, construction can proceed approximately as rapidly as in all-steel construction.

The next objection to the concrete column is naturally the increased size. This objection cannot be raised consistently except in normal buildings more than six stories high, and this in the lower stories only. If the buildings be eight stories high, the size of the columns will only be abnormally large in the two lower stories, etc. A well-constructed building, six stories high, should have columns of steel of not less than a certain outward dimension, in order to give proper rigidity to provide against eccentric loading, etc., and such steel columns, when fire-proof, will be substantially of the size of the solid reinforced concrete column, with an equivalent strength and rigidity.

With development along the lines of improved reinforcement for the concrete, in reinforced concrete columns, it is believed that in the future the sizes of concrete columns can be reduced to meet all requirements.

Concrete, in connection with reinforced floors, is usually taken with a working stress of 500 lb. extreme fiber strain. Mr. Waite.

With the usual **T**-section of floor construction, an average working load on the entire sectional area for compression can be taken safely at 450 lb. per sq. ft. Estimating the steel beams to take this load at the unit prices set forth above, the comparative costs of concrete and steel would be about as 35 to 49, showing an economy in favor of concrete, other things being the same. But, in this item of floor construction, the concrete floors have to be installed, even when the all-steel construction is used, in order to coat the steelwork and protect it against rust as well as fire. So that, in reality, if the concrete cost as much or more than the steel doing the compression work, whatever is saved by putting this concrete to work is a clear gain, other things being the same.

Other things do not always remain the same, however, and it is necessary to consider the form work for the reinforced concrete construction and the forms for the fire-proofing, used when steel construction carries all the floor loads.

With the steel beams and girders giving the working lines and offering ample supports for the wood forms, the modern system of forms for fire-proofing is very materially less than where much stronger independent framings and supports must be carefully leveled and supported for the reception of the concrete, in reinforced work.

Where the forms can be made in the same general manner as fireproofing (as in some improved systems of reinforced concrete), the discussion of the relative costs of forms can be dropped, and one may proceed to compare other items in the relative costs of concrete and steel constructions. Now, assuming that forms are the same, and that the concrete is used as a fire-proofing in each case, showing a gain for every bit of the concrete in the reinforced scheme (which is not obtained in the fire-proof scheme), then, if it is not clear that there is economy in the reinforced scheme, it is because the concrete can be made cheaper in the one construction than in the other. The floor slabs will have the same loads to carry when acting as carriers from beam to beam: the concrete, to be an effective protection to the steel against deterioration, must be rich, so that, if the ultimate objects are to be accomplished, the concrete should be substantially the same in either case.

Without making the inquiry more monotonous, it would appear that, in floors, concrete reinforced construction shows an economy in proportion to the amount of steel it is able to replace. So that, where economy alone is the object, a good steel job is necessary. In light constructions (such as dwellings and hotels), where but little steel is necessary, one cannot save as much by using concrete as where the steel is heavier; and the saving continues to increase with the amount of steel to be saved.

Mr. Waite. In the foregoing comparison of relative costs in column and floor constructions the form work is similar whether reinforced concrete or steel and fire-proofing be used. In monolithic wall and partition construction the comparison is disadvantageous when it is considered that brick walls and partitions are laid rapidly and without the inconveniences of forms, and that double forms are necessary for concrete. Further, it is very much more difficult to place the forms for straight walls or partitions than for either columns or floors. The wall forms are not easily held plumb, or in straight lines. The removal of the forms for walls is also much more difficult than for either columns or floors. The cost of common brick and mortar amounts to about 18 cents per cu. ft., and the cost of the materials composing concrete is just about the same. So that the cost of laying the brickwork, for walls of the same thickness, must be balanced by the cost of the double forms and placing the concrete.

It is not intended to burden the reader with descriptions of the difficulties of constructing form work for vertical structures; but, to anyone having much experience, it must be evident that such difficulties must be met. Economy in wall work must be looked for only in heavy work, where the quantity of material placed for any given form is sufficient to pay for it, without materially affecting the cost of the concrete.

When no finish is looked for on the concrete work, rough forms may be placed for from 4 to 5 cents per sq. ft. on each side of the wall; but, for good form work, the cost will run from 7 to 10 cents per sq. ft. on each side.

Concrete walls will be erected. They are an improved construction, and can be handled conveniently in connection with other concrete work in a building. The object of writing what seems to the writer to be the truth about their construction is that economy in their construction should be looked for along other lines than making double forms for the reception of the concrete. It is believed that there will soon be other means of erecting concrete walls and partitions, in which concrete can more than compete with the rapid and economical brick wall.

Mr. Slocum. C. L. Slocum, Assoc. M. Am. Soc. C. E. (by letter).—The science and use of reinforced concrete in the United States appears to be in its earlier stages, as compared with a longer and more thorough acquaintance and varied use in Europe. Only recently its wide application in America has been appreciated in the manifold kinds of construction which are now seen almost everywhere. Generally speaking, theory and practice do not seem to be as closely allied in America as abroad. American engineers have not learned, as well as European engineers, that knowledge of the constituent materials and thoroughness in details of construction are more important than records in speed of erection. Like everything new, much opposition, in the nature of incredulity, has to be overcome. For its age, reinforced concrete is fairly well understood, and it may be said that its newness is its greatest fault. The change in the field of design caused by the knowledge of the properties and capabilities of the combination of concrete and steel is now general, and is somewhat in the nature of a revolution in construction. There is hardly a department or particular sphere of construction which has not been changed by it. Homely and incongruous constructions in wood, steel, and stone, and other types of construction too highly commercialized, may now, at reasonable cost, give place to permanent structures, which are pleasing to the eye and are harmonious additions to the locality or landscape. Many types of construction in vogue or considered as good standard practice two or three years ago are now appropriately known or should be known as a part of the history of construction. Mr. Slocum.

If reinforced concrete can be accorded the same conscientious treatment and scrutiny as steel receives, there need be no hesitation about making the change to more permanent and artistic structures, which, if honestly built, will cause no concern or attention after they are put in place. The mature design and construction of steelwork to-day is accomplished by experts in that line, and these are necessary accompaniments of its age and maturity. The use of reinforced concrete needs more rigid inspection in construction, for it is idle to apply carefully intricate formulas to designs which when constructed suffer for want of expert superintendence and experienced labor.

In the realm of bridge construction, where ample depth is available, there is not much doubt as to its economy. This still holds true for spans with comparatively shallow depth, and with light loads, in the nature of moving concentrations. For crossings with little depth of structure available, with heavy moving concentrations, its sphere of usefulness is at present advisedly confined to short spans. However, even floor spans, up to and from 30 to 40 ft., under heavy concentrations, with less than the ordinary depth, can well be investigated. Fabricated units, of simple shapes, as reinforcement, with little or no shopwork, will afford ample stiffness. Theoretical analysis, however, must show that the unit stresses in the concrete and steel are well within the fatigue limits. Continuous framework or an interdependent system of units, easily put together, as reinforcements, but rigid in itself when complete, would seem to afford as much stiffness as steel beams bedded in concrete, which are generally calculated as carrying all the loads independently. In true reinforced work the homogeneous combination of the concrete and steel is the supporting resistance. The full use of the two materials to carry the loads must be more economical than the use of the one which has the concrete merely as a protection. The writer doubts the economy of hybrid construction.

Mr. Slocum. With old or much-used material the internal or molecular structure and properties of which have been changed, or are in doubt, half the usual unit stress allowed for new reinforcement, or doubling the usual economical percentage of reinforcement, would seem to be safe and advisable. The use of old material, of cumbersome, as well as dubious section, of, say, 4 or 5 sq. in. net section, such as old rails, is inadvisable for floor bridges in total length greater than the commercial rail lengths; because attempts to develop such sections in tension are too expensive, and are somewhat abortive.

In a series of short, independent, self-supporting arches of reinforced concrete, which are very flat, and are practically carried on continuous columns, the writer has used the cantilever method in finding the stresses in the constituent materials, and has proportioned the steel accordingly; in other words, he has considered the middle third of each span as carried by the end thirds. These arches were calculated for the heaviest moving concentrations for highways. In beam and slab bridges, carrying heavy trolley concentrations, where the design is somewhat hampered for depth, higher percentages of steel and double reinforcement may have economical advantage.

In current American practice, more time can be allowed to good advantage for this construction to attain mature strength rather than use a green structure prematurely and perhaps lessen the efficiency of the bond. Collections of materials of construction or equipment, sometimes inadvertently placed on new work, give concentrations for which the design is not calculated, and, if the work is not of sufficient age, much damage may be done, and may not be evident until some time after. Such consequent weakness may be brought out by fatigue, which, under normal conditions, could not be explained. From observation, competent, well-paid superintendence and experienced workmen of the best class give the strongest structure and the one that fulfills all the conditions of economy.

As compared with the usual heavy masonry arches of gravity section, the comparatively light reinforced arches give more and greater vibrations under moving loads, principally on account of much less bulk weight of structure. Can reinforced concrete work vibrate with the same impunity as steelwork? The writer thinks it can, if the working stresses are not too high, but are well within the fatigue limits. Much interesting and instructive information could be obtained by measuring the number of vibrations and their amplitude on bridges of different types under different kinds and speeds of rolling loads. Under any conditions, crossings of shallow floor construction can well be tested for unusual loads, and consequent deflection, if any.

Other properties and characteristics being satisfactory, a greater proportion of finely-ground cement, with a graded aggregate will, with safety, give reinforced concrete design and construction its bold quality, which distinguishes it. Too little attention is paid to the compactness or density of the mixture. The result of a few simple and inexpensive experiments in the measurement of voids, taking a comparatively short time to perform, will give a cheaper and stronger concrete. In reinforced work, such preliminary investigations are productive of economy. Mr. Slocum.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

CHARLES PAINE, PAST-PRESIDENT, AM. SOC. C. E.*

DIED JULY 4TH, 1906.

Charles Paine was born in Haverhill, New Hampshire, on April 25th, 1830, and died at Tenafly, New Jersey, on July 4th, 1906. Almost his entire working life, which began when he was 14 years old, was spent in railroad service, and the building of his professional position was in that period when the railroad art was still primitive and when the opportunities for education as a Civil Engineer, in the United States, were confined almost to actual work in the office and in the field.

Mr. Paine was descended from Stephen Paine, who came to the United States from England in 1638, and his family remained in New England for eight generations. He belonged to a fine, substantial stock, and some of his ancestors were distinguished.

Mr. Paine's school education was quite limited. In 1839, he entered the College of the Order of St. Sulpice, in Montreal, where he remained for two years and where he acquired that part of his school education which always seemed to him the most valuable, *viz.*, a good grounding in French and Latin and in the elements of what old-fashioned people call "polite education." He spent a year at an academy in Meriden, New Hampshire, and two years in New York at a school of which Mr. Charles Coudert was principal. This Mr. Coudert was the father of the late Charles and Frederic Coudert, eminent lawyers of New York and Paris. From an uncle, William T. Porter, editor and proprietor of the *New York Spirit of the Times*, young Paine, during these two years, was permitted certain opportunities to see life and people, and he always attached a good deal of value (and not without reason, perhaps) to the hours which he passed in the offices of that newspaper in the company of the wits, men about town, and famous actors and actresses. He says that in that brief period he "drank in a love of fine things in conduct, in art, in literature, and in manners which has continued a joy to me throughout my life."

In the spring of 1844, Paine's uncle, Governor Charles Paine, of Vermont, took the lad into the counting-room of his broadcloth factory, where he remained until August, 1845. Then he entered the service of the Vermont Central Railroad, the surveys for which had just been commenced. Of this enterprise, Governor Paine was Presi-

* Memoir prepared by Edward P. North and H. G. Prout, Members, Am. Soc. C. E.

dent. Here he began as rodman in the corps of Charles Brown, one of the old engineers of the period, who enjoyed a fine reputation. Other young men, afterward distinguished, who were associated with Paine in this work at this time, were the late S. M. Felton, M. Am. Soc. C. E., afterward President of the Philadelphia, Wilmington and Baltimore Railroad and of the Pennsylvania Steel Company; Mr. Charles Collins, afterward Chief Engineer of the Lake Shore and Michigan Southern; Mr. Carpenter, afterward United States Senator from Wisconsin; and Dr. Edward H. Williams, afterward General Superintendent of the Pennsylvania Division of the Pennsylvania Railroad, and a member of the firm of Burnham, Parry, Williams and Company, owners of the Baldwin Locomotive Works. Mr. Paine enjoyed the close friendship of all these men until their lives ended, and it is related that at the time of the Chicago fire, in 1871, Dr. Williams and his wife, knowing that the Paines lived in Chicago, immediately shipped by express from Philadelphia a complete outfit of clothes for each member of the Paine family, without stopping to ask if the clothes were needed.

In the autumn of 1847, the Vermont Central was completed into Northfield, and, for a very short time, young Paine got a chance to fire a locomotive, which he always regarded as one of the most valuable experiences of his life. The following winter he spent in the drafting rooms of Brown and Hastings, Civil Engineers, in Boston, and of Hinkley and Drury's Locomotive Works, where he made quite complete drawings of all the parts of a locomotive.

In 1848, Mr. Paine took charge of a division of the Vermont and Canada Railroad, under Henry R. Campbell, Chief Engineer. This road was completed in 1850. Mr. Paine then went to Montreal and took charge of the contracts of H. R. Campbell for building a railroad from Rouses Point to St. Johns, and for building a branch line from St. Lambert to intersect with the line of railway between St. Johns and La Prairie. At this time he also had charge of the building of docks at Moffatt's Island, opposite Montreal.

It will be seen that the young man had considerable responsibilities before he was of age, and he appears to have been in no way reluctant to assume still other responsibilities, for on May 13th, 1851, less than a month after reaching his majority, he was married to Olivia Blodgett Hebard, of Chelsea, Vermont. His wife belonged also to one of the most solid New England families. She was a woman of great cultivation of mind and of strong and beautiful character, and they lived together in the greatest happiness until Mrs. Paine's death in the summer of 1897. They had six children, and four sons now survive.

In 1855, Mr. Paine moved to Wisconsin, where he became Chief Engineer of the Beaver Dam and Baraboo Railroad, and of the Fox

River Valley Railroad, neither of which enterprises got beyond the stage of grading the roadbed, because of the great panic of 1857. In August, 1858, Mr. Paine became Superintendent of the Western Division of the Michigan Southern and Northern Indiana Railroad, which road was at that time five months behind in its pay-roll and physically pretty nearly a wreck. The local nickname for the road was the "Miserably Slow and Nearly Insolvent Railroad." These conditions, however, were not peculiar to that railroad in the year 1858. Mr. Paine's connection with this railroad and its lineal successors continued for twenty-three years. In January, 1864, he was made Chief Engineer of the road, and on March 1st, 1872, he became General Superintendent of the Lake Shore and Michigan Southern Railway. While in charge of this road he made such improvements and economies that by 1876 he had demonstrated his ability to carry freight for 4 mills per ton-mile, and from this, at the time, small sum, pay all the costs except for improvements, dividends, and interest on the bonded debt.

He remained Superintendent of the Lake Shore and Michigan Southern until he was appointed General Manager of the New York, West Shore and Buffalo, in August, 1881. He organized and carried through the building of this road, and upon its bankruptcy he found himself with health impaired and with the savings of his lifetime gone, for he himself had invested in the securities of the enterprise in which he believed enthusiastically.

In order to get himself in condition to rebuild his fortunes, he adopted the novel and bold scheme of traveling in Europe for a year on borrowed money. The remedy was characteristic and highly successful, and until the day of his death he never suffered another illness.

He served for a short time as the General Superintendent of the New York, Pennsylvania and Ohio Railroad, and for a few months as Second Vice-President of the Erie, and then he went to Pittsburg to help Mr. Westinghouse in developing the natural gas industry through the Philadelphia Company. There he remained until December, 1890, he having had active executive charge of the company.

He returned to New York at the end of 1890, and opened an office as Consulting Engineer, which office he maintained until 1899; part of which time, however, he was General Manager of the Union Steamboat Line, a subsidiary Erie company, and he occupied an important and confidential position in the administrative organization of the Erie.

From 1899 until a year before his death, Mr. Paine was General Manager of the Panama Railroad Company, and for a time he was also Vice-President and a Director of that Company. This service ended with the purchase of the Panama Railroad by the United States Government and the transfer of its management to the existing Canal Commission.

Mr. Paine was elected a Member of the American Society of Civil Engineers on its reorganization in December, 1867, and was the second man to join the Society: numbering 17 on the list of members as it stood for the first year. He contributed to the *Transactions* Paper XX: "History of the Iron Rails on the Michigan Southern and Northern Indiana Railway," and was President of the Society during 1883.

At the time of his death, Mr. Paine was a Member of the Century Club, in New York, an Honorary Member of the Western Society of Engineers, and of the Engineers' Club of Cleveland, and a number of other scientific and philosophical bodies.

Mr. Paine's personality was so extraordinary, and meant so much to those among whom he lived, that special mention should be made of it. His manner was commanding, but singularly gracious. He had a dignified and impressive presence. He was of generous and enthusiastic temperament. He had a broad sympathy, wide reading, and a discriminating taste in literature and art; but, beyond this, there is much more to be said. In every generation there are a few men who impress their fellow men by beauty and nobility of character, quite apart from those qualities which we may think of as purely intellectual. They have a distinction which wealth or power or achievement cannot bestow. In the deepest recesses of our minds we recognize these men as being the real nobility—the flower of humanity. Mr. Paine belonged to the small group of men distinguished by character. He had intellectual superiority, and he was a man of honorable achievement; but we, who knew him well, think of him first and respect him most for the subtle qualities of gentle manliness. His temper was naturally quick, and he had great personal dignity; but his courtesy was unfailing and his modesty was sincere. He was chivalric in thought and conduct. Honor, truth, and duty were in the roots of his nature—inherited, bred in the bone. These were his shining characteristics, by virtue of which his life was lived in a high and serene atmosphere, and in that atmosphere dwelt with him a wife, his equal in every way.

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INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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Vol. XXXIV. MARCH, 1908. No. 3.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

ERECTION OF THE BELLOWS FALLS ARCH BRIDGE.

By L. D. Rights, Assoc. M. Am. Soc. C. E.

To be Presented April 1st, 1908.

The highway bridge across the Connecticut River at Bellows Falls, Vt., is interesting because, in the United States, it stands alone as an example of a through arch with suspended floor, and also because, as an arch, it is only surpassed in span by the two deck arches at Niagara.

The residents of North Walpole, N. H., depend largely on the factories at Bellows Falls, Vt., for their employment, and on the stores for their trading. To reach the town, they were compelled to use the old wooden toll bridge at the south end, or venture on the Sullivan County (Boston and Maine) Railroad bridge, or patronize a rather uncertain rowboat ferry. For years they had urged a more convenient crossing, and this was naturally backed up by the merchants and business men on the Vermont side. The depth of the river at this point, about 25 ft., strong objections to piers above the mouth of the canal, owing to the vested rights of the Canal Company, and the freeing of the old toll bridge before another could become available, were factors contributing to the delay of the project, which resolved itself, largely, into a matter of cost.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and when finally closed, the papers, with discussion in full, will be published in *Transactions*.

Early in the spring of 1904, the agitation was again revived, and interested citizens brought forward new and old schemes. One that met with considerable favor was to locate a pier on the rock in shallow water just above the angle of the dam, shown on the map, Fig. 1. This permitted the use of two spans, but made it necessary that these spans should be at an angle with each other in order to reach convenient landing places on the shores.

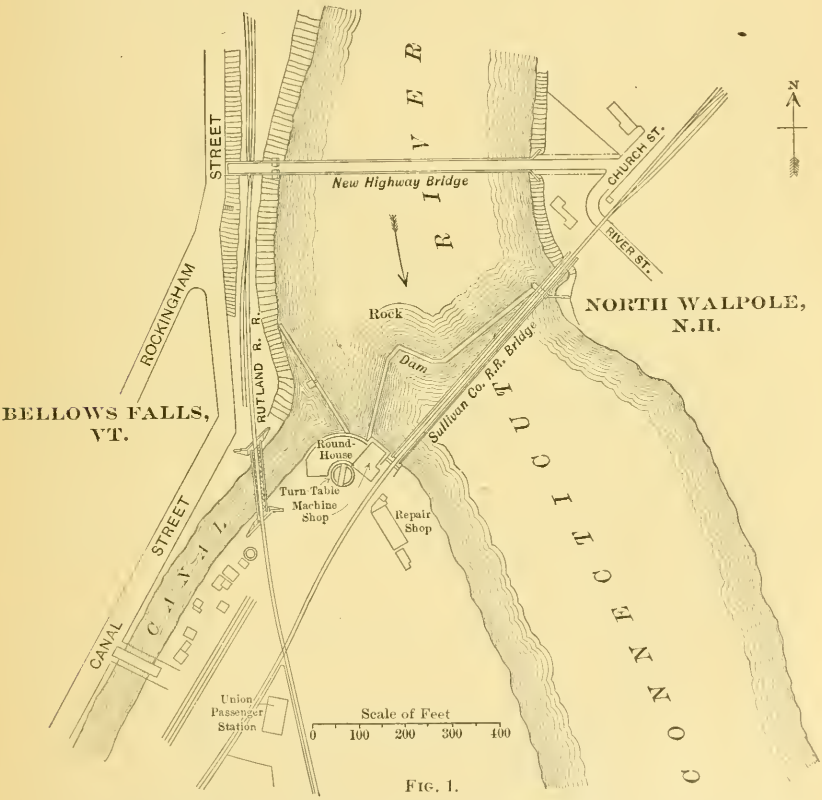


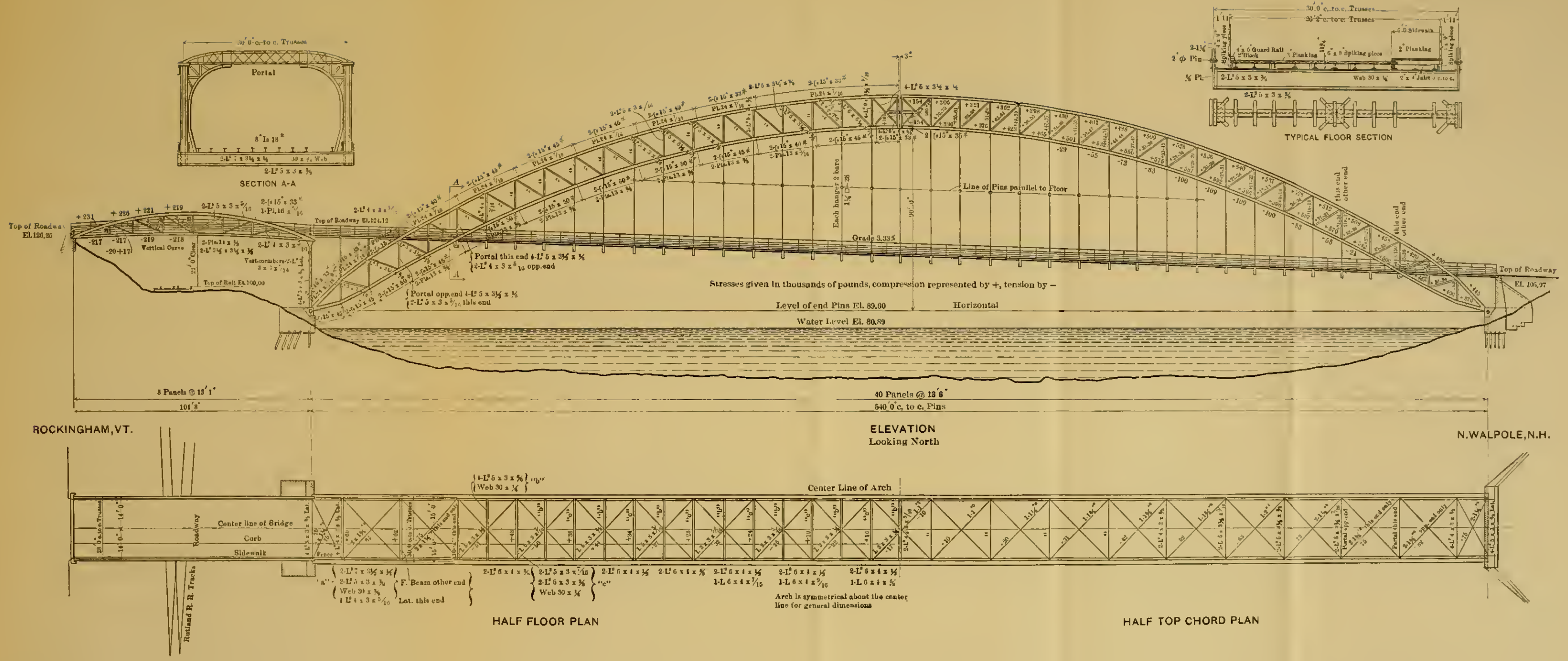
Fig. 1.

In March, the two towns, at their annual meeting, voted appropriations to cover the cost of freeing the old toll bridge and building a new free bridge, and appointed a joint committee to receive bids and enter into a contract for the work. The committee secured some preliminary estimates on the various schemes, but, owing to the probability that the new bridge might be used in the future for electric cars, it did not favor the plan for two spans at an angle, but preferred a single span.

As the preliminary estimates for a single-span bridge were unsatisfactory, some members of the committee visited Boston and appealed to the President of the Boston and Maine Railroad, who was interested to the extent of freeing the railroad bridge from unauthorized foot passengers, and who responded by offering the services of J. P. Snow, M. Am. Soc. C. E., in an advisory capacity. Mr. Snow was naturally familiar with the general surroundings, but did not feel that conditions warranted the preparation of an elaborate design; therefore, he drew up general specifications, and called for bids, requesting each bidder to submit his own plans. Among the designs submitted were several for truss and suspension bridges, but all the prices were greater than the appropriation, and the bids were rejected. Mr. Snow was satisfied that a bridge could be built within the specified sum, and recommended the employment of J. R. Worcester, M. Am. Soc. C. E., which suggestion the committee accepted. Mr. Worcester concluded to adopt an entirely different type of bridge, and decided that a three-hinged, riveted arch with suspended floor would be the most artistic and suitable structure for the location. He drew plans and specifications, and, on his recommendation, separate bids were asked for the masonry and structural steel. The results of the competition were satisfactory, as several bids were received which were within the appropriation. On the recommendation of Mr. Snow, the contract for the masonry was awarded to Joseph Ross and Sons, of Boston, and the superstructure to Lewis F. Shoemaker and Company, of Philadelphia and New York.

Design.—As will be seen by the general plan and stress sheet, Plate XXVIII, the bridge is about 650 ft. long, and consists of a single, three-hinged, arch span, 540 ft. from center to center of end pins, with a short truss span at the west end, 104 ft. 8 in. from center to center of bearings. This short span was necessary, in order to carry the street over the Rutland Railroad. It will be noted that the roadway is on a grade of 3.33%, running downward from the short span to the abutment at the east end. The height of the main arch is 90 ft. between the hinge centers. The truss chords follow the lines of two parabolas 14 ft. apart. In order to secure simplicity of detail, the trusses do not diverge at the bottom, but stand in parallel vertical planes, 30 ft. from center to center. This provides for a roadway, 20 ft. clear, and one sidewalk, 6 ft. wide, as shown by the cross-section.

PLATE XXVIII.
PAPERS, AM. SOC. C. E.
MARCH, 1908.
RIGHTS ON
ERECTION OF ARCH BRIDGE.



At the east end of the arch an abutment has been provided, and at the west end two piers take the thrust of the main arches, and support the vertical posts which carry the end of a small truss bridge.

The floor is designed for either a live load of 100 lb. per sq. ft. or a 12-ton wagon load on two axles, 10 ft. from center to center. In addition, provision is made in the hangers and floor beams for a single-track line of 18-ton electric cars, to run on the opposite side of the bridge from the walk. The connections for track stringers have been provided in the floor beams, and, if it should be thought advisable in the future to carry the trolley line over to the New Hampshire side, it can be done at a very small expense.

The main trusses are designed for a live load of 60 lb. and for a wind load of 40 lb. per sq. ft. The floor is suspended from the arches by two hanger bars, 1¼ in. square at each panel point. These are connected, with 2-in. pins, to the truss at the top and the floor beams at the bottom. The hangers can only take the vertical load, and therefore wind chords have been provided in the planes of the trusses at the level of the roadway. These chords, with the floor laterals, form a horizontal truss which carries the wind stresses to the abutments. To prevent complication in the arch stresses, this lateral system is not attached to the arches rigidly, but has expansion joints at each end transmitting shear only.

The unit stresses adopted were:

For tension:

Steel for floor	15 000
Steel for trusses	16 000
Laterals	17 000
Wrought-iron loop rods	9 000

For compression:

Main arch chords	15 000
Struts	$\dfrac{13\,500}{1 + \dfrac{l^2}{36\,000\, r^2}}$

For pins and rivets:

Shear	10 000
Bearing	18 000
Bending on extreme fiber of pins	22 000

Allowable bearing stress for concrete, 400 lb. per sq. in.

The steel was furnished in accordance with the specifications of the American Railway Engineering and Maintenance of Way Association, with an ultimate tensile strength of 60 000 lb. per sq. in. The flooring timber is long-leaf Georgia yellow pine of prime quality.

One of the unique features of the design is the arrangement which Mr. Worcester has adopted for the middle of the arch. It will be noticed that, instead of converging the chords at the center panels, in order to take the stress of the center hinge, he has taken the horizontal thrust on a short strut, composed of two 15-in. channels, and has then divided this thrust into four parts at the intersection of the diagonals in the panels next to the center. In this way he has secured the architectural features of a two-hinged arch with continuous chords, adding considerably to the appearance of the bridge. It will also be noticed that he has dispensed with the pin at the center, the struts merely bearing against one another with faced ends.

In order to avoid the optical illusion of a sag in the suspended floor, this was cambered 18 in. at the center of the arch. It was felt that this camber would take care of any slight deflection due to live load, and the design and details of the arch were developed on theoretical lines, with no provision in the arch for either live- or dead-load camber.

Erection Design.—In securing data for the preliminary design for the estimate of the cost of erection, the idea of building towers some distance apart and erecting the trusses by the cantilever method, suggested itself to the writer, the main question being to determine the economic spacing of these towers. As it was not considered advisable to do any riveting until the arch was swung, it was necessary to plan to bolt all connections temporarily. The strength of these bolted connections, therefore, was a factor in the length of the cantilever. Another consideration was the distance that material could be hauled satisfactorily with a standard 60-ft. steel boom. The writer made a design, and decided that a span of six panels, or about 80 ft., would give the best results. The sizes and details were then developed in the drawing room, under the direction of the chief engineer.

On account of the narrow width of the structure in comparison with its height, the center falsework towers were battered in the plane at right angles to the bridge, thus enabling them to withstand better the shock of winter storms or floating ice. The general elevation of the falsework towers is shown on Plate XXX, which indicates the

FIG. 1.—BELLOWS FALLS ARCH BRIDGE IN PROGRESS.

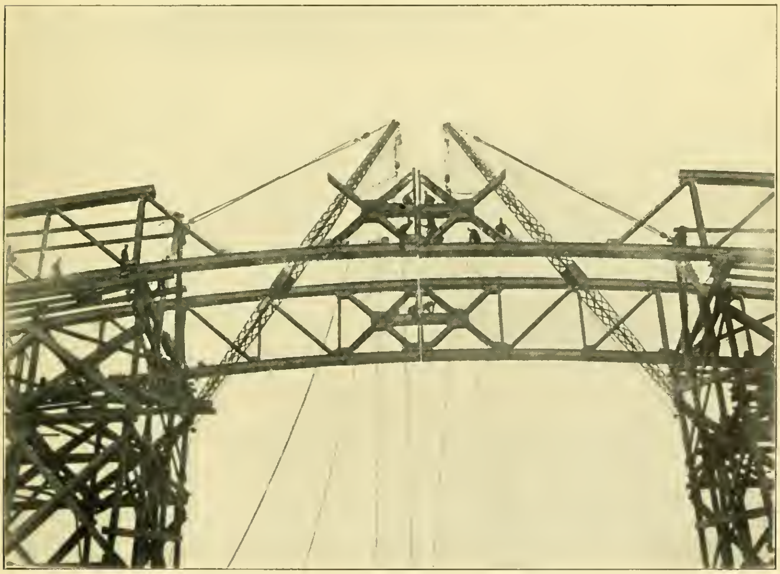


FIG. 2.—CONNECTING ARCH AT THE CENTER.
(CAMERA POINTED UPWARD AT AN ANGLE OF 15 DEGREES.)

number and spacing of the pile supports. The details of the center Bents, F, F_1, and F_2, are shown by Fig. 2, which gives the size of the main posts and bracing, and indicates the splices and number of bolts.

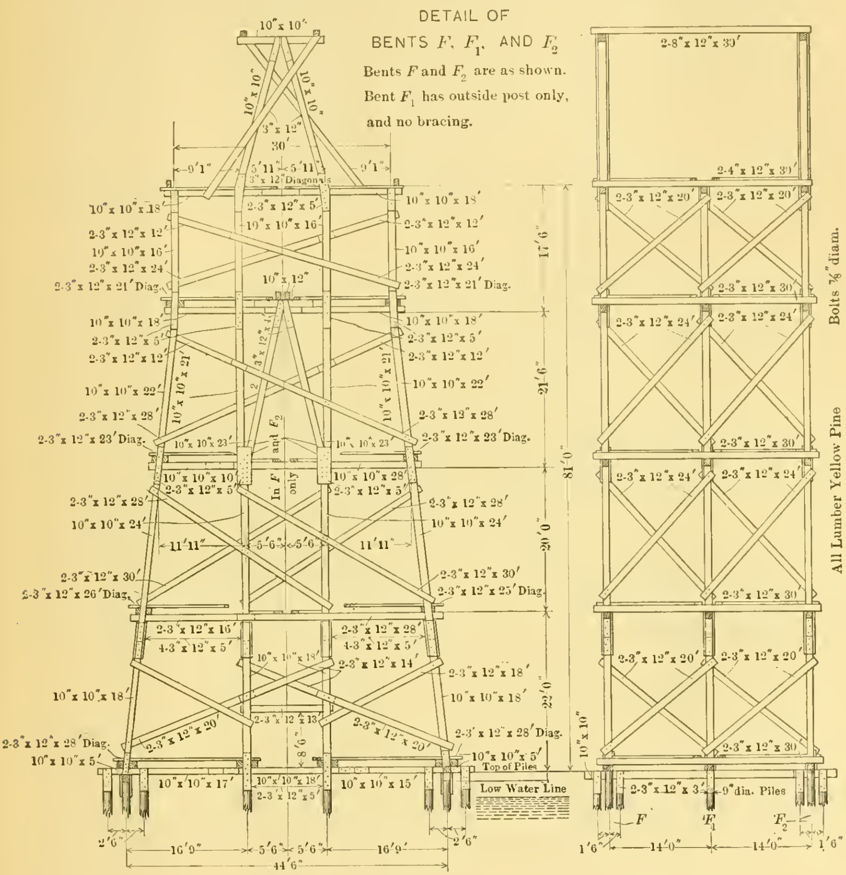


FIG. 2.

It was considered advisable, on account of the switching facilities, to unload the material on the Vermont side of the bridge and carry those pieces required for the east side on a standard-gauge service track running between the towers, along the center line of the bridge. A 10-ton stiff-leg derrick was placed on the west shore, to unload material from the cars and transfer it to the trucks on the service track. Two 30-h.p. hoisting engines, with two drums and four spools each, were located at Bent E to raise the steel.

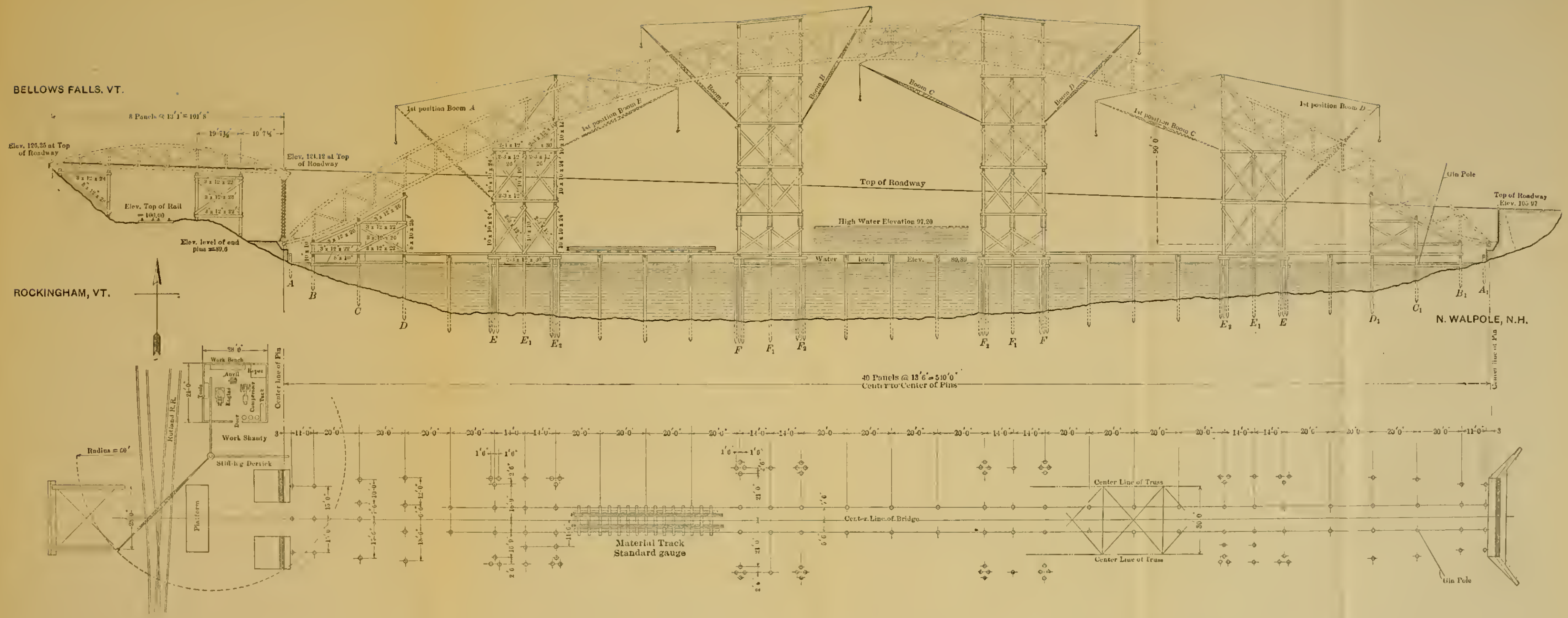
Erection.—The masonry plans provided for piles to be driven in the foundations for the arch, and, as the masonry contractor was well equipped to do the work, the contract for furnishing and driving the piles for the falsework was sublet to him. The water has an average depth of about 25 ft., and the bottom is hard gravel. Spruce piles were specified, in order that they might be sold to the pulp mills after the work was completed. They were driven from 8 to 10 ft. into the bottom, and were cut off and capped about 3 ft. above the low-water line. The falsework towers were completed in November, and the shoes were set on December 6th, 1904.

Two gangs were started, one from each end of the arch, and the rivalry between them helped not a little in the rapid erection of the work. The severe weather of the winter of 1904-05 will no doubt be remembered, but, even in that latitude, there were breaks in the cold, and on two separate days considerable rain fell. The fear that the ice might go out, which would mean taking out the falsework and everything with it, was a constant incentive to hasten the erection in every way possible. It had not been the intention to do any work on the ice, but after it had frozen to the thickness of about 2 ft., it was found to be very convenient when assembling the chords, which were handled in two sections of four panels each. The ice also acted as a hindrance, for when the canal gates were closed, on Sundays and holidays, the river rose about 18 in., and it was necessary to keep the ice chopped free from the piles.

When the ice first began to form about the falsework, the structure showed a tendency to move down stream. This was carefully noted, account of it being taken in placing the steel. When the arch was swung, some of the bents were found to have moved down stream about 4 in. As the load was put on the falsework, some of the bents sank slightly, but this settlement was adjusted with wedges under the blocking at each point of support.

The two end panels of the lower chord and the end panel of the upper chord were shipped riveted together. These members, each weighing about 8 tons, constituted the heaviest pieces to be handled.

In beginning the erection, the shoes and end panels of the west end were set with the stiff-leg derrick used for handling material from the siding to the material trucks. On the east end a gin pole was placed to set the shoes and end panels. Provision had been made in the plans


BELLOWS FALLS, VT.
ROCKINGHAM, VT.
N. WALPOLE, N.H.
8 Panels @ 13'1" = 104'8"
Elev. 126.25 at Top of Roadway
Elev. 121.12 at Top of Roadway
Elev. Top of Rail = 100.00
Elev. level of end pins = 89.0
1st position Boom A
1st position Boom B
1st position Boom C
1st position Boom D
Boom A
Boom B
Boom C
Boom D
Top of Roadway
High Water Elevation 97.30
Water level Elev. 80.80
Gin Pole
Top of Roadway Elev. 105.97
40 Panels @ 13'6" = 540'0"
Center to Center of Pins
Center line of Pin
Work Bench
Anvil
Ropes
Tools
Engine
Compressor
Tank
Door
Work Shanty
Rutland R.R.
Radius = 60'
Stiff-leg Derrick
Platform
Material Track
Standard gauge
Center Line of Truss
Center Line of Bridge
Center Line of Truss
Gin Pole

for a clearance of 4 in. behind the steel shoes. As soon as the shoes and the first panels had been set and lined up, this space was filled with concrete.

Two standard steel booms, *A* and *D,* Plate XXX, of 10 tons capacity, were erected at Bents *E,* and with these the steelwork was erected so as to rest on top of the falsework at these bents. Booms *B* and *C* were then erected at Bents E_2, and, with the assistance of the first two booms, the structure was laid across the *E* towers. Booms *A* and *D* were then taken down and re-erected at Bents *F,* thus giving two booms to handle the material over the 80-ft. space between Bents E_2 and *F.* The arrangement of the booms in this last position is shown clearly on the progress photograph, Fig. 1, Plate XXIX. The lower chords for two sections of four panels were assembled on the ice, with the first diagonal at Bent E_2 attached to them. Then, while Booms *A* and *D* held up the material, Booms *B* and *C* were used to fill in the web members and put on the top chord. The last section of bottom chord from the cantilevered end of Bent *F* was placed by Booms *A* and *D,* which filled in the web members and the top chord. While this was being done, Booms *B* and *C* were taken down from their first position and re-erected at Bents F_2. The steelwork was carried across the *F* towers, and the erection of the 80-ft. space between them was taken care of by these two booms. The photograph, Fig. 2, Plate XXIX, was taken with the camera pointed almost directly up at the work, and shows the connection of the lower chords of the arch members at the center.

To overcome the natural tendency of the chords to lengthen during erection, and to adjust any slight errors in the measurement between the end-pin centers, the details at the center were arranged to allow a slight leeway at the meeting points. The chords and the vertical posts at the center were set back 2 in. on each half from the center line, making a total clearance between them of 4 in. The center strut projected a little beyond the vertical posts, but was set back 1½ in. from the center line on each half, making a total clearance of 3 in. between the bearing points. With the intention of being able to take up the theoretical opening of 3 in. and for any adjustment that it might be necessary to make, 6 in. of fillers were provided. When the trusses reached the *F* tower, it was found that the erector had failed to make sufficient provision for the settling of the falsework at Bents *D* and *E.*

The result was that the arch was low, and measurements showed that the steelwork would overlap at the center. The trusses were raised to their proper height by wedges at the tops of all the towers. When the steelwork was joined at the center, it was found that it had slightly over-run the calculated distance. Instead of 3 in. of fillers, theoretically needed, a thickness of only 1½ in. was required.

As soon as the work was connected up it was thoroughly bolted. The web members were fastened with bolts in 75% of the holes, and the chords with bolts in 50% of the holes. When this was done, the wedges were knocked out and the arch ribs were swung off. As the only load the arch ribs had to support was that of the steel in the trusses, the deflection at the time of swinging off was comparatively small. The erection superintendent reported that the settlement at the crown was about 1½ in.

The trusses were connected and swung off on January 10th, 1905, making a total of 35 days from the time of setting the shoes until the arches were swung off. From this time should be deducted one holiday, three Sundays, and three days of cold or wet weather, which gave an actual working period of 28 days. During a good portion of this time the thermometer was about at zero, which made it difficult for the men to work actively.

The falsework was taken down the day after the arches were swung off, and the erection gang started from both sides to put in the hanger rods and place the steelwork of the floor system. The wood floor was not put on at this time, so that no local deformation of the arch ribs was noticed. While the steel floor system was being placed, other gangs adjusted the bracing and lined up the arches. It had been the intention to do the field riveting with pneumatic riveters, but a day before the riveting was to begin, the tool-house was destroyed by fire, and the compressor was injured so that it could not be used for this work. Eight gangs, of four men each, drove the field rivets by hand. As soon as the weather would allow, the field painting was completed and the wood floor laid. The finished bridge is shown by Figs. 1 and 2, Plate XXXI.

Thirty-six men were employed when the erection of the steelwork began, and the number was increased afterward to forty-five. The weight of the structural steel was 450 tons for the main arch span, and 75 000 ft., B. M., of lumber were used in the floor. The contract price for the masonry was $6 000, and for the superstructure $41 000.

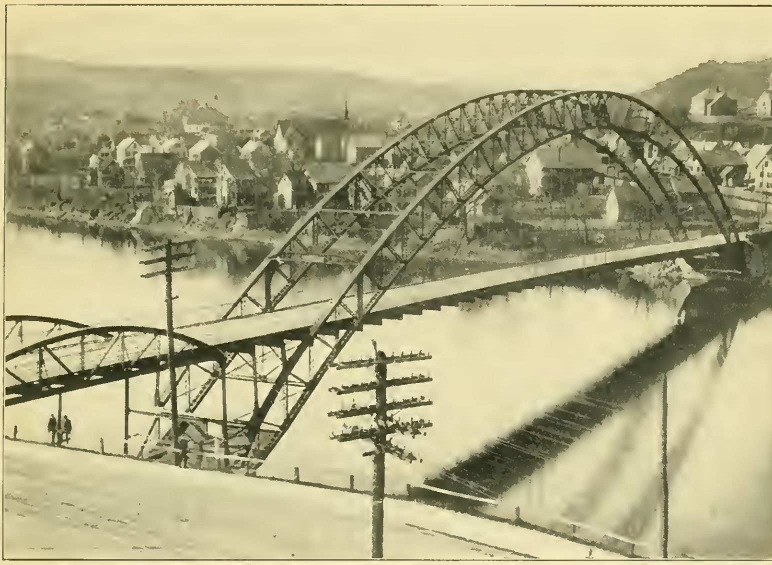


FIG. 1.—THE BELLOWS FALLS ARCH BRIDGE.

FIG. 2.—END VIEW, BELLOWS FALLS ARCH BRIDGE.

Credit for the work should be given to Mr. J. H. Fichthorn, Chief Engineer for Lewis F. Shoemaker and Company, and to Mr. A. L. Westbrook, Field Superintendent. The writer also wishes to express his thanks to Messrs. J. P. Snow and J. R. Worcester for the information furnished for the preparation of this paper.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

RECENT DEVELOPMENTS IN PNEUMATIC FOUNDATIONS FOR BUILDINGS.

By D. A. Usina, Assoc. Am. Soc. C. E.
To be Presented April 15th, 1908.

The purpose of this paper is to review briefly the recent and very interesting development in foundations of the class generally used for the high buildings being erected in the lower section of New York City. The earth there overlies a stratum of rock, the depth of which varies from 40 to 100 ft., and the enormous loads are carried most securely by concrete piers built with pneumatic caissons, and resting directly on the substratum of rock.

Prior State of the Art.—Prior to the present improvements, the conventional type of construction was as illustrated in Figs. 1 and 2. The working chamber was built with sides and roof of heavy timber or of sheet steel with stiffeners at suitable intervals. The coffer-dam was built up in successive sections (also of timber or stiffened steel), the horizontal joints being made by angles on the inside, and the walls being braced by transverse struts, where the shape and size demanded it. The shaft was of steel tubing fastened to the roof and at the several horizontal joints by outside angles.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

As the structure was sunk, to bring the upper edge of each section of the coffer-dam near the ground level, a new section of coffer-dam and a new section of shafting were added, and the space between the coffer-dam and the shafting was filled with concrete. When bed-rock was reached, the working chamber and the shaft were also filled with concrete. The finished pier consisted of two entirely separate bodies of concrete—an inverted T-shaped portion bounded by the shafting and the roof and walls of the working chamber, and a ring-shaped portion surrounding the shaft and enclosed within the coffer-dam.

The surrounding shell, consisting of the coffer-dam and the sides of the working chamber, whether of timber or of steel, could only be considered a mould for the concrete and a curb or lining for holding back the earth during the sinking of the caisson. It could not be calculated as supporting any weight, but, on the contrary, was certain to rot or corrode in time, and leave a more or less free space around the pier. The shafting, and especially the roof, where the latter was of metal and was left in place, presented very serious possibilities. Their protection from corrosion depended on the care with which the concrete was rammed into contact with them. If either corroded to a substantial extent, it would produce a very large surface of weakness. The permanence of these important elements of the structure, therefore, depended on the care of workmen, who are not to be relied on for more care than is necessary at the moment. Furthermore, the angles at the several horizontal joints formed grooves in the concrete from 3 to 6 in. deep. Only under unusually favorable conditions could the shafting angles be calculated to act as supporting a share of the load in the ratio of their horizontal area to that of the complete cross-section of the pier; but the angles at the joints of the coffer-dam would not transmit any substantial pressure to the concrete below them, because the concrete would never be rammed under them sufficiently. The only transmission of pressure would be to the decaying or corroding walls, and the angles themselves would corrode in time. The greatest area upon which the bearing strain could be calculated correctly, therefore, was that within the inner edge of the angle-irons (*X*, Fig. 2), rather than that within the inner face of the coffer-dam (*Y*, Fig. 2). As a matter of fact, the latter standard was generally used, but the error was swallowed in the large factor of safety made necessary by the uncertainties of the problem.

Furthermore, the useless, and to some extent harmful, materials left in the ground, were very expensive parts of the structure.

There were thus two powerful incentives for the elimination of these materials from the finished structure, either by sinking the pier without them, or by withdrawing them after use. Nevertheless, there was a period of many years during which little or nothing was accomplished.

The recent activity in high building construction in New York City, however, making necessary a very extensive use of caissons of this type, has witnessed the substantial elimination of every material but concrete. The sinking of the coffer-dam and of a metal or timber roof for the working chamber, has been rendered unnecessary, and the steel shafting has been designed to permit its ready removal after it has served its purpose in the sinking of the caisson. These improvements have been put into practice in the foundations of the building for the United States Express Company, at the corner of Rector Street and Trinity Place; the New Trinity Building; the building for the United States Realty Company, at Broadway and Thames Street, and the Singer Building, on Broadway near Liberty Street.

Elimination of the Roof.—The most serious objection to caissons of the style described has been the existence of the roof, constituting a dividing plane across almost the entire cross-section. The objection to such a dividing plane was appreciated from the earliest use of pneumatic caissons. The late Theophilus E. Sickles, M. Am. Soc. C. E., in 1870, and John F. O'Rourke, M. Am. Soc. C. E., in 1898, proposed the removal of the roof after the sinking of the caisson and before the introduction of the concrete above the working chamber.

The Sickles caisson is shown in Figs. 3 and 4. The roof consisted of four segmental plates bolted to the under side of internal flanges of the casing and attached to each other by bolts passing through radial flanges on the under side. After sinking to the required depth, and sealing the cutting edge with a sufficient filling of concrete to prevent the entrance of water, the air was cut off and the roof removed by withdrawing the bolts passing through the several flanges. The caisson of the type shown had a high roof and no separate air-shaft supported upon the roof, as in the modern type, the coffer-dam or outer shell being made air-tight throughout its height. For a caisson of this type, the design of the roof was probably entirely satisfactory.

The O'Rourke caisson, Figs. 5 and 6, utilized a similar roof in half-round sections, but the roof was bolted on the top of the inward flange of the casing, and the flanges connecting the segments to each other were at the top. This would permit the filling of the working chamber with concrete clear up to the roof before removing the latter.

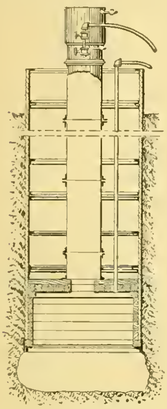
FIG. 1

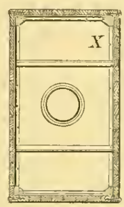

FIG. 2

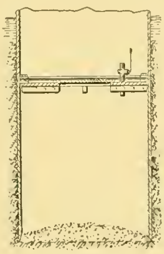
FIG. 3

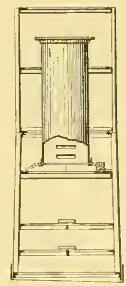
FIG. 5

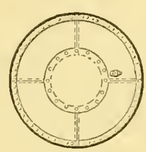
FIG. 4

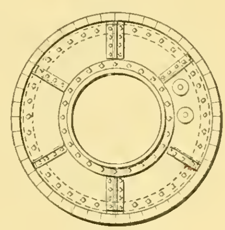
FIG. 6

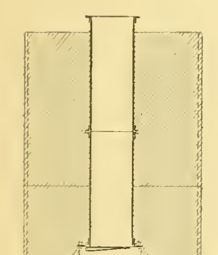
FIG. 7

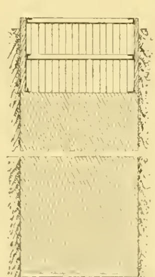
FIG. 8

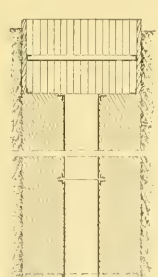
FIG. 9

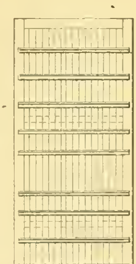
FIG. 10

The chief defect of these methods, however, appears in cases where, in order to get the requisite weight, the concrete is filled into the space above the roof during the sinking operation, as is usual in sinking through earth for building foundations. In such operations it has been impossible to eliminate the roof of the working chamber until the introduction of a recent improvement which, at the same stroke, eliminated the coffer-dam which had previously passed for a necessary evil in sinking caissons in earth. The feasibility of the improvement was first demonstrated by sinking all the caissons for the building for the United States Express Company by this method, and at a substantial reduction in cost.

Elimination of the Coffer-dam.—There had been previously suggested, in 1904, the elimination of the coffer-dam and roof by sinking practically a solid pier of concrete, with only a central air-shaft and a working chamber hollowed out of the bottom. Fig. 7 gives a sufficient idea of the construction proposed. There was no distinction between different parts of the structure, except in so far as the lower portion of the concrete might be considered as the roof and side walls of the working chamber, and the concrete above this might be considered as the coffer-dam extending solidly from the surrounding earth to the shaft. It was proposed to build the whole of annular blocks of concrete laid one above another, or to form substantially a monolith by building up the structure *in situ* as fast as it was sunk. The difficulties in the way of moulding the concrete working chamber with suitably strong roof and sides and hardening it sufficiently in the short time available at the works then in hand prevented the utilization of this design, and, instead, the contractors adopted the design shown in Figs. 8, 9, and 10.

The working chamber was built of heavy timber, and across the top were laid angle-irons, a few inches below which was fastened a temporary flooring. The steel shafting was supported on this flooring, and a roof of concrete was moulded thereon to a substantial height, and of the same outside dimensions as the working chamber. The earth being excavated, and the chamber sunk to a sufficient depth, another section of concrete was added. The shafting was built up from time to time to maintain it above the concrete. After the first section of concrete was finished, the successive sections were moulded in place without interruption of the sinking operations; the excavation and the building up proceeding of course at the same ultimate rate, but quite independently of each other, and the coffer-dam, reduced to merely a mould for the concrete, being removed before the sinking of each concrete section.

In a previous design, it had been proposed to divide each section of the coffer-dam into flat units which might be readily transported and only united to each other when in place on the next lower section, this method having the further advantage of avoiding the necessity of breaking the air-pipes (see Fig. 1), which had been a cause of delay with the use of sections which were completed before being put in place; and such flat units were now found to be excellent moulding plates, only four being needed for each section of concrete, and excessive lengths being unobjectionable, because one might overlap the next at the corner.

The temporary flooring carried the concrete roof until the latter was hardened, and was removed before putting on the air pressure and the necessary lock. The angle cross-bars remained embedded in the concrete, transmitting its weight to the timber walls, although they were not necessary for the purpose after the concrete had hardened; and, in fact, after reaching a comparatively slight depth, the weight of the concrete was sustained by the skin friction and the air pressure, and added weights were necessary to force the caisson down. The cross-bars might have been designed and connected so as to permit their removal after the hardening of the concrete, if such removal had been thought of importance.

Only one accident occurred, and this demonstrated the advisability of using timber rather than concrete for the walls of the working chamber. The earth under one wall of the working chamber had been excavated previously to remove the footing of an old wall. When the first section of concrete had been moulded on this working chamber and the mould had been removed. preparatory to sinking the concrete section, the old material replaced in the excavation allowed one side to settle so as to tilt the structure, and, before it could be righted, it fell over. The concrete was tied to the working chamber only by the crossing angles embedded in the base of the concrete, and swung bodily about the upper edge of a side wall of the working chamber, thus for a time putting its entire weight on this single wall. But the chamber was built so strongly that it was substantially uninjured, and the workmen in it at the time were unscathed. The accident, while indicating the necessity for greater precaution in building and sinking the first concrete section, demonstrated the practical excellence of the design.

When such a caisson was sunk to its final depth, there was no metal or timber roof to be removed. The cost of making first a sectional bolted roof, like that of Sickles or O'Rourke, and subsequently removing it, was saved; and, which is probably more important, the introduction of concrete above the working chamber did not have to await the sinking of the caisson. Its weight could be utilized in the sinking of the structure, and this weight, in caissons passing for a great depth through earth, is a very substantial consideration. It constituted probably the greatest of the series of advance steps under discussion.

Elimination of Shaft Lining.—The finished pier included, besides the concrete body, the cross-bars, which are a negligible consideration, being entirely embedded so as to prevent corrosion, and being of such slight cross-section as not to form cleavage planes in the concrete; and the steel shaft lining, which, at the very best, added not a pound to the load for which the pier might be safely designed, and, at the worst, might prove an element of weakness, and was certainly an element of substantial expense.

The progress of improvement in eliminating the shaft lining was the reverse of that in eliminating the roof. In the latter case, the idea was first advanced of making the roof removable after the caisson had been sunk; and the successful solution of the problem lay in avoiding the building of a true roof. In the case of the shaft lining, the first proposals endeavored to avoid its use entirely, but practical success came only with the idea of sinking the caisson with a shaft lining similar to those previously used, and removing the lining after sinking and before introducing the filling of concrete.

The first idea is shown in Fig. 11. A shaft lining of moulded concrete is shown. To avoid excessive loss by leakage of air through the concrete, it was proposed to coat the inner surface of the shaft lining with air-tight material, such as a paint containing lime. The difficulty of connecting the shaft lining to the air-lock with sufficient strength to resist the upward air pressure on the latter was to be obviated by long tie-rods extending from the lock to the lowest section of the shaft lining, as indicated in dotted lines. It was also proposed in this design to eliminate the lining entirely, merely coring the concrete body and coating the surface with paint, as above, the manner of fastening the air-lock not being specified.

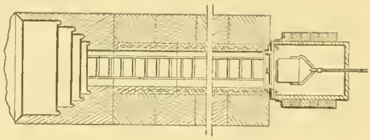

Fig. 11

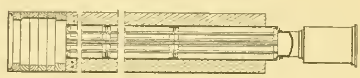

Fig. 12

Fig. 13

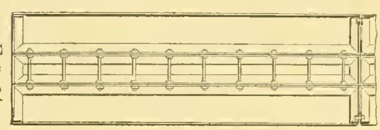

Fig. 14

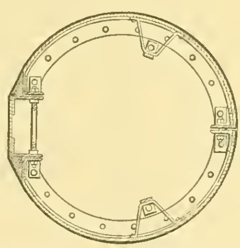

Fig. 15

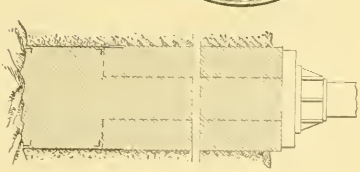

Fig. 16

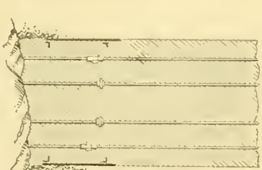

Fig. 17

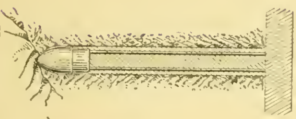

Fig. 18

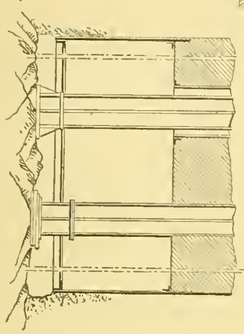

Fig. 19

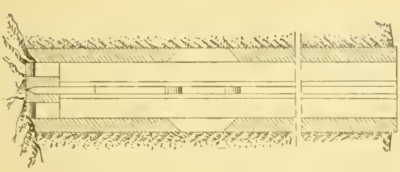

Fig. 20

Fig. 21

The first successful attempt to eliminate the shaft lining, however, involved the use of a removable lining, which, while costing more than those of common design, is usable again and again indefinitely, and, in the long run, effects a great economy. The design used in sinking the caissons of the new Trinity addition, and the adjoining building of the United States Realty Company, is shown in Figs. 12, 13, 14, and 15. It was found that a comparatively small number of sections served for the sinking of many piers. There was no material loss of time involved in removing the sections and reassembling them for further use. In fact, the job was completed in much less than the previous record time for such work.

Figs. 12 and 13 show the shaft lining in place; Figs. 14 and 15 show the construction of one of the collapsible sections. Each section was composed of two approximately semicircular plates internally flanged for bolting to each other along one vertical edge, and a key interposed between the opposite edges of the plates. Internal flanges at the ends served for bolting successive sections to each other. Ladder rungs were arranged conveniently between the flanges of the key, and vertical guides were arranged just inside the line of the end flanges to guide the bucket past them. In some cases the tubing was made oblong in cross-section instead of circular. Packing was provided in all the joints, and this was the only part of the structure requiring renewal, it being cheaper to provide new packing for each re-use than to try to save the old.

Fig. 16 shows the finished pier, supposing the working chamber to be built of sheet steel. The dotted line indicates the joint between the concrete set up in sinking the pier and the filling introduced afterward.

Comparison with Concrete Piles.—Side by side with the progress in caisson work, recent years have seen a rapid improvement in the sinking or building of concrete piles in the earth. The first attempts to substitute concrete for timber or steel in piles contemplated the manufacture of the concrete piles above ground and the sinking of them by one or another of the methods used for timber or steel piles. But, at present, there are in the market several styles of concrete piles made by first forming the excavation and subsequently filling in the concrete. These methods permit the formation of piles of great depth and of theoretically unlimited diameter. Starting from widely-separated points, the two arts, caisson work and pile work, have constantly converged toward the same goal, a simple concrete column, bearing upon a rock or similar sub-foundation in the case of caissons and some piles, and supported by skin friction in the case of other piles.

The analogy has been carried even further by more recent improvements in which vertical reinforcing rods of steel, similar to those sometimes used in concrete piles, are embedded in the concrete of the pier. The base of such a pier is shown in vertical section in Fig. 17, and Fig. 18 shows a concrete pile similarly reinforced. The reinforcing rods in the pier should extend down to the rock sub-foundation, and are most easily introduced in that method of construction in which the roof of the working chamber is omitted, turn-buckles being introduced for putting the rods under stress before embedding them in concrete. The non-adjustable flange joints may be used for the rods which run through the shaft, and substantially the entire length of which may bear freely on the sub-foundation before the concrete is filled in about them.

Most Recent Modifications.—The steel rods in the foregoing designs merely reinforce the concrete. Should the concrete fail, or be designed or built so as to shift a substantial portion of the load to the rods, the latter would be unable to stand the strain. A recent design includes the introduction of columns of sufficient strength to carry a substantial load. In fact, they may be proportioned to carry all or the greater part of the load. Fig. 19 shows the caisson sunk to rock, and the columns in place, ready to be filled with concrete. The columns are of ordinary style, built up of Z-bars riveted to a central plate. One column is embedded in the concrete from the beginning, and is wedged up at its lower end. This column may be duplicated as often as desired. Another passes down through the shaft, and is properly supported before its embedment. The shaft lining may or may not be withdrawn, as desired.

Since it is possible to carry concrete piles in many cases to a rock sub-foundation, where they act as true columns, the idea has been conceived of substituting steel, with its immensely greater strength as a column, and surrounding it with concrete, which stiffens the column to some extent, but which performs the principal function of protecting the steel from corrosion. The finished pile or column is indicated in Figs. 20 and 21. The column is hollow, which serves to carry a water-jet for sinking the column itself, and has a surrounding shell, which is afterward filled with concrete around and within the center of the column. The shell may be withdrawn as the concrete is introduced. The column may be shod at its lower end so as to secure a good bearing by ramming it down on the rock.

Invention is largely accidental, and its progress is apt to be most erratic. The writer has never observed a series of improvements progressing more logically and consistently in the same direction than those here considered. The engineering profession owes to Daniel E. Moran, M. Am. Soc. C. E., and John W. Doty, Assoc. M. Am. Soc. C. E., who conceived these improvements, and to the Foundation Company, by whom they were put into practice, a very large debt for the originality and progressive spirit with which they have met the demands of modern builders for economical methods of providing foundations of maximum bearing strength.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

SUBSTRUCTURE OF PISCATAQUIS BRIDGE, AND ANALYSIS OF CONCRETE WORK.

By G. A. Hersey, Jun. Am. Soc. C. E.

To be Presented May 6th, 1908.

This paper gives a general description of the construction of the Piscataquis River Bridge, built for the Bangor and Aroostook Railroad, and also the results attained with the different classes of concrete used.

The Piscataquis Bridge was built during 1907, and is a part of the "Medford Cut-off," an extension of the Northern Maine Seaport Railroad, a branch of the Bangor and Aroostook Railroad. This extension begins at the present terminus of the Northern Maine Seaport Railroad, and runs northward about 28 miles until it again strikes the main line of the Bangor and Aroostook Railroad. It shortens the distance between the two points on the main line 4.3 miles, reduces the curvature considerably, and gives much easier grades.

The "Cut-off" crosses the Piscataquis River in the Town of Medford, and on the line of a very high horse-back—a formation peculiar to that section of the country—which was of considerable value in the construction of the railroad. The line follows the horse-back in a general direction for about 14 miles, and for 6 miles skirts along its side; it can even be said that the entire road was made from it, for, as the

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northerly half of the line passes through low land, material from the horse-back was used for filling, as well as ballasting. The material in it varies from sand to coarse gravel, and, in a few instances, clay. At the river the best kind of gravel was found, and the hills on either side afforded excellent sand and stone for concrete.

The grade of the railroad is 55.5 ft. above the average water level, with about 8 ft. of water in the river.

A bridge of the deck type was adopted, with four river piers and two shore abutments of reinforced concrete. The line crosses at a bend in the river, the piers being placed at an angle of 55 degrees. The total length of the bridge is 607 ft. 10 in. About 13 tons of steel were used for reinforcement, mostly in the two abutments, there being but little placed in the tops of each of the piers.

The work was handled with a Lidgerwood cableway, 800 ft. long, placed on the center line of the bridge. This cableway was used in making all the excavation, in conveying and placing concrete, moving machinery, and, later, in erecting the temporary trestle bridge. The cableway clearly demonstrated its suitability in this case, and, for rapid and profitable work, it would be hard to find anything better. In landing the north abutment, it was necessary to go about 50 ft. into the side of a 40-ft. bank and remove about 3 000 cu. yd. With the cableway, all this material was saved and used directly for concrete and for banking coffer-dams, whereas, by almost any other method, it would have been necessary to rehandle it several times.

The concrete was all machine-mixed, and dumped into buckets which were run out under the cableway and carried to any part of the bridge. The greatest number of buckets used in one day was 182, for 9 hours' work. Each bucket held 1 cu. yd.

Crib coffer-dams, of 8 by 8-in. timber, in 8-ft. sections, were made on the river bank. Alternate sections were floored about four tiers from the bottom. These cribs were then set in place, and the floored sections were loaded with rock. The outside was covered with 2-in. planks driven into the river bottom as far as possible by hand-mauls. The cribs were then banked with earth to above the water level. These coffer-dams gave excellent satisfaction, and only in one instance was there any trouble from leakage, and that was quickly remedied by a generous use of straw and gravel. The pumping was done by five centrifugal pumps having a combined discharge of 24 in., and they were able at all times to take care of the water.

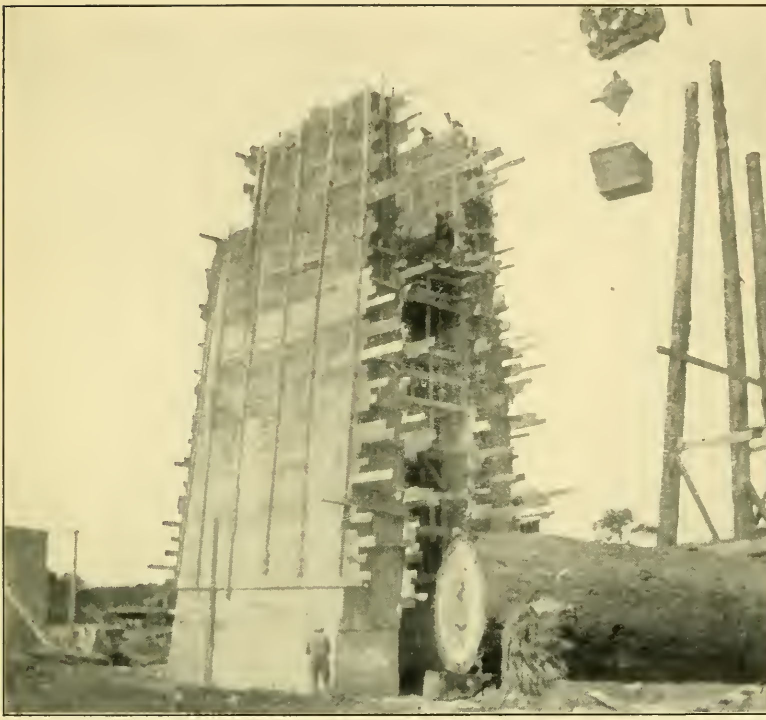

Fig. 1.—South Abutment, Showing Method of Placing Concrete in Finishing Abutment. Total Height, 57 Ft. (Above Ground, 45 Ft.)

Fig. 2.—Building Coffer-Dam of Pier 3, Showing Cableway.

The river bottom is very rocky and gravelly, and almost as hard and compact as if cemented. This feature could not have been improved upon for the foundation. Excavations were carried down to an average depth of about 5 ft. below this bottom, and all footings rested on hard gravel through which test-bars could not be driven more than 2 ft. The piers were liberally rip-rapped with the rock which had been used to load the coffer-dams, and other larger rock.

Two mixtures of concrete were used, namely: 1 : 2 : 4 for under water, and 1 : 3 : 5 for above water. Suitable gravel was found mixed with sand in about the right proportions and was used without screening. Daily tests of the aggregates were made, by volumes, so that it was known that the proper ratios were being maintained. The sand in the gravel was very clean and sharp, and free from loam and clay. The treatment of the aggregates was as follows: The specification called for measurement by volume, 1 bbl. of cement, 3 bbl. of sand, and 5 bbl. of stone, etc. Test boxes, holding half a batch of gravel, were filled with the aggregates, as placed upon the mixing platform, the contents were screened, and the sand and rock measured separately. If not in the right proportions more sand or more rock was added, as the case required. By making daily tests, and inspecting closely the materials as used, the proper proportions were maintained.

Table 1 shows the results obtained, and it is interesting to compare them with the results from mixtures made under ideal conditions, or those obtained where the ingredients were screened and graded more carefully. The ratios were determined by weight as well as by volume.

In making the calculations in Table 1 for the number of cubic feet of concrete per barrel of cement, the quantities used in the putty coats were not considered, as so few barrels were used that they would not have much effect on the results. The 1 : 2 : 4 mixture took 1.36 bbl., and the 1 : 3 : 5 mixture, 1.21 bbl. of cement per cubic yard of concrete. A perfect mixture of the 1 : 2 : 4 class would require 1.46 bbl. per cu. yd., and of the 1 : 3 : 5 class, 1.11 bbl. per cu. yd., which, as compared with the results obtained, shows that the 1 : 2 : 4 mixture fell short $\frac{1}{10}$ bbl. per cu. yd., and the 1 : 3 : 5 mixture over-ran $\frac{1}{10}$ bbl. per cu. yd. This would indicate that the cement used in the entire bridge was 83.88 bbl. more than called for theoretically.

The fact that the 1 : 2 : 4 mixture is short in cement and that the 1 : 3 : 5 mixture has a surplus, may be accounted for in two different

TABLE 1.—Results Obtained in the Concrete Work for the Piscataquis River Bridge.

	1 : 2 : 4 Concrete.			1 : 3 : 5 Concrete.			Barrels of 1 : 2	Total Cubic Yards.	Total Barrels.
	Cubic Yards.	Barrels.	Cubic Feet per Barrel.	Cubic Yards.	Barrels.	Cubic Feet per Barrel.			
North Abutment	....	....	....	205.00	251.50	22.00	4.0	205.00	255.50
No. 1 Pier	174.93	237.00	19.50	236.99	280.00	22.00	3.0	411.92	520.00
No. 2 Pier	268.88	337.00	21.30	292.63	348.00	22.70	3.0	561.51	688.00
No. 3 Pier	219.83	311.00	19.00	291.91	366.00	21.60	3.0	511.74	680.00
No. 4 Pier	124.57	182.00	18.60	239.24	302.00	21.40	1.5	363.81	485.50
South Abutment	70.00	101.00	18.70	429.03	508.50	22.80	3.0	499.03	612.50
Totals	858.21	1 168.00	19.80	1 694.80	2 056.00	22.20	17.5	2 553.01	3 241.50

FIG. 1.—GENERAL VIEW, LOOKING SOUTH. NORTH ABUTMENT AND PIERS 1 AND 4 FINISHED. PIER 2 NEARLY COMPLETE, AND PIER 4 BEING EXCAVATED.

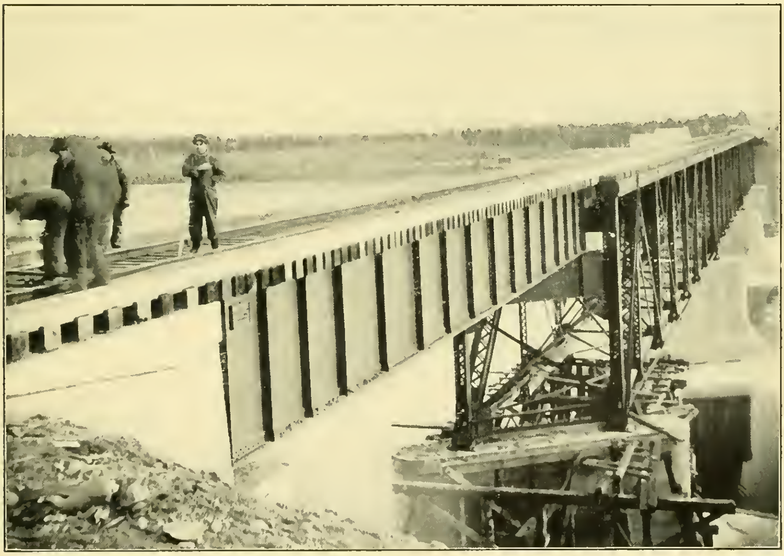

FIG. 2.—GENERAL VIEW OF COMPLETED BRIDGE.

ways: First, the aggregates were not graded properly, so that the voids were not completely filled; second, the quantities of the aggregates used for the different mixtures were not always exactly the same. The material was loaded into the mixer by wheel-barrows, the necessary quantity for a wheel-barrow for each mixture was determined, and they were all supposed to be loaded the same; in this, however, there was bound to be some variation.

For 1 bbl. of cement, or 4 bags (all the cement being in bags) 8 wheel-barrows of the aggregate were used in the 1 : 3 : 5 mixture, and 6 wheel-barrows of the aggregate in the 1 : 2 : 4 mixture. A check was had on the quantities used by the space occupied by the completed batch in the bucket, after being dumped from the mixer, as the buckets each held 1 cu. yd., it was found by test samples just how full they should be for the two different mixtures. Then, again, the aggregates not being uniformly graded by Nature would have a tendency to throw the resultant, cubic feet per barrel, under or above the theoretical results obtained from perfect mixtures. It seems, however, that where suitable ingredients can be found in their natural state, free from loam and clay—although a small percentage of either will not decrease the strength of the concrete—that as good work at less cost per yard can be obtained as where the sand is screened from the gravel and they are again mixed artificially; for, when the ingredients go into the mixer in their natural state, the machine has to do less work in mixing the cement with them, than when the sand, rock, and cement go in separately, for the mixer has not only to mix the cement in, but the sand and rock as well. Thus, with a mixer running for the same time, under the two different conditions, it would seem that a better mixture could be made from the natural ingredients. There were instances during the progress of the work when the resultant mixtures agreed exactly with the required quantity of cement per barrel. This would indicate that in such cases the voids were completely filled, the sand filling the interstices of the rock and the cement those of the sand.

The piers were designed with round noses, having a slight batter. At a point well above high water, on the front ends, a circular breakwater was made, having a batter of 2 : 3 and extending down to within 3 ft. of the footing course. The forms for these noses were made of 2-in. plank, about 4 in. wide, sections being built up in 8-ft. lengths as the concrete advanced. With plank of this width, the circular form could be made very readily, and with very smooth surfaces.

The concrete was mixed wet, no tamping being required, other than the shoveling over it received after being dumped from the bucket. Care was taken, however, that the sides of the forms were well worked around with shovels, which kept the rock back and allowed the soft material to come to the outside. The piers when stripped and dry received a coat of whitewash. The writer is not wholly convinced of the worth of this coat for work of this class, as it usually cracks and peels off. Where a good surface has been obtained, he would prefer to omit the whitewash coat.

Although the season was unusually wet, the progress of the work was delayed only for a few days. The river is affected rapidly by the rains, there being no storage in its water-shed, and the water rises and falls quickly. High water was encountered only once, when the second and third piers were first started, but the only damage was the washing away of a little of the coffer-dam embankment.

In the construction of the temporary falsework, piles were driven from a driver on a barrel raft. In no case could the piles be driven more than 5 ft., and the average was about 3 ft. This proved fully the firmness of the entire river bottom, as found during the excavation at the pier locations. Most of the piles were furnished with steel points. For absolute safety, piles should not be driven without some protection for the point, no matter through what kind of ground, as one can never tell what material a pile is to pass through.

The contractor for the concrete work was Mr. J. B. Mullen, who has had wide experience in similar work. Moses Burpee, M. Am. Soc. C. E., is Chief Engineer of the Bangor and Aroostook Railroad. W. S. McFetridge, M. Am. Soc. C. E., was Engineer of Construction, in charge of the "Medford Extension," and the writer was Engineer in Charge at the bridge.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

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OVERHEAD CONSTRUCTION FOR HIGH-TENSION ELECTRIC TRACTION OR TRANSMISSION.

Discussion.*

By Messrs. Joseph Mayer, W. K. Archbold, Charles Rufus Harte, Farley Osgood, and W. S. Murray.

Joseph Mayer, M. Am. Soc. C. E.—This paper, in the main, is an interesting collection of data and tables, useful in the design of overhead contact and transmission lines. Mr. Mayer.

The tables of wind velocities and pressures are especially useful for forming a correct opinion of the actual pressures. More stress might be laid on the fact that the observations of the Weather Bureau are made on the tops of high buildings, while the transmission lines, and especially the contact lines, are near the surface of the ground, where the wind pressures are much less. It would also be reasonable to assume less ice on the contact wire than on steel carrying strands, especially on lines of large traffic. Mr. Coombs' recommendations, in regard to wind pressures and unit strains, are generally reasonable. His paper, however, in common with most writings on the same subject, suffers from an insufficient consideration of the bending strains in the wires. In many designs, these bending strains are greater than the tensions, and their neglect leads inevitably to the selection of unsafe designs. They vary so greatly in amount that they cannot be provided for by neglecting them and adopting a large, but uniform factor of safety.

* This discussion (of the paper by R. D. Coombs, M. Am. Soc. C. E., printed in *Proceedings* for December, 1907), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Mayer. Mr. Coombs suggests for discussion a suspension from a single steel strand with 300-ft. spans and a distance of 6½ ft. from the points of suspension of the strand to the wire. Taking the wire as horizontal, and at the highest temperature, this, with a distance of 6 in. from the lowest point of the strand to the wire, gives a maximum vertical deflection of 6 ft. for the strand. He prefers this to the double catenary and strandless suspension. What justifies this preference, and what makes a superior suspension?

To have adequate conductivity, the wire must be copper; to give a smooth path for the bow, it must be solid; it is impossible to make it straight, but, under the pressure of the sliding bow, especially with high train speeds, it must have a large minimum radius of vertical curvature. The wire should not deviate horizontally far from the center line of track, or, if hung on the side, from a line parallel to it. It must remain safely suspended under the influence of its weight and that of sleet, the wind pressure, the pressure of the sliding bow, and the changes of temperature. The wire and its supporting structure should interfere as little as possible with the view of the signals. These ends should be attained by the simplest means, entailing the least cost of construction and maintenance. The bow lifts the wire, and the curvature of its motion, and not that of the freely hanging wire, must be considered. If the wire is supported at short intervals, it is lifted by the passing bow to positions above its supports; if these are rigid, the bow, at high train speed, oscillates rapidly up and down. Excessive bending strain in the wire, and jumping of the bow, with sparking, may result.

To ascertain whether the bow will run smoothly, its equivalent weight, the train speed, the cross-section of the contact wire and its tension, the distance apart of the suspenders, their weight, and that of the carrying strand, and the nature of the connection of the wire to the suspenders must be known. For high train speeds, a small equivalent weight of the bow is essential. Long bows are inevitably heavy; high train speeds, therefore, require short bows and small lateral deflections of the contact wire. The equivalent weight of the bow, however, depends even more on the design adopted than on its length. If smooth running is called for, at high speed, with an inferior heavy bow, short spans are inevitable with all suspensions. The rapid vertical oscillation of the bow is avoided in the strandless and the Siemens-Schuckert suspension described by Mr. Coombs.

In the strandless suspension, with long spans, there is a large change in the direction of motion of the bow at the infrequent suspenders; to make this practicable at high speed, the curvature of this motion must be chosen so that where it is convex downward the bow will not jump, and where it is convex upward neither the wire nor the bow nor the suspender will suffer from the increased pressure. In all suspensions there are large changes in the direction of motion of the bow at low overhead crossings and tunnel entrances, and sometimes at grade crossings. With inferior, heavy sliding bows a perfect design of the contact line for high speeds at these points is difficult or impracticable. A suspender fitted to change the variable direction of approach of the sliding bow into another variable direction of its departure, by a transition curve of large least radius at all temperatures, is here needed with all suspensions. Mr. Mayer.

To obtain a safe wire, its maximum strain must nowhere and never exceed about three-fourths of its elastic limit. It is exposed to bending strains and tensions. The former are often much larger than the latter. In catenary suspensions, a large grooved copper wire, about 0.3 in. wide and 0.6 in. in height of cross-section, is suspended from steel strands made up of very small wires. The copper wire has an ultimate strength of from 50 000 to 60 000 lb., the steel wires, 140 000 lb. or more. The modulus, *E,* of copper wires is 16 000 000, that of steel strands is 26 000 000 lb. per sq. in. of solid section. It is evident that the bending accompanying changes in vertical and horizontal deflections will produce much more serious bending strains in the large and weak copper wire than in the small and strong steel wires. For calculating them, the vertical and horizontal deflections of the wire under all conditions of load, wind pressure, pressure of the sliding bow, and changes of temperature must be determined. It is easy to provide steel ropes strong enough to carry the wire, the ice loads and the wind pressures. The main difficulty arises from their expansion and contraction caused by changes of temperature and tension. These and the wind pressures cause lateral and vertical curvature of the contact wire. Both the deflections and the drop of temperature increase its tension and produce at certain points large bending strains. To determine the degree of safety of the various suspensions, the largest bending strains and tensions in the contact wire and the ropes must be calculated. With regard to obstruction to the view of the signals, the fewer ropes, suspenders, posts, and bridges or brackets, the better.

For judging the suspension suggested by Mr. Coombs by these standards, and for finding whether it is sufficiently rigid to be suitable for use with a sliding bow that will run smoothly at high speed, and sufficiently strong to resist with adequate safety the incident forces, the size of the steel strand and the distance and nature of the suspenders must be reasonably assumed, and the deflections and consequent bending strains and tensions calculated.

Following Mr. Coombs' specification, a $\frac{3}{4}$-in. steel strand is ample, and weighs 0.89 lb. per ft. of span. The 0000 copper wire weighs 0.64 lb., and gas-pipe suspenders, 12 ft. apart, about 0.33 lb., giving a total weight of 1.86 lb. per ft. of span. Ice $\frac{1}{2}$ in. thick on all parts

Mr. Mayer. weighs 1.63 lb., giving a total weight, with ice, of 3.49 lb. per ft. of span. Taking the wire to consist approximately of two 0 wires directly above each other, it is 0.65 in. high and half as wide. The wind pressure on the bare metal of wire suspenders and strand, taking 12 lb. per sq. ft. all through, is 1.63 lb., and that on the ice-covered structure, with 8 lb. per sq. ft., 2.57 lb. per ft. of span. Assuming a tension of 1 000 lb. in the contact wire, the lateral deflection of the wire, at maximum temperature, would be 3.15 ft. This assumes that the wire is held at the ends of the spans by steady braces. If these were absent, the lateral deflection would be much larger and the needed sliding bow would be altogether impracticable.

Where sliding bows are used, the wire must run alternately to the right and left of the center line of track so as to distribute the wear over a considerable length of the bow. Taking the lateral displacement of the wire at the brackets to be 1 ft., and allowing ½ ft. for the lateral vibration of the sliding bow, the latter must be 6.96 ft. long to catch the wire, with the strongest winds assumed. A sliding bow of this length, which will not jump at the suspenders, with moderate train speeds, can be designed.

For the highest present steam railway speeds, a much larger tension in the contact wire or more frequent suspenders are needed to prevent jumping and sparking, with the usual connection of the wire to the suspender. Jumping and sparking might also be prevented by a contrivance allowing the wire to rise at the suspenders, without lifting them, when the sliding bow passes. With steady braces, which are practically unavoidable with this design, the wire carries, at maximum temperature, 0.49 lb. of the total wind pressure of 1.63 lb. per ft., to the steady braces, and its tension is thereby increased. Much more serious is the bending strain in the wire at the clamps which connect it to the steady braces. These clamps may be designed to avoid bending strain in the wire at maximum temperature without wind. In this case, the lateral bending strain in the wire at the end of the clamp, at highest temperature and wind pressure, is 35 300 lb. per sq. in. For the wire here assumed, this bending strain is given by the formula, $s = \frac{19\,200\; Q\, h}{\sqrt{T}}$ where s is the bending strain, in pounds per square inch, and T is the tension in the wire. If T is decomposed into a component having the direction of the wire at the end of the clamp and one normal to it, the horizontal component of the latter is $Q\ h$, and the vertical component $Q\ v$. For the vertical bending of the same wire, the formula is $s = \frac{18\,000\; Q\, v}{\sqrt{T}}$. At the same time, with this bending strain of 35 300 lb. per sq. in., the tension in the wire is 10 800 lb., giving a combined strain of 46 100 lb. per sq. in.

The passing sliding bow increases the total strain to nearly 48 000 lb. per sq. in. The horizontal bending strain may be reduced, theoretically, to one-half, by using two steady braces at each bracket and connecting them to the wire by hinged clamps, the hinges being vertical. The steady braces themselves should have horizontal hinges permitting the clamps to rise and fall with changes of temperature and wind pressure and the passing of the sliding bow. By this rise and fall, the principal vertical bending strains of the wire, except those due to the passing bow, are transferred to the nearest suspender; therefore, they need not be added to the horizontal bending strains occurring at the steady braces. With hinged double steady braces, connected to the wire by hinged clamps, the total strain per square inch at the highest temperature, with the assumed wind pressure, is approximately 30 500 lb. This assumes that all the hinges work without friction. The friction of the hinges may increase this strain considerably. At the lowest temperature, without ice, and with the largest wind pressure, the lateral deflection of the wire is 15 ft., its tension is 5 000 lb., or 30 080 lb. per sq. in. Mr. Mayer.

The bending strain at the steady braces due to horizontal bending is 27 200 lb. per sq. in. with single, and half as much with double, braces and hinged clamps. This gives combined strains of 57 280 and 43 680 lb. per sq. in. These strains are both increased about 1 000 lb. by the passing sliding bow. Since they exceed the elastic limit, the wire will bend before the strains reach the amount calculated. Repeated forward and backward bending will produce rupture. To reduce these large bending strains and tensions in the contact wire, smaller deformations must be obtained. These can be secured by smaller deflections of the carrying strand which requires either heavier strands or shorter spans.

With a $\frac{7}{8}$-in. strand of 3 ft. maximum vertical deflection and 300 ft. span, carrying 6 in. below the strand at the center of the span a 0000 grooved wire which is horizontal and has 1 000 lb. tension at the highest temperature, without wind, the deflections and strains in the wire are as follows: At the highest temperature and wind pressure, the horizontal deflection of the wire is 2.17 ft., its upward deflection is 0.23 ft., the tension is 8 260 lb. per sq. in., the horizontal bending strain is 20 580 lb. per sq. in. with single and half as much with double steady braces. This gives, with the latter, a combined strain of 18 550 lb., which is increased to about 20 500 lb. per sq. in. by the passing sliding bow. This is a great improvement over the corresponding 30 500 lb. with 6 ft. deflection of the strand.

At the lowest temperature, with the highest wind pressure, without ice, the vertical deflection of the strand is 1.85 ft., its lateral deflection is 0.9 ft. The vertical upward deflection of the wire is 1.18 ft. and its lateral deflection 1.05 ft. The tension in the wire is 4 760 lb., Mr. Mayer. or 28 640 lb. per sq. in.; the horizontal bending strain, with double steady braces, is 9 290 lb., giving a combined strain of 37 930 lb. per sq. in. This is increased to about 39 000 lb. by the passing sliding bow. With hinged steady braces, most of the vertical bending strain, amounting, if concentrated, to 24 800 lb. per sq. in., is transferred to several of the nearby suspenders. If these suspenders have clamps attached to them by horizontal hinges, allowing oscillation of the wire in a vertical plane, then the vertical bending strains are certainly smaller than the horizontal ones; if there are no such hinges, such strains are probably larger than the horizontal bending strains with double steady braces, but they cannot easily be calculated with accuracy. With ice and wind, at the lowest temperature, the lateral deflection of the wire is 1.48 ft., the upward deflection is 0.46 ft., its tension is 4 760 lb. or 28 640 lb. per sq. in. The horizontal bending strain, with double steady braces and hinged clamps, is 13 100 lb., giving a combined strain of 41 740 lb. per sq. in. This is increased about 1 000 lb. by the passing bow.

The maximum tension in the strand is 16 200 lb., which gives a factor of safety of 3, provided the small bending strains are neglected. A sliding bow 5 ft. long is needed, and can be designed so as to give smooth running at all but the highest speeds. It is evident that the strains in the wire are still excessive where the wind pressures and ice loads prescribed by the specification really occur. As these are of rare occurrence, a structure of this design, with improved hinged double steady braces, connected to the wire by hinged clamps, will probably, in most situations, last a number of years. It would have about the rigidity of a double catenary suspension of the same span with two strands of $\frac{9}{16}$ in. diameter and 6 ft. vertical deflection, having the wire 6 in. below the lowest point of the strands. The largest lateral deflection of the wire of this latter suspension, with the wire tension and wind pressure here assumed, is approximately 2.25 ft. In the double catenary suspension, no steady braces are used, the bending strains in the wire due to its vertical and lateral deflection are distributed to several clamps near the ends of the spans. They cannot easily be calculated with accuracy, but are probably somewhat smaller than in the best single catenary suspension of the same span. The tensions in the wire are nearly the same in both designs here compared. The double catenary suspension is certainly superior in strength to a single catenary suspension of the same span and lateral deflection with single steady braces. All these designs are far inferior in safety to railroad bridges.

Taking now a 0000 round wire, hung from special suspenders, with 300-ft. spans and 4 ft. maximum vertical deflection, with strain adjusters 1 mile apart, the adjusters changing the length of the spans four times a year, so that the variation of temperature with one length of span does not exceed 84° fahr.: The largest lateral deflection of the wire is 2.54 ft. A sliding bow 6 ft. long is required. The tension in the wire at the lowest temperature, with wind and without ice, is 3 260 lb. or 19 600 lb. per sq. in. The maximum bending strain at the same time is 6 250 lb., giving a total strain of 25 850 lb. per sq. in. The corresponding strain in the suspension suggested by Mr. Coombs is 44 680 lb. per sq. in. In the single catenary suspension, with 3 ft. maximum vertical deflection of the strand, it is 39 000 lb. with the best design. Mr. Mayer.

With a coating of ice, ¼ in. thick on the contact wire, increasing its diameter ½ in., the maximum tension at the lowest temperature, and with a wind pressure of 8 lb. per sq. ft. of ice-covered wire, is 21 530 lb., the bending strain is 6 250 lb., giving a total of 27 780 lb. per sq. in. A greater thickness of ice on the contact wire would make it difficult to collect the current. Where there is considerable traffic, the wire will be generally several degrees warmer than the atmosphere, and less ice will form on it than on steel strands carrying but little current, and the passing sliding bow will knock off much of that which forms. It is reasonable, therefore, to assume a smaller amount of ice on the contact wire than on the strands. The maximum strain in the wire, with ice ½ in. thick, and a wind pressure of 8 lb. per sq. ft., at the lowest temperature, in the best of the single catenary suspensions of the same span is 42 700 lb. per sq. in.; this would be but little reduced by assuming the ice on the contact wire ¼ in. thick, the strain without any ice being 39 000 lb.

The strandless suspension here described requires, for smooth running with a speed of 70 miles per hour, a sliding bow of 4 lb. equivalent weight, 6 ft. long. Such a bow can easily be designed, but, as far as the speaker is aware, it is not at present in the American market. The bows in use are designed for smaller speeds. If they are to be used with high speeds, shorter spans are necessary. The 4-ft. deflection of the contact wire requires a larger range of vertical motion of the bow than the catenary suspension of the same span, in which the height of the wire varies only about 2 ft. Though 300-ft. spans are entirely practicable and safe, with improved strandless suspension and a speed of 70 miles per hour, they will not give continuous contact at this speed without improved sliding bows.

The speaker has invented another suspender, which can be used at any speed with ordinary sliding bows of large equivalent weight, and reduces still further the bending strains. With it, 300-ft. spans can be safely used. As the sliding bow may be heavy, it may be made longer, and a maximum vertical deflection of 4½ or 5 ft. may be adopted, thus reducing greatly its maximum tension. This suspender will be described later.

The calculation showing the excessive bending strains in the

Mr. Mayer. single catenary suspensions of 300-ft. span is confirmed by practical experience with long-span electric transmission lines. In these, solid wires were first used, but they broke at the insulators even with moderate tension per square inch. Stranded wires, therefore, are now used with long spans. Many experiments have been made with single catenary suspensions. As a result, the present practice in America and in Europe, as far as known to the speaker, does not show any existing spans of more than 160 ft. The importance of reducing the lateral and vertical deflection of the contact wire, and thereby its bending strains, is fully appreciated by the designers of most, if not all, of the existing structures in America. The sliding bows are generally 4 ft. long, and could not be used with large lateral deflections.

Taking a design with 150-ft. spans of a steel strand of $\frac{7}{16}$ in. diameter, with 16 in. maximum vertical deflection, the 0000 grooved wire being horizontal at the highest temperature and 4 in. below the strand at the center of the span, the following deflections and strains are obtained with a variation of temperature of 140° fahr., and the loads and wind pressures mentioned by Mr. Coombs: The weight of wire, strand and suspenders is approximately 1.04 lb. per ft., the wind pressure, without ice, is 1.15 lb. per ft. The weight, with ice, is 2.33 lb., and the wind pressure is 2.10 lb. per ft. With the greatest wind pressure, and at the highest temperature, the lateral deflection of the wire is approximately 0.95 ft., its tension is 1 284 lb., or 7 730 lb. per sq. in., the horizontal bending strain, with ordinary steady braces, is 17 410 lb., giving a combined strain of 25 140 lb. per sq. in.; this is increased to about 27 100 lb. by the passing of an improved sliding bow of a maximum dynamic pressure of 25 lb.

At the lowest temperature, with the greatest wind pressure, and without ice, the upward deflection of the wire is approximately 0.3 ft., the horizontal deflection is 0.38 ft., its tension is 4 640 lb., or 27 920 lb. per sq. in. The horizontal bending strain is 6 630 lb., giving a combined strain of 34 550 lb. per sq. in. This is increased to about 35 500 lb. by the passing sliding bow. If the wire is firmly held at the steady brace, so that it cannot rise and fall, the combined strain due to tension and horizontal and vertical bending is about 40 000 lb.

With ice having an average thickness of ½ in., at lowest temperature and with the greatest wind pressure, the wire has a lateral deflection of 0.6 ft. and a downward deflection of 0.07 ft. Its tension is 28 160 lb., and the horizontal bending strain is 10 510 lb., giving a combined strain of 38 670 lb. per sq. in. This is increased by about 1 000 lb. per sq. in. by the passing sliding bow. The horizontal bending strain may be reduced to one-half and the vertical bending strain transferred, by a perfect double steady brace, allowing the wire to rise and fall and turn.

Wire having an elastic limit of 40 000 lb. per sq. in. can be obtained, and, with little ice and moderate wind pressures and changes of temperature, gives an approximately safe structure with the usual designs. Mr. Mayer.

The maximum tension in the steel strand, with the foregoing loads, is 5 000 lb., the ultimate strength of a cast-steel strand is about 13 600 lb.

These results explain why much longer spans are not used with single catenary suspension.

It is evident that a structure having the factor of safety of a railroad bridge is not practicable with ordinary catenary suspension, in most climates. If Mr. Coombs' description of the Siemens-Schuckert suspension is correct, and if the pulleys over which the contact wire is carried have a diameter of 12 ft., larger pulleys being impracticable, the bending strain in the wire here assumed would be 36 000 lb. per sq. in. If the tension in the contact wire is made small, the lateral deflection of the wire would be much increased, and long spans with large strand deflection would be impracticable. This suspension is used with spans of 48 m., with a steel carrying structure where long spans are very desirable. If the designers had thought them practicable, they would probably have adopted them. Spans of 300 ft., with single catenary suspension, therefore, are not sustained by precedent; they cannot be defended successfully by theory, and they will probably prove short-lived if tried under conditions approximating those here assumed.

The maximum strains which demonstrably exist in the contact wires with catenary suspension show what a copper wire can stand, at least for a few years. They make it extremely probable that a wire in which the maximum strain never and nowhere exceeds 30 000 lb. per sq. in. is abundantly safe.

W. K. Archbold, Esq.—Regarding the matter of protective structures where transmission lines cross railroad tracks, Mr. Coomb's paper should help to standardize the practice, which has varied extremely, as the speaker has had occasion to note. Under the direction of Thomas H. Mather, M. Am. Soc. C. E., an overhead construction has recently been designed and installed on the line of the Syracuse, Lake Shore and Northern Railroad, running from Syracuse to Baldwinsville, N. Y. The line is about 5 miles long, and is provided with single-catenary trolley construction presenting some new features. Mr. Archbold.

Mr. Mather thinks that the work has not yet advanced far enough to warrant the presentation of a formal paper, and therefore the speaker will make simply a preliminary presentation of the prominent features of the construction.

The trolley wire is hung from a messenger cable supported on bridges spaced 300 ft. apart, from center to center. The bridges, as

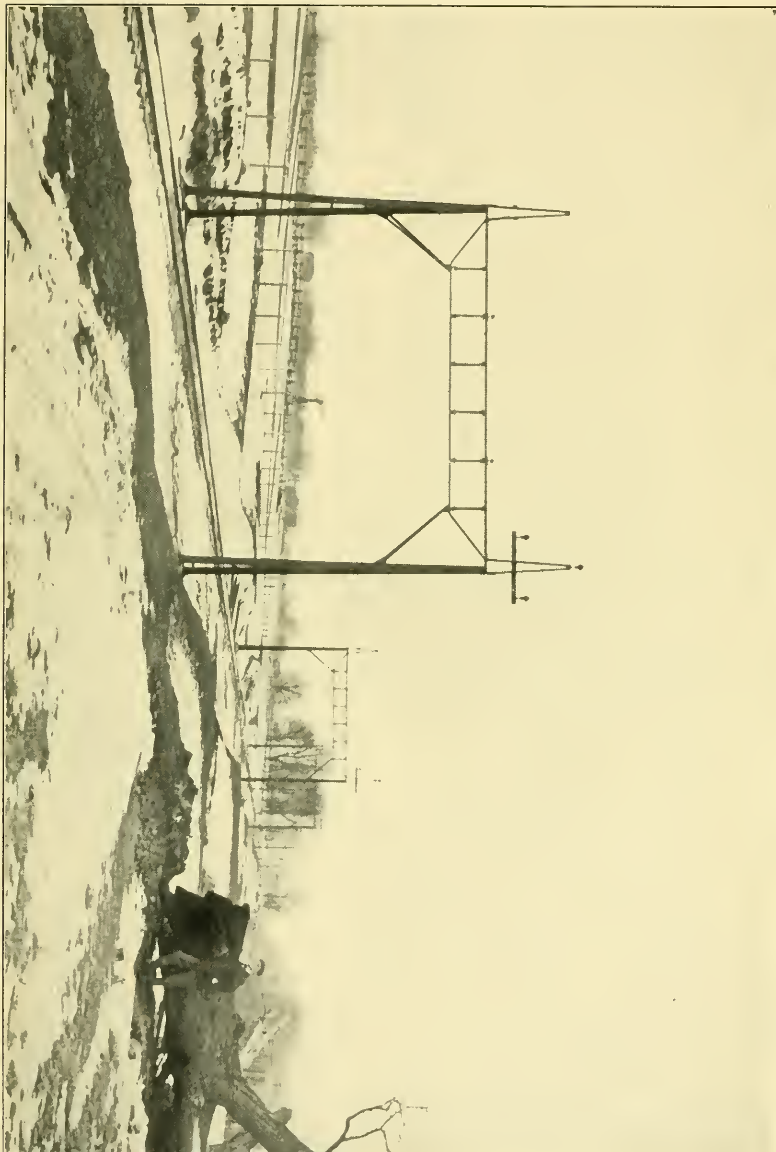
Mr. Archbold. shown in Plate XXXIV, consist of light trusses on bents 30 ft. apart, from center to center. The bents are each composed of two 8-in. channels, 6 ft. apart at the base, converging to 8 in. at the top, and supported on concrete pedestals, 20 in. square, and of depth varying with the nature of the ground. The trusses have an 8-in. channel top chord and 6-in. channel bottom chords, set with the flanges down. The diagonal members are $\frac{5}{8}$-in. rods, and the struts $2\frac{1}{2}$ by $2\frac{1}{2}$ by $\frac{1}{4}$-in. angles. The struts are flattened and bent over at the ends, and are riveted to the channels.

To the top chord of each truss are bolted malleable-iron pins to which are cemented porcelain insulators for the messenger cable. The three-phase high-tension line is supported on steel A-frames at each end of each bridge. The construction is designed for a wind pressure of 8 lb. per sq. ft. on the trolley and messenger cables, covered with $\frac{1}{2}$ in. of ice, a somewhat lower ice load being assumed on the high-tension cables, which are of No. 2 copper. The structure is computed as a braced portal, the unit strains under the assumed wind and ice load being 22 500 lb. per sq. in., reduced for compression members.

The catenary is strung for a net sag of 6.5 ft. at 100° fahr. At 20° fahr., the sag is about 5.5 ft., and the trolley is about 1 ft. higher at the center of the span than under the bridges, the height from rail to trolley being 18 ft. at the bridges. Stranded steel messenger, 15 000-lb. wire, $\frac{7}{16}$ in. in diameter, supports steel hangers, $\frac{5}{8}$ in. in diameter, spaced 10 ft. apart, from center to center. These hangers are of the Ohio Brass Company type, and are attached to the messenger cable with a sister hook through the base of which the rod is threaded and drawn up tight against the messenger cable. The 0000 grooved trolley is secured to the hanger by Detroit clamps.

At each bridge there is a span-wire steady strain (not shown in the photograph), the trolleys being insulated from the bridge and from each other by 6-in. wheel-type porcelain strain insulators. The messenger cable is dead-ended on an equalizer attached to a pair of these insulators, which are connected by short cable loops to a similar pair secured to the anchor bridge.

A test which came on the line during construction showed the effect of a broken messenger wire. When about half a mile of wire had been pulled up, and hangers were on three or four of the spans, the dead-end arrangement broke, allowing the line to go. The insulator broke on the bridge next to the dead-end bridge, but the trouble did not extend any further than that point. Some of the men on the work thought the foundation of the second bridge was raised a little, but there seems to be considerable difference of opinion on that point, and certainly no damage was done. The idea has been that the bridges would be what might be called semi-anchored; or, in other words, the effect of a break would not go beyond two or three bridges in either



SINGLE CATENARY CONSTRUCTION, SYRACUSE, LAKE SHORE AND NORTHERN RAILROAD.

direction. It is possible, too, that the heavy rods and tight clamping effect, obtained both at the messenger and trolley wires, assisted considerably in holding the line. At any rate, no damage was done which could not be repaired quickly. Mr. Archbold.

The high-tension wire is strung with a sag of 40 in. at 20° fahr., and has a net clearance of about 24 ft. from the track. As the trolley may at some time be operated at 6 600 volts, single-phase, all the insulation of the catenary, steady strains, etc., is designed to withstand this voltage. At present, however, it is being operated at 600 volts, direct-current. In regard to the lateral stiffness of the single catenary with supports 300 ft. apart, it may be noted that the line has been in operation about 10 days, and, thus far, very little deflection or rolling can be observed under the action of the wheel trolley with a tension of about 25 lb. During this time there have been temperature changes of 50° and wind velocities of 45 miles per hour. These conditions seemed to make no difference in the operation, even before the steady strains were installed.

The speaker feels justified in saying positively that, with this type of construction, there will be no difficulty from side-sway on the 300-ft. span. It is yet to be determined whether the line is too stiff in the vertical plane, but that cannot be determined positively except by operation extending over a considerable period, and including hot as well as cold weather. The preliminary tests which have been made indicate that there will be no difficulty.

This line is a re-location, to shorten the running time and provide a double track on private right of way between Syracuse and Baldwinsville. The old single-track line is on a highway, and is about ¾ mile longer than the new location, which will form part of a new high-speed electric road between Syracuse, Fulton, and Oswego, a total distance of about 35 miles.

Charles Rufus Harte, M. Am. Soc. C. E.—In the field of transmission- and distribution-line construction, each engineer has been largely a law unto himself, and Mr. Coombs' effort to secure some measure of standardization is much to be commended. At the same time, local conditions very largely govern, and the successful construction of one locality may be of little value even in comparatively near sections. Mr. Harte.

In addition to the causes given by the author, a short circuit may be caused by the swaying together of two phases of the circuit. This, however, may be prevented by spacing the wires a distance apart equal to at least twice the versed sine of the sag. Thus the Missouri River transmission has a spacing of 78 in.; the 1 450-ft. Connecticut River span of the Springfield-Suffield Line, 84 in.; while the Madison River Line, of Montana, has 108 in. As an additional precaution, the wires are often arranged so that no two are in the same horizontal plane.

Mr. Harte. This is characteristic of the Connecticut River span, the Anglo-Mexican, the Southern Power, and many other transmission lines. The large spacing (of the triangle), with two wires in a vertical plane, also materially reduces the likelihood of interference from large birds, or branches or wires blown against the line. In practice, however, because of their weight and low periodicity, long spans usually swing in unison, thus maintaining the spacings.

Mr. Coombs states that insulator troubles are largely due to misdirected savings. While it is true that there are to-day many lines operating without change at higher voltages than designed for, and on which the insulators were poor for the original voltage, there are many other lines where, although no expense has been spared, insulator troubles are very serious.

On the seacoast, particularly in Southern California, heavy salt fogs cause troubles which, as far as the speaker knows, have not yet been overcome successfully; in the alkali deserts, the so-called salt storms result in losses by very remarkable brush discharges and leakages; and where lines are near steam-railroad right of way the oil and water from the exhaust condense on the insulators and then collect coal and other dust until the creeping surface is largely covered, causing heavy leakage, and burning wooden pins. This condition promises to be a very serious problem in steam-road partial electrification. The deposits from salt and dust storms are washed off by the rains, but the oily coating resulting from locomotive exhausts is not affected by water.

While a "campaign of education" may be of assistance, the small boy with his sling-shot and the man with the gun will always menace seriously the welfare of insulators in settled sections. Dark-colored glazes, being less conspicuous, are being used in many cases. With medium voltages, compact insulators of the Redlands or Crown type are used, and one large manufacturing company grooves the insulator top, with the idea that the marksman will knock out the portion inside the groove and then retire satisfied, leaving enough insulator on the pin to protect the line. As a matter of fact, against a bullet of any weight there is little choice as to type. The speaker tested the three types shown, Fig. 1, Plate XXXV, using a Winchester $\frac{38}{100}$-caliber rifle, reproducing line conditions as far as possible, and firing one shot at each. Fig. 2, Plate XXXV, shows the result. In New England or the East generally, a gun of such heavy caliber would rarely, if ever, be used.

Serious sleet storms, fortunately, are not common, and are rarely of great extent. While failure may be due to the dead weight of the accumulation, the usual cause is more complex. In a strong wind, the sleeted wires, because of the greatly increased area and weight, sway

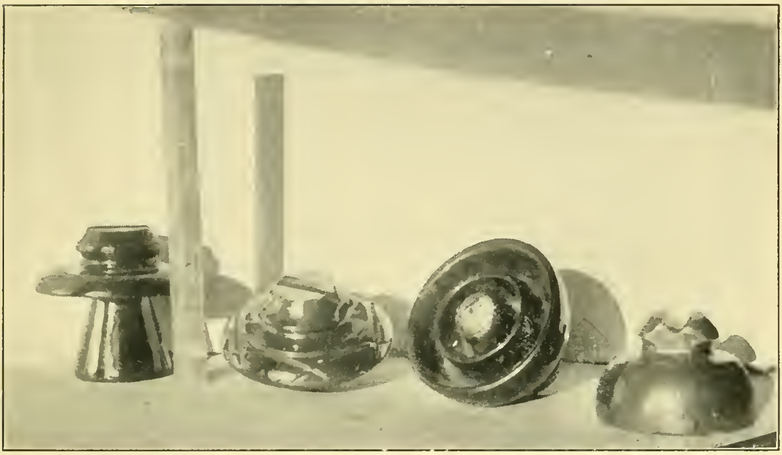


FIG. 1.—INSULATORS, BEFORE THE TEST.

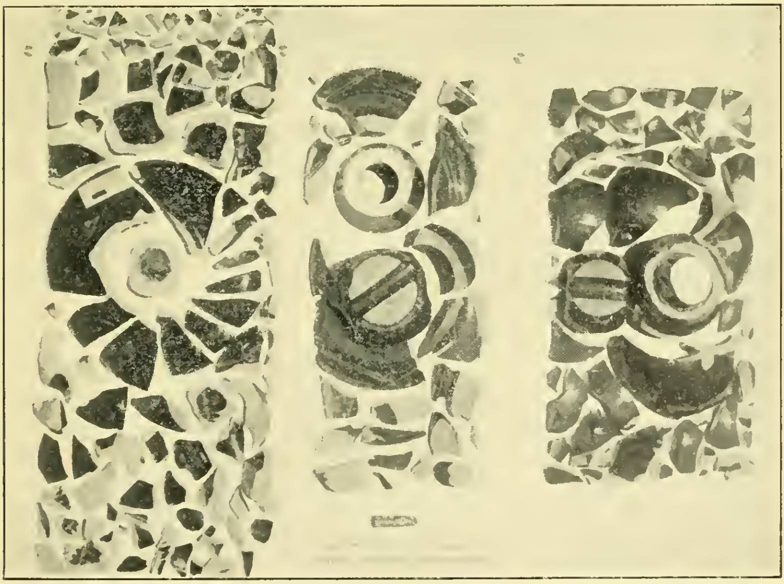


FIG. 2.—INSULATORS, AFTER THE TEST.

until the vibrations become synchronous with the natural period of a pole of the line. If this pole fails, the resulting long span usually has weight enough to pull down other poles on each side. Mr. Harte.

Aluminum does not appear to hold sleet as does copper, and lines transmitting much power are usually a little warmer than the air, so that they throw off the coat quickly, therefore, an allowance of ½ in. of ice around the wire is sufficient to provide for all reasonable contingencies. The sleet coatings, however, may become very heavy. Fig. 1, Plate XXXVI, shows an accretion of clear ice on a twig, the coating having a diameter of practically 3 in.; and C. J. H. Woodbury, M. Am. Soc. C. E., recently advised the speaker of specimens he had seen, one from Wayland, Mass., 4½ in. in diameter, and one from Nahant, 8 in. in diameter. Fig. 2, Plate XXXVI, shows the accumulation on wires caused by the same storm which developed the twig coating. This was at Winsted, Conn., on February 21st, 1898.

Occasionally, sleet and snow storms make trouble by bridging over the creeping surface of the insulator, thus causing leakage to the cross-arm. This, however, can be prevented by designing the pins and cross-arms so as to leave no large catchment area, and by maintaining a considerable distance from the insulator top to the arm.

The pressure variations in wind storms, cited by Mr. Coombs as a reason for using a low value, are more properly reasons for introducing a sway factor. A field of grain or long grass, or ivy on a house, observed in a wind storm, shows very clearly the successive pressure waves. On a transmission line, when such impulses are synchronous with the natural period of one of the poles, stresses are set up in the latter far in excess of those due to the direct forces themselves. In wooden pole lines there are stress transfers, due to the elasticity of the poles and the slipping of the wires at the insulators, which relieve these unusual conditions; in tower lines, the greater rigidity of construction largely prevents such relief, and the action must be considered in designing.

Mr. Coombs gives a series of very valuable tables of wire factors, but there is a matter in connection with solid copper that, as far as the speaker is aware, has been given practically no attention by any investigator. The standard American copper-wire bar weighs approximately 200 lb. This is rolled to a rod of diameter depending on the gauge of the wire it is to make. For 0000 trolley, this rod is about 560 mils in diameter, and is less than 300 ft. long. To secure the long commercial lengths of trolley, the rods are brazed together, the scarf having an angle of about 20°, the brazing being done with a mixture of silver and tin at a temperature near 800° fahr. As a result, the rod is annealed at the scarf, and the subsequent drawings to a diameter of 460 mils do not harden this annealed portion.

From tests of brazes, it appears that their strength is only eight-

Mr. Harte. tenths of that of stock wire. Incidentally, it should be noted that the strength of grooved 0000 wire is from 4 to 5% lower than that of round wire, due to the fact that more work is done upon the latter. Grooved wire cannot be given a second reduction after the groove has been made.

TABLE 11.—TESTS OF 0000 GROOVED TROLLEY WIRE.

Length tested in each case, 10 in.

TESTS OF BRAZED JOINTS.

Test number.	Height of cross-section at braze, in inches.	Length of joint, in inches.	Breaking strain, in pounds.	Percentage of elongation, in 10 in.	Remarks.
3 629	0.475	1.4	5 852	1.5	Apparently good braze. Broke in joint, with partial separation of braze.
3 630	0.476	1.5	6 306	6.7	Apparently good braze Broke outside joint.
3 631	0.448	0.9	4 429	14.7	Apparently good braze. Metal reduced in section by filing. Broke in joint, with partial separation of copper.
3 632	0.475	1.4	6 175	6.5	Apparently good braze, broke outside joint.
3 633	0.452	1.1	4 515	13.9	Apparently poor braze. Metal reduced in section by filing. Broke in joint, with partial separation of copper.
3 634	0.476	1.3	6 390	3.8	Apparently good braze. Broke in joint. Separation very slight. Flaw in copper at point of rupture.
Average.	0.467	1.3	5 611	7.9	

TESTS OF WIRE FROM SECTIONS BETWEEN JOINTS.

3 628	0.478	....	7 087	7.0	Fracture silky, angular.
3 635	0.478	....	7 085	5.7	" " "
3 636	0.478	....	7 166	5.4	" " "
3 637	0.478	....	7 077	8.3	" " "
3 638	0.478	....	7 076	5.9	" " "
Average.	0.478	....	7 098	6.5	" " "

The failure at a braze does not occur in the braze itself, but in the area immediately adjoining; the break is usually parallel to the scarf, but there is invariably a skin of copper on the braze.

At least one wire manufacturer uses a specially heavy wire bar when requested, thus having fewer brazes per mile, but, in any case, it is the braze which determines the strength of the line. For this reason, as well as because of its flexibility and consequent ease of handling, stranded copper is much better than solid for transmission-

Fig. 1.—Sleet Accretion on Twig, Winsted, Conn., February 21st 1898.

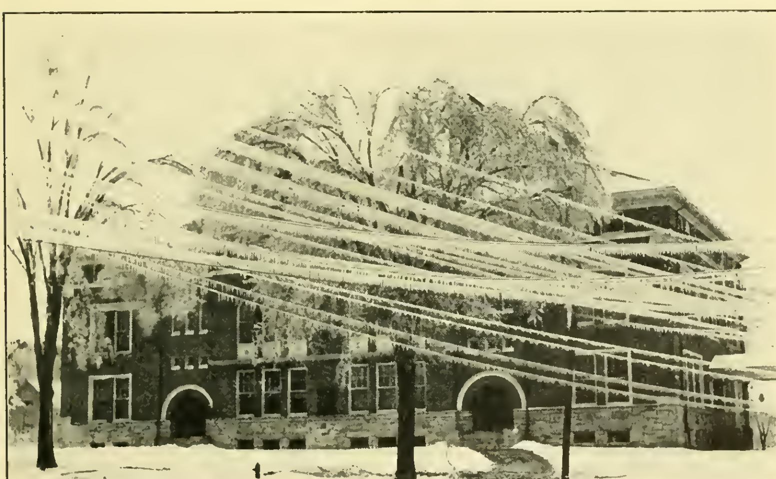


Fig. 2.—Sleet Accretion on Wires. Winsted, Conn., February 21st, 1898.

line work, the strand brazes being distributed along the made-up cable. Mr. Harte.
It should be added that the problem of the braze is now occupying the attention of a number of the large wire manufacturers, and it is hoped that decided improvements will follow.

Mr. Coombs refers to a grooved trolley wire having an area of 0.155 sq. in. and an ultimate strength of 8 800 lb. The speaker would like to have further details of this wire. Commercial, American, 0000, grooved wire, having a cross-section of 0.167 sq. in., this being a little more than 7% greater than the wire referred to, in a series of tests, failed to reach an ultimate strength of 8 000 lb., the break usually occurring at about 7 800 lb.

Mr. Coombs' type of anchor is good for comparatively light stresses, but for heavy spans it is desirable to arrange the insulators in pairs, or, if in tandem, to the catenary of the span, to secure uniform stress distribution.

Where the sag must not fall below fixed limits, provision must be made for adjustments of considerable extent. In Mr. Coombs' design, any considerable take-up on the turn-buckles would result in slack on the saddle, not readily cared for. This may be avoided by using a double saddle, both parts being movable, the slack looping between. The Connecticut River crossing of the Springfield-Suffield line has a crossing span attached to a movable cross-head controlled by a long screw with an adjusting nut, Fig. 9. The main line taps into the crossing span at the cross-head, and has a "pigtail" to care for the variation in length.

Under specifications, to bar thin wiped galvanizing, it is desirable to require the galvanized metal to stand four immersions, of 60 sec. each, in a saturated solution of copper sulphate, at 70° fahr. After each immersion the test piece should be dipped into clean water and then wiped dry; no metallic copper should appear after the fourth immersion.

The speaker wishes Mr. Coombs had treated the subject of protection of line crossings at greater length. Apparently, the crossing described is protected, over and above the general line, only by taking additional precautions to relieve the crossing towers from stresses due to adjoining spans, and in certain details of anchorage of line, provision being made for the installation of a cradle at a later date if desired.

As far as line strength is concerned, a crossing differs from a normal span only in possible greater length, or in restrictions as to height of wire; and a failure at this point, in its effect on the service, does not differ from a failure elsewhere; but, in the possibilities of damage to train, to passengers or others on platforms or highways, or to other lines, the crossing becomes one of the most critical line points, and the method of safeguarding it is of the utmost importance.

Mr. Harte. Protection may be effected by:

1.—Mechanically preventing a broken wire from getting into the danger section;
2.—Strengthening the upper wires so that failure is practically impossible;
3.—Grounded arms to cut off a broken wire at the pole top before it can reach the line below.

Of the first class are the various cradles, all open to the criticism that they offer large areas for sleet lodgment, and most of them that they are very expensive.

A very simple type consists of telegraph wires strung between multiple pin arms. The system is grounded, but the small section of the wires is usually a guaranty that they would be burned through by the arc in case of a fall of the power line. Fig. 1, Plate XXXVII.

A modification consists of three longitudinal wires with cross-bars of hard wood; other variations include sheets of wire netting, networks of wire strand more or less substantially fastened together, up to the very impressive cradles of heavy strand with cross-bars of flat iron. As clearly appears in the case shown, Fig. 1, Plate XXXVIII, wood bar cradles are apt to lose members from breakage or otherwise, while the heavier wire cradles often sag, becoming a positive menace. In Fig. 2, Plate XXXVII, is shown a wire-strand cradle which has sagged to an extent requiring the power company to protect its lines by the support wires strung on the top arm of the transmission line.

While the more substantial types, if large enough to keep a fallen wire from blowing out again, are no doubt efficient along certain lines, their great cost, and the excessive stress imposed by them upon their supports, even without the great loads of sleet they are sure to catch, make them very undesirable.

The ideal protection is the so-called short-span method. Here the crossing span and the two spans adjoining are arranged so that the distance apart of the poles is less than the distance from the cross-arm of the upper line to the lower line; it is thus impossible for the two lines to touch, under any circumstances, while the adjoining short spans prevent a broken wire from swinging into the crossing-span section. Unfortunately, crossings usually occur in highways where limitations as to pole locations prevent the use of this method.

The method most generally applicable, and, in the speaker's judgment, the best, is that of reinforcing the line by a set of messenger cables. This plan has the great advantage that the line is locally doubled in strength, with but little increase in weight or in exposure area, and the cost is nominal.

In a recent and satisfactory design, messengers of No. 2 stranded copper are used, and to them the line wire is tied every 4 ft. A

FIG. 1.—WIRE GRIDIRON UNDER 33 000-VOLT TRANSMISSION LINE.

FIG. 2.—WIRE CRADLE OVER 11 000-VOLT TRANSMISSION LINE

Mr. Harte.

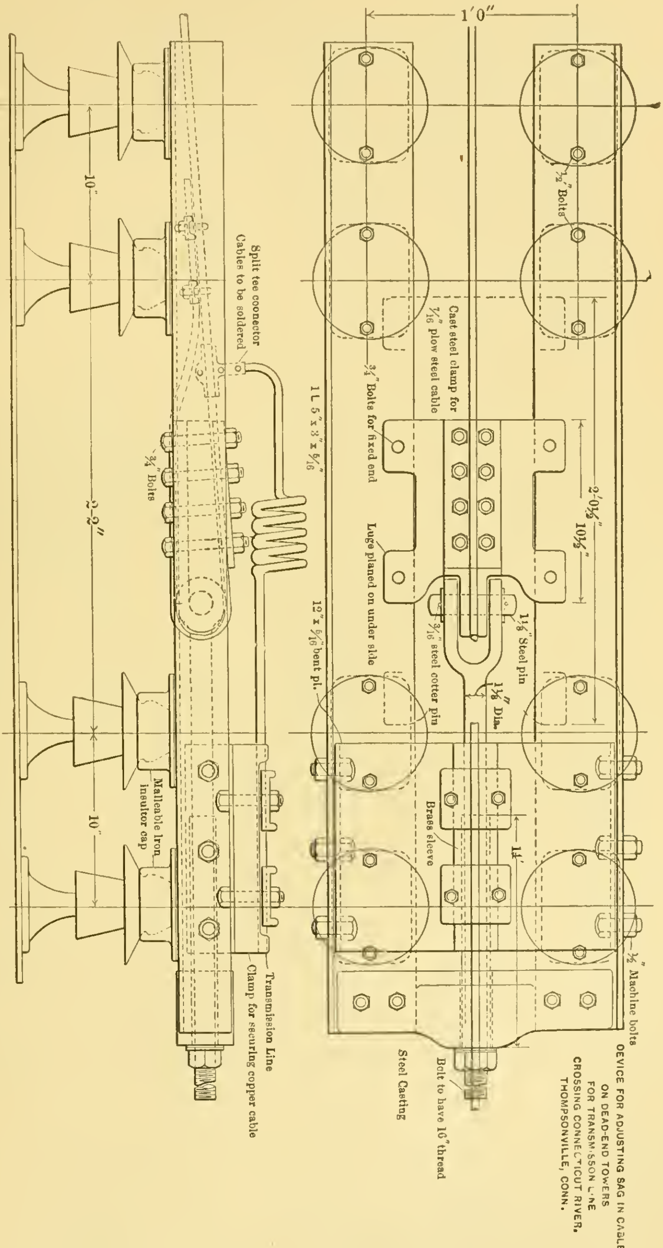


FIG. 9.

Mr. Harte. grounded angle-iron frame just beneath the line provides an automatic cut-off in case the line breaks in the span adjoining the protected span, Fig. 10, and Fig. 2, Plate XXXVIII.

A protection device should meet the following requirements:

1.—Complete protection of the lower line, including protection from a failure in an adjoining span with wires whipping into the protected section;
2.—A minimum of areas exposed to sleet or wind;
3.—Little increase over weight of normal line;
4.—Simplicity of design;
5.—Construction familiar to linemen;
6.—Few special parts;
7.—Low cost of installation and maintenance.

The foregoing types of protection assume that the power line is above, and the additional safety of such arrangement will usually justify a considerable expenditure to secure it. In some cases, however, it is practically impossible to go above with the transmission line. In such cases the method of the American Telephone and Telegraph Company, of practically enclosing the upper line in a sheath of $\frac{3}{8}$-in. strand network is the best, provided the design is such that it will prevent dangerous sagging of the cradle.

In any cradle design, the tendency of a broken wire to curl and therefore jump out of the cradle should be recognized; in Germany and Switzerland it is customary to compel transmission companies to make all lines crossing railroads pass through a regular tunnel of ironwork.

Where the line is on very narrow right of way, and where it carries trolley brackets, straight poles are essential, and it is often desirable to use selected stock in important highways, but in the majority of cases considerable crook can be allowed. Certainly, where chestnut is to be used, Mr. Coombs' requirement of only 1 in. of crook in 10 ft. of length is unnecessarily rigid.

The Western Lumberman's and the Idaho Cedarmen's Associations have defined commercially straight cedar poles as having a crook in one direction only, and a sweep not to exceed 1 in. in 6 ft. For chestnut, the American Telephone and Telegraph Company allows practically 1 in. sweep in $2\frac{1}{2}$ ft. of length for poles up to 40 ft. total length, and of 1 in. sweep in 3 ft. for poles more than 40 ft. in total length, the measurements to be made between the top and a point 6 ft. from the butt.

Chestnut from seed often grows very straight; stump-grown stock, which to-day forms a large proportion of the supply, almost invariably shows a sharp crook near the butt, due to the growth of the shoots, first out to clear each other and then straight upward. If this crook is large, it increases the cost of pole setting, but a diver-

Mr. Harte.

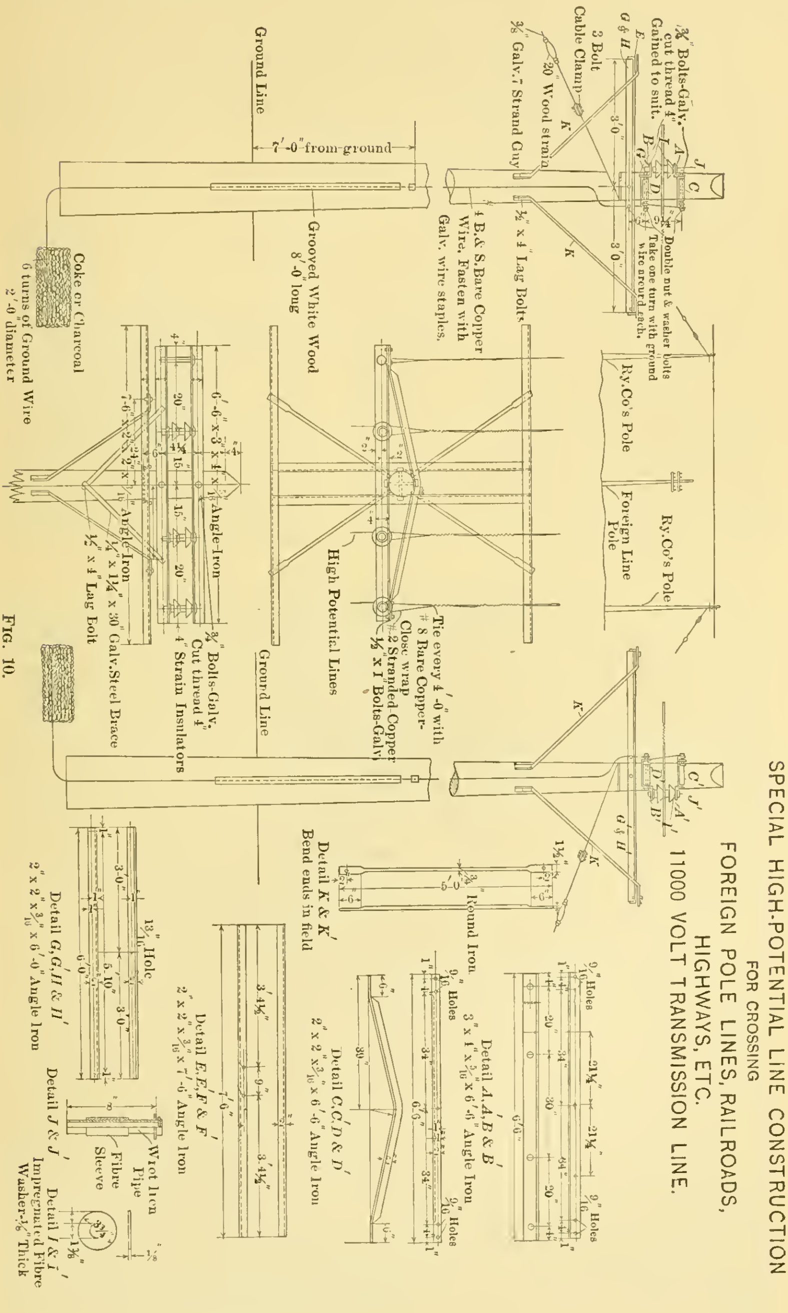


FIG. 10.

Mr. Harte. gence from the general axis of the pole of not more than 12 in. in the lower 6 ft. can be cared for without additional work.

In the speaker's judgment, the severity of the specification does not increase the line strength, and, as it materially increases the cost, it would seem that it might better be changed, to meet current practice and market limitations, to the following:

Cedar poles shall have but one crook, this in one way only, the sweep not to exceed 1 in. in 6 ft.

Chestnut poles shall have but one crook, this in one way only. The sweep shall not exceed the following limits between butt and top:

Pole length, in feet: 30—35—40—45—50—55—60—65—70.
Sweep, in inches: 9—10—11—11—11—12—13—14—15.

As far as the speaker is aware, there has as yet been no wreck of any magnitude on any of the electrified steam lines, and, until such a try-out, certain questions of design must remain unanswered.

It is not at all unlikely, however, that, at least for lines on which freight trains, with their capacity for trouble, are handled; the ultimate development in steam railroad electrification will be in the direction of independent overhead lines for each track or group of tracks.

With one live trolley, emergency movements can be made on adjoining tracks; with all overhead wires down, as may well be feared in a wreck under bridge construction, not only is the electrical equipment helpless, but a large additional burden of clearing away the material devolves on the wrecker.

Whatever the design, the speaker feels that the unit stresses allowed by Mr. Coombs are too high. With long and frequent trains, and particularly with high voltages, the failure of any part of the overhead system offers too great an opportunity for serious results to justify any close paring in the design.

Steam railroad electrification for some years to come will be undertaken only where there is in sight a very marked gain by the change, or where legislation compels it. The complications with which the simplest distribution system involves maintenance operations, and the awkward fact that for wrecking, and in "dead" sections, some self-contained motor must be used, weigh heavily with men familiar with steam-road operation, and offset many of the obvious advantages of electrification. It will rarely happen that a cost variation several times in excess of the difference between thoroughly dependable construction and "probably safe" construction will be of weight in influencing the decision, and in the few cases where it is a factor it is far better for the art that the work be deferred rather than incur an unjust discredit because of failure, either physical or in performance, as to expected maintenance and operation costs.

Whatever the unit stresses, the bridges, brackets, or poles should

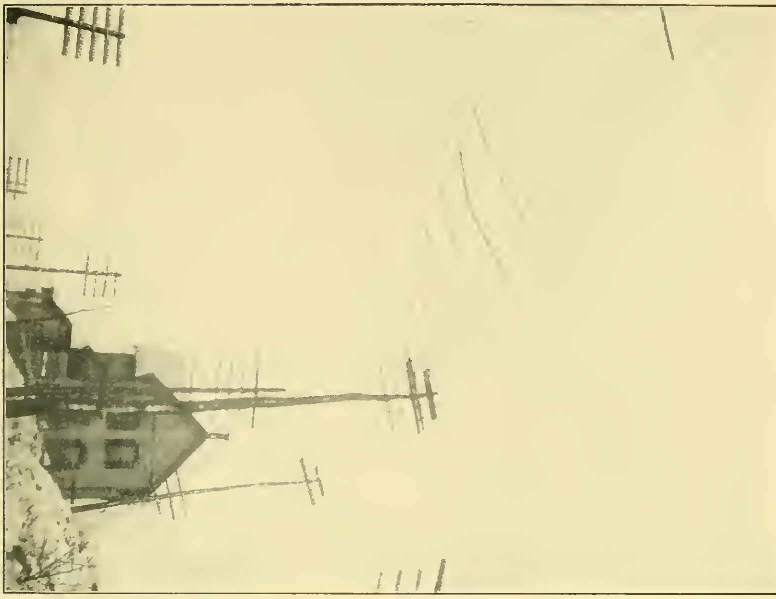


FIG. 1.—WOOD-BAR CRADLE UNDER 6 600-VOLT LIGHTING CIRCUIT.

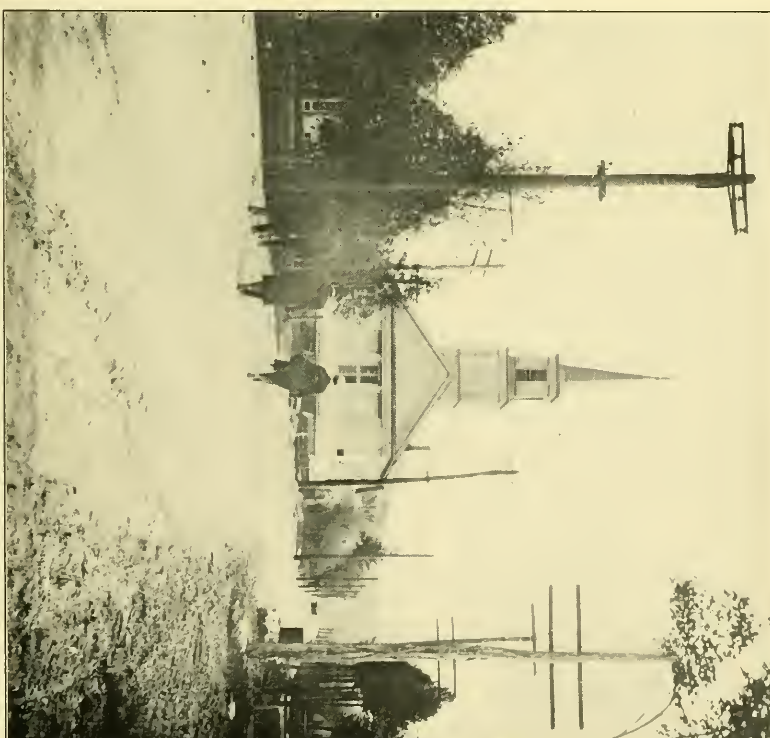


FIG. 2.—CATENARY CROSSING, WITH SINGLE CUT-OFF ARMS.

have a safety factor considerably in excess of that of the overhead system proper, and messengers, hangers, and trolley should mark a regularly descending scale, in order that any failure may be of the least extent possible. Mr. Harte.

While it is true that the ordinary trolley suspension gives a catenary curve, the general practice to-day is to apply the term "catenary construction" to systems supporting the trolley from one or more messengers. Following this practice, overhead systems would be classified as:

1.—Simple Suspension.—Trolley carried by hangers directly connected to span wire, bracket, or bridge;

2.—Catenary Suspension.—Trolley hung from one or more messenger cables in turn carried by the span wires, brackets, or bridges;

3.—Single Catenary.—Having but one main messenger (as in the Erie Railroad electrification);

4.—Multiple Catenary.—Having more than one main messenger (as in the New Haven electrification);

5.—Simple Catenary.—Trolley carried directly by main messenger; may be simple or multiple (as in the Erie or New Haven electrification);

6.—Compound Catenary.—Trolley carried by a secondary messenger system, in turn carried by the main messenger (as in the Blankenese-Ohlsdorf Railway).

The kind of conductors best adapted to the collection of the power is an open question. The two chief difficulties, with high speeds, are the chattering of the shoe, due to alternate hard and soft spots in the line, and the pressure variations, due to the great vertical range required of the pantograph.

The first problem may be solved either by floating or by fixing the trolley wire; for undoubtedly a perfectly flexible line or one perfectly rigid would give excellent results as far as it alone was concerned. Whether the shoe will not chatter on the rigid line, as a result of the irregular movements of the car, remains to be seen. It is interesting to note that Mr. W. S. Murray, Electrical Engineer of the New York, New Haven, and Hartford Railroad, in discussing his recent paper before the American Institute of Electrical Engineers, is quoted* as saying that, in his judgment, either the shoe or the line must be flexible.

The second problem is largely a function of overhead crossing limitations, and, to a large degree, is independent of the overhead construction; therefore it must be cared for in the design of the collector itself.

Both problems are of the field rather than of the office. Mr. Mayer†

* *Street Railway Journal*, January 18th, 1908, page 81.

† *Proceedings*, Am. Soc. C. E., for December, 1907.

Mr. Harte. has given a very elegant mathematical analysis of shoe pressure under certain conditions, but the discussion is based on the supposition that the car end of the collector traverses a path bearing a definite and regular relation to the conductor. As a matter of fact, however, this path is most irregular. Unevenness of track, as to grade and line, gauge variations of rail and wheel, side play in axle boxes, spring action, and movements in the car framing itself, all affect the shoe pressure entirely independently of the variations due to the collector mechanism and the character of the overhead system.

Mr. Coombs recites five objections to the double, as compared with the single, catenary. That the double catenary has greater first cost and greater mass overhead is true, although, by the time the single form has been properly secured by pull-offs, guys, and steady braces, there is a surprising amount of material in the air.

As to maintenance, however, the speaker doubts whether a single catenary is not at least as troublesome. A double catenary can stand severe punishment and still permit the movement of trains. On the other hand, the hangers of the single catenary are more out of the way, and therefore less likely to be injured.

Either type requires the tower car for repairs, but the double catenary has twice as many connections to make; on the other hand, its greater strength and rigidity undoubtedly reduce the troubles above the trolley.

That the double catenary offers greater obstruction to the view of the signals, the speaker cannot admit. If the signals are on bridges, they will be between the tracks, and a curve that would bring the overhead structure across the line of sight would also bring the pole, towers, or truss posts also into line. The difficulty relates to the secondary supports rather than to the type of suspension.

Mr. Coombs sums up the situation admirably. If anything remains to be said, it is this: In the present state of the art there is a great lack of, and need for, data resulting from practical tests of the various theories.

In closing, the speaker wishes to express his obligations to the many friends who have kindly assisted in the experiments, and have loaned illustrations for use in this discussion.

Mr. Osgood. FARLEY OSGOOD, ESQ. (by letter).—If the high-tension wires are of sufficient mechanical strength to have a factor of safety of 3, under correctly-assumed general conditions, it is very doubtful if a conductor will part in the span.

Up to crossings of 600 ft., it is not considered that the wires are likely to cross in high winds, even though spreaders are not used, as experience seems to indicate that the wires will swing from their normal positions about equally.

Protection, in the form of lightning rods, seems desirable at crossings where very high wooden towers are used, or on lower wooden crossings at points of high altitude. Mr. Osgood.

If steel towers are used at railroad crossings, the use of lightning rods is desirable, if the crossings are at such points in the line as are known to be affected by lightning disturbance.

The use of cradles, suspended from high-tension poles, under the high-tension wires, is not advocated by the writer, for any ordinary circumstances.

An ideal high-tension crossing would have the supporting towers of sufficient height to make it impossible for one of the transmission wires to touch the ground in case it should break in the crossing span, but this condition is usually impossible, from a rational standpoint, owing to the length of the section.

A second choice for crossings seems to favor a supporting tower at the edge of the railroad company's right of way, and another between the telegraph or signal wires and the outside rail, thus using four poles or towers to a crossing, making a short span on each side of the tracks over the telegraph or signal wires and the longer span over the tracks, it being assumed that the railroad company has wires to be protected on each side of its tracks.

These poles should be of sufficient height to prevent one of the high-tension conductors from touching the telegraph or signal wires, if it should break, so that, if a high-tension conductor should give way over the tracks, the signaling system would not be affected, although this might not always be true if any of the rails were used as part of the signaling circuit.

If railroad companies feel that screens or cradles should be placed under high-tension wires, let such devices be placed in the top arm positions on the poles carrying the telegraph or signal wires, as their purpose can then be accomplished and no unnecessary burden be placed on the more important high-tension towers.

A simple and inexpensive type of screen, if used as suggested, can be made up as follows: In the top gain of the pole on each side of the high-tension crossing, place a ten-pin cross-arm, somewhat longer than the standard cross-arm used on the line to be protected, and run in between these arms, ten No. 6 or No. 8 galvanized steel wires, strapped together, and grounded at each pole.

W. S. MURRAY, ESQ.* (by letter).—Mr. Coombs has handled this very interesting subject in an analytical and conservative manner. With reference to that part of his paper concerning the strength of the materials to be used for wires and supporting structures, the basis of his assumptions could not be better founded than on the records given in his several tables. With reference to the equations relating to the unit pressures per square foot of projected area, the writer is pleased to be Mr. Murray.

* Electrical Engineer, New York, New Haven and Hartford Railroad.

Mr. Murray. able to confirm these figures in actual practice, as the catenary wires and supporting structures in the New Haven electrification were worked out by an equation practically identical with the one suggested by Mr. Coombs; and it is of interest to note here that these wires and structures have passed through storms approximating quite closely those stated as maximum conditions upon which the equations are based.

Mr. Coombs' specifications of general requirements are to the point, and, in addition to those which are generally recognized as standard, he has made many original and valuable suggestions.

In connection with the general subdivision of superstructures, although the writer is not quite able to agree with Mr. Coombs that the upright signals when supported from four-track trusses are obscured from the engineer's view at a distance of 1 200 ft., it is unquestionably true that the general envelope produces a difficult foreground for the engineer, and naturally the cross-span or cantilever-bracket construction clears up this disadvantage to a considerable degree.

As recently stated in a paper before the American Institute of Electrical Engineers, the writer is not loathe to believe that even four-track main-line electrification will be effected by the use of cross-catenary spans interspersed at proper intervals with fabricated steel truss anchor bridges; but believes that the form of this construction will be guyed steel uprights supporting the cross-catenary span, with distances between bents of, say, not greater than 300 ft.; and, further, he believes that, in the future, the single catenary will receive more favorable consideration than the double catenary construction. It can be readily seen that the first cost of the former will be much less, and the flexible contact offered by the single catenary construction, due to the fact that the trolley is supported from a single messenger, with the messenger in turn supported from a flexible cross-catenary, gives it a great advantage.

Practice seems to demonstrate the fact that either the shoe or the trolley must be flexible. As a matter of fact, flexibility in both would be of great advantage, and it cannot be questioned that the cross-catenary span will offer more flexibility than either the cantilever or bridge-truss type of construction. At this point, particular attention is called to the fact that experimentation with the deflection of trolley wire supported from a messenger, which is in turn supported at rigid points, shows that in the middle of the span the deflection is as much as 400% greater than that in the immediate vicinity of the bridge or cantilever supporting the messenger wire, it being understood, of course, that equal upward pressures are applied in each instance. This illustrates the value of the flexible feature in the cross-catenary support. A point of much value in the cross-catenary construction should be emphasized, namely, that the cross-spans may be supported on strain insulators, thereby not only doubling the actual insulating value of the line, as measured under normal atmospheric conditions, but, in point of fact, many times increasing the insulating value due to the insulation being placed at the side, and thus out of the direct line of steam locomotive blasts, which have such a deleterious effect on insulation. Mr. Murray.

An argument that will be advanced against the use of the cross-catenary construction is that it is not as reliable as the cantilever or truss construction. The answer to this is that, in this form of construction, any factor of safety that may be used in other types can be selected; in fact, larger factors of safety can be chosen with less proportionate expense.

In conclusion, the writer agrees with Mr. Coombs in his summation, under five counts, concerning the undesirability of double catenaries. The root of all trouble with the alignment of catenary construction is the change of temperature. The fact that a low temperature means a tight wire and *vice versa* for a high temperature must be considered. The ideal condition of suspension would be a free-running suspended wire, tension being supplied at one or both ends to counteract the variations in its length due to temperature. It is very seldom that ideal conditions can be secured in the field, however, and the results are generally a combination of compromises and approximations. What one fails to accomplish with the contact wire may be accomplished by a properly devised shoe, of strong construction, flexible and light, the last-named element eliminating inertia, the arch enemy to the hard spots in the line, which, as Mr. Coombs has pointed out, are at the "hanger points." To-day is not the time for standardization, but for observation. The experiences and mistakes of to-day will be invaluable in comparison with theories.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

A NEW SUSPENSION FOR THE CONTACT WIRES OF ELECTRIC RAILWAYS USING SLIDING BOWS.

Discussion.*

By Messrs. R. D. Coombs and Charles Rufus Harte.

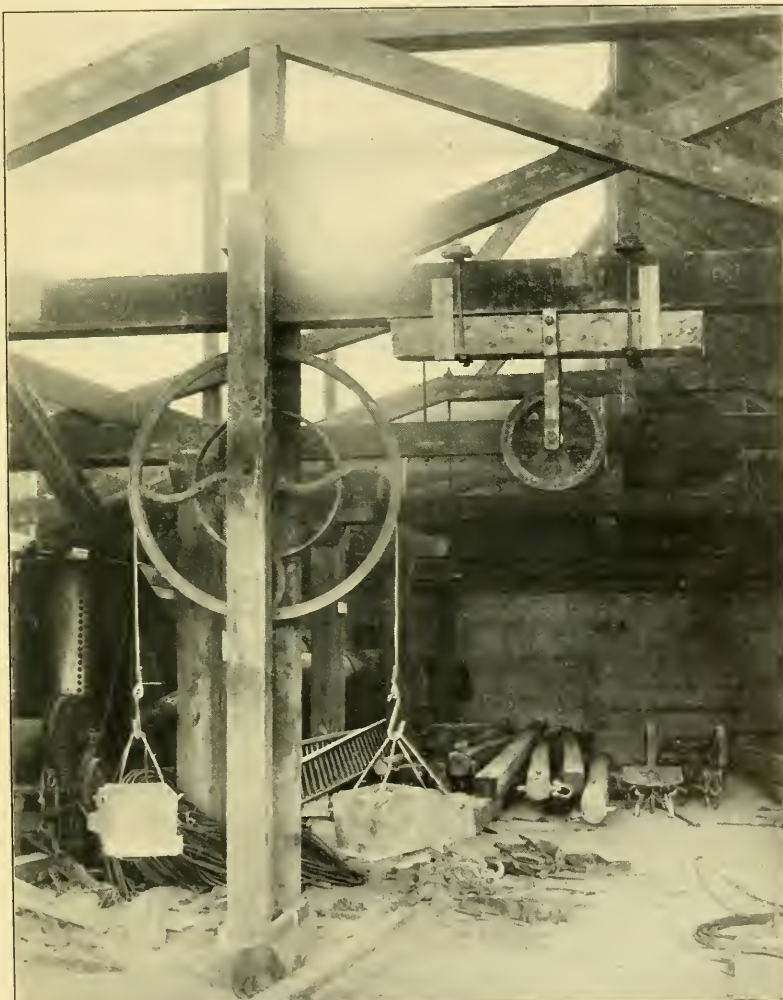
Mr. Coombs. R. D. Coombs, M. Am. Soc. C. E.—The speaker is of the opinion that interruptions in service caused by lateral displacement are improbable on either the 240-ft. spans with a sag of 2½ ft., as used by Mr. Mayer, or on 300-ft. catenary spans having a sag of 6 ft.

Based on the fact that the hot and cold sags in the catenary, and therefore in the trolley wires, are approximately equal for a total variation in temperature of 140°, the tension in the trolley wire is given by Mr. Mayer as about 26 000 lb. per sq. in. Assuming that the catenaries are erected with the normal tension at normal temperature, it would seem that the increased tension in the trolley wire should be merely that due to a rise or fall in temperature of half the total variation.

The speaker is not familiar with the details of the automatic adjustment of the trolley wire used in the Blankenese-Ohlsdorf line, or other foreign lines equipped with the secondary catenary, but thinks it should not be necessary to run the comparatively inflexible trolley wire over the adjusting pulleys, as this might be avoided by attaching a flexible wire which would permit the use of pulleys of moderate diameter.

The elastic limit, of from 40 000 to 45 000 lb., assumed for trolley wire having an ultimate strength of from 50 000 to 60 000 lb. per sq. in. seems to be rather high, and the maximum stress of about 26 000 lb.

* This discussion (of the paper by Joseph Mayer, M. Am. Soc. C. E., printed in *Proceedings* for December, 1907), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.



IMPROVISED "ATWOOD'S MACHINE" FOR TESTING THE STIFFNESS OF TROLLEY WIRE.

per sq. in. in the trolley wire under dead load, plus 6.0 lb. wind pressure, plus bending of the wire, does not give the contact wire in the saddle suspension the same factor of safety as that used in designing first-class railroad bridges. Mr. Coombs.

A parallel condition to that of many railroad bridges would be dead load, plus ice ½ in. thick, plus 8 lb. per sq. ft. for wind pressure, plus lateral bending; the total stress from which would exceed 26 000 lb.

Assuming that the single catenary and the trolley wire supported by it can be designed with suitable factors of safety, and constructed so as to give satisfactory operation, the extra expense of the messenger wire and hangers may be justified as a safeguard to prevent falling wires and the troubles incident to them.

Mr. Harte. CHARLES RUFUS HARTE, M. AM. SOC. C. E.—Mr. Mayer has developed a very interesting construction which it is to be hoped may have a practical trial in the near future. At the same time, it should be noted that the excessive stresses feared by Mr. Mayer do not always develop in the older forms of suspension. Undoubtedly, on long level tangents, with heavy anchoring, there would be heavy stresses at low temperatures, if the trolley had been well pulled up in warm weather, but, as a matter of fact, grade changes and curves offer relief, and trolley pulled to a tension of 2 200 lb. in summer apparently does not materially increase this stress in winter under usual conditions, owing to the yielding of supports. Where trolley is hung slack, however, the changes of length are chiefly taken up in the sag, and here an adjuster may be desirable; but, certainly in New England, such a device must be automatic or else receive constant attention in order to meet the rapid and large changes of temperature of that climate.

A system of counterweights offers the ideal method of securing uniform tension, and there is little difficulty in arranging bell-cranks or of splicing into the trolley a section of steel strand, if the trolley itself is too stiff to lead direct to the counterweights. It must be apparent, however, to anyone familiar with trolley wire, that Mr. Mayer's figures and practical conditions do not agree.

To determine roughly the flexibility of 0000 B. & S. gauge hard-drawn, grooved copper trolley wire, the speaker arranged a crude form of the "Atwood's machine," of physics, the trolley wire forming the cord, and the head sheaves of a dumb-waiter the wheel. (Plate XXXIX.) Balanced weights were hung from the wire, and then one side was loaded until motion occurred. No correction was made for the considerable friction of the wheels used.

With a pulley 55 in. in diameter at the root of the groove: 127 lb. on each side required 13 lb. additional on one side to move; 232 lb. on each side required 18 lb. additional on one side to move; and, 792 lb. on each side required 64 lb. additional on one side to move.

Mr. Harte. With a wheel 33 in. in diameter: 232 lb. on each side required 33 lb. additional on one side to move.

With a wheel 17 in. in diameter: 127 lb. on each side required 83 lb. additional on one side to move; and, 324 lb. on each side required 155 lb. additional on one side to move.

The Blankenese-Ohlsdorf trolley is described* as a grooved wire, having a gauge practically equivalent to 0000; Mr. Mayer gives the area as 100 sq. mm., which is 3½% less than the area of the wire used in the foregoing rough test.

For assistance in the tests, the speaker is greatly indebted to his assistant, Mr. John F. Trumbull, and to Messrs. C. W. Blakeslee and Sons, Contractors, of New Haven.

* *Street Railway Journal*, April 6, 1907.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
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PAPERS AND DISCUSSIONS.

SAFE STRESSES IN STEEL COLUMNS.

Discussion.*

By Messrs. Henry B. Seaman, Luzerne S. Cowles, Charles M. Emmons, Henry S. Prichard, Horace E. Horton, F. P. Shearwood, L. D. Rights, and A. W. Carpenter.

Henry B. Seaman, M. Am. Soc. C. E. (by letter).—It may be too early yet for a Special Committee to advise as to the proper column formula to be used in structural work, but Mr. Worcester's paper brings us one step nearer its appointment. Mr. Seaman.

To the writer's mind, there never has been sufficient reason for abandoning the Rankine formula. The basis of its formation is the provision that a column receives both direct strain and bending strain. The direct strain is readily provided for, and the effect of bending is found by experiment, the results of which are used in determining the coefficient of r^2. It would seem better to plot the results of these tests upon the basis of ultimate strength, rather than working strength, as it keeps the mind more directly on the actual data observed. The formula can then be modified for working strength, either by taking a certain proportion of the numerator as a factor, or by other modification, if preferred.

It should be remembered that details are designed upon an assumed value of $\frac{l}{d} = 12$, where l equals the length and d the least diameter of a solid rectangular column. In designing columns, therefore, a greater strain should not be permitted than that for which the details are designed, that is, for a less value of $\frac{l}{d}$ than 12. This serves, as Mr.

* This discussion (of the paper by J. R. Worcester, M. Am. Soc. C. E., printed in *Proceedings* for January, 1908), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Seaman. Worcester has expressed it, to truncate the formula for very short columns. Since the Rankine formula, however, provides for bending as well as compression, it conforms with the tests on long columns better than does the straight-line formula, and, for that reason, to the writer's mind, is the most valuable formula we have. There has never seemed to be any excuse for the adoption of a straight-line formula, except simplicity in plotting and ease in memorizing. Confessedly, it does not conform to tests on long columns, it is not applicable beyond certain restricted limits, and finally, since it involves r instead of r^2, it cannot be used as readily without the assistance of tables; yet tables will assist equally well with any formula.

A recent study of the tests mentioned by Mr. Worcester has led the writer to adopt the following formulas:

For Steel and Wrought Iron:

$$p = \frac{a}{1 + \frac{l^2}{8\,000\,r^2}}$$

For Cast Iron:

$$p = \frac{a}{1 + \frac{l^2}{1\,000\,r^2}}$$

Mr. Worcester very properly calls attention to the fact that the failure of a column occurs when it begins to cripple, while, with the tension member, if allowed time to rest, the material becomes even stronger because of the work of overstrain which it has received. This would enable us to permit a higher factor of safety upon tension members than upon compression members, were it not for the fact that a permanent elongation of a tension member would deform the structure to such an extent as to change the strains for which it was designed, and possibly cause failure on that account. It must also be remembered that the element of fatigue—and possibly that of momentary impact—need not be considered in the bending of the column, and therefore the extra material used, in order to prevent bending, is an additional factor of safety, which the tension member does not possess.

The recent tendency in structural design seems to be to increase the live loading by a given factor in order to derive an equivalent static strain, and then to design the parts for these static strains, rather than the old method of using a factor of safety to cover defects in material, increase, and extraordinary effects of loading, etc. If the live-load strains can be increased so as to cover all possible contingencies, and if a dead load can be assumed which will not be exceeded under any circumstances, it would seem safe to place the allowable strain at one-half or two-thirds of the elastic limit. It is on this basis that long-span bridges are designed; and, by the adoption of a formula in which this factor would vary with the various lengths of span, the same method of proportioning could be adopted for shorter spans. Future specifications will probably tend in the direction of some such method of design.

LUZERNE S. COWLES, ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Worcester need hardly offer an apology for continuing the agitation concerning compressive stresses to be allowed in designing structural steel-work. The writer, from the beginning of his career, and in fact during his college course, has been decidedly baffled by the numerous formulas for allowable safe compressive stresses, and had begun to believe that most so-called "rational" formulas were made up to assist the designer in making a comparatively safe guess. The question arises, however, as to whether all the commonly accepted formulas do really give the margin of safety that is desired and is assumed to exist. Mr. Cowles.

C. C. Schneider, Past-President, Am. Soc. C. E., has frequently called attention to the fact that the elastic limit, and not the ultimate strength, should be especially considered in deciding the real factor of safety. This gives, for tension, and supposedly for compression, a real factor of approximately 2, on the basis of 16 000 lb. per sq. in. for static loads. The writer agrees with this, particularly where compression is involved.

In the light of recently published data of experiments on full-sized compression members, it would seem that this real factor of safety of 2 had even been seriously encroached upon, leaving far too lean a margin of safety for structures where human life is at stake. When one considers the astounding results of Mr. Buchanan's tests,* where the fiber stress at crippling, even for so-called "short" columns, was below the accepted elastic limit, it seems to be high time to consider reducing the allowable unit stress for compression below that for tension, even though the modulus of elasticity and the elastic limit appear in the laboratory, and no doubt are, approximately the same for each.

Most railroad bridge specifications insist that no compression member shall have a length exceeding 100 times its least radius of gyration, except for bracing, where a ratio of 120 may be used. In other words, a main compression member in which the $\frac{l}{r}$ is 100, will carry safely 9 000 lb. per sq. in., whereas the use of a main member in which the $\frac{l}{r}$ is greater than 100 is disapproved. This is according to a standard straight-line formula, and it seems that the use of very "long" columns is not discouraged to the extent that it should be.

Is not then the really "rational" formula one which gives comparatively low results for the allowable fiber stress for the longest columns consistent with good design, and errs on the side of safety for the occasional exceptionally short strut? Mr. Worcester's proposed formula seems to fill these conditions, and while it may not be perfect in its present form, it is surely a step in the right direction, and furnishes a basis for a truly sensible formula. With his customary

*Engineering News, Vol. LVIII, pp. 685-695.

Mr. Cowles. modesty, the author of the proposed formula has failed to point out its commendable features. The writer suggests the following:

(a).—Reducing the allowable stress for "short" columns so as to give a reasonable factor of safety;

(b).—Discouraging the use of columns in which the $\frac{l}{r}$ is greater than, say, 90 to 100;

(c).—Placing the allowable stresses, for columns in which the ratios of $\frac{l}{r}$ lie between 30 and 90, at figures which are slightly below the average results, as shown by numerous tests.

Mr. Emmons. CHARLES M. EMMONS, M. AM. SOC. C. E. (by letter).—The writer is much interested in this paper. Mr. Worcester's plotting of the results of actual tests, reduced by a safe working factor, and also his plotting of the several column formulas, to the same scale, reveals very graphically the inconsistencies and the wide divergencies of these formulas.

The writer is not as fully impressed with the idea of a formula being self-limiting at the highest allowed value of $\frac{l}{r}$. In attempting to do that, the author's curve appears to be as inconsistent with the tests as would be a straight-line formula. The writer realizes that a formula should be of such form that, if the allowed value of $\frac{l}{r}$ be not fixed arbitrarily, it will yet be in no wise dangerous, for the reason that someone with more "nerve" than judgment, or through ignorance or other cause, will occasionally use such a formula as

$$\frac{12\,500}{1 + \frac{l^2}{36\,000\, r^2}}$$

to the limit. This danger was just lately brought to the writer's attention in a case where, for compression members, more than 10 ft. long, having a stress of about 4 000 lb., a prominent engineer used a single angle $2\frac{1}{2}$ by 2 by $\frac{1}{4}$ in. The $\frac{l}{r}$ is more than 300, and yet, according to the formula, it should carry the load safely.

The use of any formula which may be based on a series of observations, like those given by the author, should not, with confidence, be pushed very far beyond the limits of those observations. Such a formula, however, should take full advantage of what is indicated as safe by those observations.

Again, the formula proposed by the author, where one would be practically limited to $\frac{l}{r} = 112$, would be prohibitive, in many classes of work, especially for secondary members.

In view of these considerations, the writer would prefer the use of a parabola with values, say, $12\,000\left(1 - \frac{l^2}{(160r)^2}\right)$, with the limiting allowed value of $\frac{l}{r}$ prescribed. The formula is practically self-limiting at $\frac{l}{r} = 150$. It will be observed that this curve gives practically the same values as the circle up to $\frac{l}{r} = 80$, and from that point it follows the tests far better, taking advantage of what the tests indicate as safe, and yet in no case becoming dangerous. Mr. Emmons.

Henry S. Prichard, M. Am. Soc. C. E. (by letter).—The introductory paragraphs of this paper give the impression that the author attaches slight importance to theory in regard to columns. Would it not be well to discriminate somewhat in this regard? It is unfortunate that a single word, "Theory," is popularly used (with the sanction of the dictionaries) to designate "a body of the fundamental principles underlying any science or application of a science," and the radically different conception "a proposed explanation designated to account for any phenomena," no matter how visionary the assumptions, fallacious the argument, or foolish the conclusion. It is natural and proper that many of the proposed explanations of the behavior of columns should be held in light esteem, but it is highly desirable that engineers should understand and apply the principles of mechanics to the design of columns. Without such an understanding, the phenomena observed in practice and in the numerous compression tests are to a considerable extent a set of seemingly discordant facts. Mr. Prichard.

Referring to the fact that the practice of steel designers with regard to columns may well bear further consideration, the author states:

"The reason for this is that all 'rational' column formulas, based on the elastic properties of steel, are founded on considerations which are applicable only to ratios of length to radius of gyration far beyond those allowed in actual construction."

It is difficult to reconcile this statement with the analyses and equations developed by Euler, Cain, Fidler, Marston, J. B. Johnson, Moncrieff, and others who have determined important facts regarding short as well as long columns by reasoning based on the elastic properties of steel and iron. The names of Tredgold, Gordon, and Rankine have purposely been omitted from this list for the reason that the formula which they, by successive steps, developed is based on the erroneous application to columns of the principle, strictly applicable to beams, that the greatest possible deflections within the elastic limit, of beams similar as to section, manner of loading, and end conditions, are proportional to the squares of their lengths multiplied by the elas-

Mr. Prichard. tic limit. In columns, under analogous conditions, the greatest deflections within the elastic limit are proportional to the squares of their lengths, multiplied, not by the elastic limit, but by the differences between the elastic limit and the mean compressive stresses in the various columns.

Euler's formula applies only to long columns, but he should be included among those who, by analysis, have determined facts as to short columns, for the reason that his formula carries with it the necessary consequence that, under ideal conditions, columns which are too short to have Euler's formula apply to them will have a uniform distribution of stress and no deflection, up to the elastic limit, a condition which is sometimes closely approached in laboratory tests. In practice, of course, the conditions may be far from ideal, but Fidler, Marston, J. B. Johnson, Moncrieff, and others have made valuable and instructive analyses of the effect, within the elastic limit, of departures from ideal conditions.

The author objects to the practice of using the elastic limit as the criterion of strength without regard to the ultimate. When rest occurs between the periods of straining beyond the yield point, the elastic limit, which at first is somewhat below the yield point, can be raised somewhat above it, thus making a permanent gain in strength, the usefulness of which is greatly lessened by the fact that when structural steel of the usual quality is overstrained it becomes very ductile.

When only a small portion of a steel member is overstrained, and the conditions are such that a very small flow of the ductile metal brings relief, the overstrained steel, by regaining its elasticity during a rest, accommodates itself to the conditions with comparatively slight distortion. Thus ductility, combined with the recuperative powers of the steel, may be useful in adjusting the length and shape of members and details, and in raising the strength of pins, etc., but if the stress over the entire cross-section of a member is even slightly greater than the yield point, and there is no other direct path for it to follow, the member, if in compression, will buckle, unless it is very short and stiff, and, if in tension, will elongate so much that it will not only be irreparably injured, but will cause ruinous distortion in the remainder of the structure, and possibly the failure of some adjacent compression member, to the supposed weakness of which the disaster may be erroneously attributed.

Between ruinous distortion and collapse there is a great difference: Ruinous distortion means the loss of the structure, while collapse may, in addition, cause great damage and loss of life. The possession of strength in excess of the yield point, even though it be but temporary, is, therefore, of some value, and a somewhat higher unit stress could be allowed in members which possess it than in those which do not.

It should be remembered, however, that even a tension member, from some action which starts from a nick, a flaw, a jagged edge, a thread, or a rivet hole, or in some detail from no distinctive point, may break without marked elongation, especially under shock. Mr. Prichard.

There is no analytical method by which strength within the elastic limit can be equated with strength beyond it, but, other things being equal, would not an advantage of 4 000 lb. per sq. in. fully offset the absence of any strength there may be in a highly ductile metal beyond that limit? To assist in considering this question, the elongations beyond the yield point during a test of a fairly typical eye-bar are submitted in Table 1.

TABLE 1.—Elongation of a Typical Steel Eye-bar, Measured in a Length of $262\frac{1}{8}$ In. from Center to Center of Pins.

Load per square inch, in pounds.	Elongations:		Load per square inch, in pounds.	Elongations:	
	Inches.	Percentage.		Inches.	Percentage.
35 000	0.55	0.02	45 000	6.92	2.64
36 000	2.07	0.79	50 000	9.75	3.72
37 000	2.75	1.05	55 000	13.60	5.17
38 000	3.77	1.44	60 000	20.63	7.84
39 000	4.21	1.60	64 410	38.25	14.6
40 000	4.56	1.74	56 710	40.65	15.5

A strain of 4 500 lb. per sq. in. in excess of the yield point, with a percentage of elongation equal to the percentage in the bar cited, if it occurred in the diagonal eye-bars of a bridge panel 25 ft. long by 25 ft. high, would cause a distortion of the bridge, in one panel length, of $9\frac{1}{2}$ in. This would be a severe test of the floor system and top chords.

The relation of the yield points in compression to those in tension was well shown by a set of comparative tests by the late Charles A. Marshall,* M. Am. Soc. C. E., a synopsis of which is given in Table 2.

In Table 2 the strength of the steel increases, in a general way, as the size and thickness of the sections are reduced. A similar variation in the strength of wrought iron was shown, by tests made by the United States Board on Testing Iron and Steel,† to be due to reduction in rolling. In most cases, the results for compression are each an average of two tests, and for tension, of three or four tests. The average of the yield points given in Table 2 for compression is 1 432 lb. per sq. in. greater than for the corresponding results for tension. The results of these tests cannot be applied directly to tension in eye-bars. The fact that eye-bars are annealed puts them in a different class from material as it comes from the rolls, as the steel is softened, some of the good effects of rolling are taken away, and the proportion of yield

* *Transactions*, Am. Soc. C. E., Vol. XVII, p. 68.

† Vol. I, 1881, pp. 35-45.

Mr. Prichard. point to ultimate is lowered, especially if the bars are cooled slowly. Except in rare cases, steel as it comes from the rolls will have a yield point in tension exceeding 55% of its ultimate strength, while the average of all the tests (some 570 or more) of full-sized eye-bars, made during the last few years at the Ambridge plant of the American Bridge Company, gives a yield point equal to 52½% of the ultimate strength of the full-sized bar, with variations above and below this percentage. It is not wise to count on a yield point of more than 50% of the ultimate strength of the bar.

TABLE 2.—COMPARATIVE TESTS, IN TENSION AND COMPRESSION.

All from the same blow of Bessemer steel as it came from the rolls.

Size and shape of test piece.	Yield Point.			Ultimate strength in tension, in pounds per square inch.
	Compression, in pounds per square inch.	Tension: In pounds per square inch.	Tension: Percentage of ultimate.	
4 by ½ in.	Not given.	53 800	75.5	71 255
$\frac{8}{15}$ in. square	49 055	47 815	68.8	69 390
3 by ⅜ in.	Not given.	47 363	68.9	68 657
¾ in. round	47 300	46 090	66.8	68 995
3 by ½ in.	Not given.	44 417	65.8	67 527
¾ in. square	43 845	44 273	64.7	68 427
1 in. round	46 020	44 202	65.0	67 970
1 in. square	Not given.	43 560	63.6	68 510
4 by ¾ in.	" "	41 527	62.1	66 917
3 by ¾ in.	" "	41 447	61.9	66 987
4 by 1 in.	" "	41 415	62.4	66 230
1¼ in. square	42 300	41 060	60.4	67 973
1¼ in. round	43 460	40 747	60.8	67 040
1½ in. "	41 290	40 275	60.7	66 363
1¾ in. "	42 075	40 017	60.3	66 333
3 by 1 in.	Not given.	39 397	59.1	66 700
1½ in. square	42 740	39 317	58.8	66 833
3 by 2 in.	Not given.	39 302	59.1	66 537
3 by 1¼ in.	" "	38 482	57.7	66 640
2¼ in. square	39 940	38 310	....	Not given.
2 in. round	38 830	38 207	58.2	65 663
1¾ in. square	40 630	38 193	57.5	66 400
3 by 1½ in	Not given.	37 820	57.0	66 342
4 by 1½ in.	" "	37 580	....	Not given.
2¼ in. round.	" "	37 000	56.5	65 460
4 by 1¼ in.	" "	36 680	....	Not given.
2½ in. round	36 840	36 100	....	" "
3 by 1¾ in.	Not given.	35 917	54.6	65 762

It appears from the foregoing that the higher yield point in compression, of steel as it comes from the rolls, as compared with the yield point of annealed eye-bars, would about offset the advantage which the latter possesses of some temporary strength in excess of the yield point, even when the same ultimate tensile strength is specified for the eye-bars, as determined by full-sized tests, and steel for compression members, as determined by specimen tests. A comparison between the strength of steel in compression and the tensile strength of built steel members is a different matter, and introduces a somewhat different set of questions. Few engineers, however, unless they are opposed to pin connections, would permit a higher unit stress for the net area of a built tension member than for an eye-bar. Mr. Prichard.

With the exception of columns made of pipes and single angles, practically all columns are built of sections and plates. The rivets are usually assumed to fill the holes and take the place, as far as compression is concerned, of the sections they replace. As a matter of fact, they do not completely fill the holes, and it is very doubtful whether they wholly make good the loss in section. They are seldom placed closer than an average of 4 in., and probably they are more than half as effective as the metal they replace. On this basis, the allowed stress per unit of gross section of column area would be about seven-eighths of the allowed stress per unit of net section in tension; that is, if 16 000 lb. per sq. in. is allowed in tension, 14 000 lb. would be a corresponding limit for compression. There are other considerations, however, chief of which is the weakening influence of slenderness in either the column as a whole, or in its details.

Notwithstanding the large number of tests that have been made in the endeavor to determine the influence of slenderness (Moncrieff, in his paper on "The Practical Column," cites more than 1 000),* the practice of engineers in this regard, as shown by the author, is very diverse; from which it would appear that the lessons taught by the tests are not very definite, or that they have not been generally understood.

A knowledge of the principles involved is of great importance, both as a guide to the making of useful tests and as a key to understanding the phenomena observed. The theory of columns has been partially developed by correct analysis, but it has frequently been elaborated so much that the essential facts have been buried under what Trautwine called "heaps of mathematical rubbish." It may be well, therefore, to present a concise analysis of the influence of length and eccentricity on the strength and stiffness of columns.

Consider a column with frictionless hinged ends, of length, l, and radius of gyration, r, with constant cross-sectional area, A, subjected to a longitudinal load, of intensity, p, acting with an intentional eccentricity, e, and an accidental eccentricity, e'.

In consequence of the eccentricity, there will be a primary intentional bending moment, $p\ A\ e$, a primary accidental bending moment, $p\ A\ e'$, and a secondary bending moment, $p\ A\ \Delta$; Δ being the deflection. The value of Δ can be obtained from the well-known equation:

$$\Delta = \frac{\text{moment}\ l^2}{C\ E\ A\ r^2} \qquad \text{(1)}$$

* *Transactions*, Am. Soc. C. E., Vol. XLV, p. 334.

Mr. Prichard. in which E is the modulus of elasticity and C a factor which varies with the shape of the moment diagram.

The moment diagram can be divided into two parts, the diagram of primary moments and the diagram of secondary moments. For the determination of the deflection due to the secondary moments, the value of C will vary between limits of which the upper is π^2 and the lower depends on the form of the primary moment diagram: if it is a rectangle, the lower limit will be 9.6, while, if it is in the shape of a bow, the lower limit will approach very close to the upper limit, π^2 = about 9.87. The limits are so narrow that it can be taken as π^2 without serious error.

For the determination of the deflection directly due to the primary moment, the value of C will vary according to the conditions, but, for convenience, it may be designated $\frac{\pi^2}{z}$. (If the cause of the primary bending moment is the eccentric application of the load, z will equal 1.234, and C will equal 8; but, if the cause is a bow-shaped bend in the axis of the column, z will be approximately equal to unity. In the applications made subsequently in this discussion, z is taken as equal to 1.234, which, in some cases, is a trifle high. The resulting stresses and deflections, therefore, are a trifle high, especially for the higher ratios of l to r.)

Substituting the primary intentional, primary accidental, and secondary moments in Equation 1 gives

$$\Delta = \frac{z\, p\, A\, l^2\, (e + e')}{A\, \pi^2\, E\, r^2} + \frac{p\, A\, \Delta\, l^2}{A\, \pi^2\, E\, r^2} \qquad (2)$$

To simplify the development, let

$$q = \frac{\pi^2\, E\, r^2}{l^2} \qquad (3)$$

This is Euler's formula, and, as it facilitates the application of the final equations to have the values of q, which may be termed a modulus of rupture, determined for various values of $\frac{l}{r}$ and tabulated, Table 3 is submitted.

Substituting q for its value in Equation 2, and reducing, gives

$$\Delta = \frac{p\, z\, (e + e')}{q - p} \qquad (4)$$

Hence the secondary moment is

$$p\, A\, \Delta = p\, A\, (e + e')\, \frac{p}{q - p}\, z \qquad (5)$$

Let V = the distance from the neutral axis to the extreme fiber on the concave side of the column.

The stress from bending, in the extreme fiber on the concave side of the column, is $\frac{\text{moment} \times V}{A\, r^2}$. Hence, if f = the combined stress in the extreme fiber on the concave side,

$$f = p + \left(\frac{p\, e\, V}{r^2} + \frac{p\, e'\, V}{r^2}\right)\left(1 + \frac{p}{q - p}\, z\right) \quad \ldots\ldots\ldots \quad (6)$$ Mr. Prichard.

In investigating physical laws, the work of the study and the laboratory should be complementary—a proposition generally conceded, but practiced too little in investigating the mechanics of structures.

TABLE 3.—VALUES FOR MODULUS OF BUCKLING.*

Values of the modulus of buckling. $q = \dfrac{\pi^2\, E}{\left(\dfrac{l}{r}\right)^2}$

$E = 29\,000\,000$ lb. per sq. in.

$l =$ length, in inches. $r =$ radius of gyration, in inches.

$\frac{l}{r}$	Values of q.	$\frac{l}{r}$	Values of q.	$\frac{l}{r}$	Values of q.
2	71 555 000	82	42 567	162	10 906
4	17 889 000	84	40 564	164	10 642
6	7 950 500	86	38 700	166	10 387
8	4 472 200	88	36 960	168	10 141
10	2 862 200	90	35 336	170	9 904
12	1 987 600	92	33 816	172	9 675
14	1 460 300	94	32 393	174	9 454
16	1 118 000	96	31 057	176	9 240
18	883 390	98	29 802	178	9 034
20	715 550	100	28 622	180	8 834
22	591 360	102	27 511	182	8 641
24	496 910	104	26 463	184	8 454
26	423 400	106	25 473	186	8 273
28	365 080	108	24 549	188	8 098
30	318 020	110	23 655	190	7 929
32	279 510	112	22 817	192	7 764
34	247 590	114	22 024	194	7 605
36	220 850	116	21 271	196	7 451
38	198 210	118	20 556	198	7 301
40	178 890	120	19 876	200	7 155
42	162 260	122	19 230	202	7 015
44	147 840	124	18 615	204	6 878
46	135 260	126	18 029	206	6 745
48	124 230	128	17 469	208	6 616
50	114 490	130	16 936	210	6 490
52	105 850	132	16 427	212	6 368
54	98 155	134	15 940	214	6 250
56	91 269	136	15 475	216	6 135
58	85 083	138	15 029	218	6 023
60	79 506	140	14 603	220	5 914
62	74 459	142	14 195	222	5 808
64	69 878	144	13 803	224	5 704
66	65 707	146	13 427	226	5 604
68	61 899	148	13 067	228	5 506
70	58 412	150	12 721	230	5 411
72	55 212	152	12 388	232	5 318
74	52 268	154	12 068	234	5 227
76	49 553	156	11 761	236	5 139
78	47 045	158	11 465	238	5 053
80	42 722	160	11 180	240	4 969

* From *Proceedings*, Engineers' Society of Western Pennsylvania, July, 1907, p. 341.

Mr. Prichard. To substantiate Equation 4, which differs in form, but is actually similar to one given by Moncrieff,* a comparison is submitted between the deflections calculated therefrom and the actual deflections in tests made at the Watertown Arsenal† of four wrought-iron columns; two consisting of two 8-in. channels and one 12-in. cover-plate; and two of two 10-in. channels and one 13-in. plate. In both cases the columns were latticed on the side opposite to the plate, the length was 74 radii of gyration, the load was applied by three ½-in. pins at right angles to the webs of the channels, and placed in the center of gravity of the channels. The modulus of elasticity was assumed as 27 000 000 lb., and the value of q correspondingly determined as 48 600 lb.

TABLE 4.

Load, in pounds.	Test 1632: e = 1.62 in. A = 17.57 sq. in. Deflections, in inches.		Test 1633: e = 1.64 in. A = 17.72 sq. in. Deflections, in inches.	
	Calculated.	Actual.	Calculated.	Actual.
10 000	0.02	0.00	0.02	0.00
20 000	0.05	0.01	0.05	0.01
50 000	0.12	0.10	0.12	0.09
100 000	0.27	0.22	0.27	0.21
200 000	0.61	0.54	0.61	0.51
250 000	0.83	0.77	0.83	0.74
280 000	0.97	0.95	0.97	0.91
300 000	1.08	1.15	1.08	1.20
306 000	......	Ult. Load.	......	.....
307 000	......	......	......	Ult. Load.
310 000	1.14	......	1.14	......

Load. in pounds.	Test 350: e = 1.7 in. A = 12.48 sq. in. Deflections, in inches.		Test 351. e = 1.3 in. A = ... Deflections, in inches.	
	Calculated.	Actual.	Calculated.	Actual.
10 000	0.04	0.0	0.03	0.0
20 000	0.07	0.03	0.06	0.2
50 000	0.19	0.14	0.17	0.12
100 000	0.41	0.37	0.38	0.32
150 000	0.69	0.67	0.64	0.58
180 000	0.89	0.90	0.83	0.82
200 000	1.03	1.20	0.98	1.05
202 700	......	Ult. Load.	......	......
208 200	.....	......	......	Ult. Load.
210 000	1.11	......	1.06	......

* *Transactions*, Am. Soc. C. E., Vol. XLV, p. 359.

† Reports for 1883, pp. 167 and 168; and 1884 pp. 54 and 55.

Considering the fact that, in making the comparisons in Table 4, no allowance was made for unintentional eccentricity or pin friction, the agreement is as close as could reasonably be expected. Moncrieff, in connection with his equation for deflection, previously referred to, gives a large number of comparisons between deflections obtained by applying his equation, and those observed in tests, which tend strongly to establish its substantial accuracy. Mr. Prichard.

The maximum compression in the extreme fiber of the column bearing the test number 1632, has been calculated for the ultimate load by Equation 6, which gives 36 600 lb. per sq. in.

The amount of the unintentional eccentricity will fluctuate greatly in practice, but, in devising rational formulas for use in designing, either by direct application or through empirical formulas founded on them, the greatest amount which is reasonably possible with ordinary care should be assumed. The amount assumed should cover inaccuracies in boring pin holes, the shift in the position of the axis from over-runs, and shortages in sectional area as compared with the area of the sizes specified, inequalities in the modulus of elasticity in different parts of the cross-section, curves or kinks in the axis, and potential curves or kinks in the axis from the relief of internal stresses. Owing to internal stresses produced by cold-straightening or otherwise, the metal is likely to be overstrained in spots before that in the main body of the column reaches the elastic limit. The internal stresses may be relieved to some extent by overstraining followed by a rest, but the column is likely to have a permanent deflection as a result thereof. From some causes, such as inaccuracies in pin holes, short columns are likely to have as much accidental eccentricity as long ones; while, from other causes, such as initial curvature of the axis, the probable limit of eccentricity will vary with the length. The following value for the factor, $\frac{e' V}{r^2}$, in Equation 6 is suggested:

$$\frac{e' V}{r^2} = \frac{1}{10} + \frac{l}{700\,r} \qquad (7)$$

For ordinary built columns with pin connections, in which the relation of V to r is about as 7 is to 5, the eccentricity corresponding to Equation 7 is:

$$e' = 0.07\,r + 0.01\,l \qquad (8)$$

For a column with a radius of gyration of 5 and a length of 500 in., the eccentricity given by Equation 8 is about $\frac{7}{8}$ in.

From Equations 6 and 7, a formula can be deduced which will give the load, p, per unit of column area, but such a formula is not submitted, for the reasons that it is so complicated and difficult of application that it is of no practical value, and the results which it gives for columns having a length of less than 100 radii of gyration can be

Mr. Prichard. approximated closely by short empirical methods. The method suggested is as follows:

For structural-steel columns with hinged ends, the stress per square inch, in pounds, in the most compressed fiber, from combined direct compression and intentional primary bending moment, shall not exceed

$$\frac{35\,000 - 1.5\dfrac{l^2}{r^2}}{\text{Factor required.}} \qquad (9)$$

For a factor of 2.5, the expression becomes

$$14\,000 - 0.6\frac{l^2}{r^2} \qquad (10)$$

From Equations 6 and 7 it is evident that the minimum value which can be assigned for the stress from the accidental bending moment is $0.1p$, from which it follows that a limitation of 35 000 lb. per sq. in. for combined direct compression and intentional primary bending moment corresponds to a limitation of 38 500 lb. per sq. in. for stresses from all sources. Hence, if proportioning by Equation 9, with a factor of one for maximum possible loads, gives results closely in accord with theory, as claimed, the stress per square inch in columns thus proportioned, as determined by Equations 6 and 7, should be close to 38 500 lb. How close they come to this amount is shown by Table 5.

TABLE 5.—MAXIMUM STRESSES PER SQUARE INCH, DETERMINED BY THEORY (EQUATIONS 6 AND 7) IN COLUMNS PROPORTIONED BY RULE (EQUATION 9, WITH FACTOR OF ONE), FOR VARIOUS LENGTHS AND INTENTIONAL ECCENTRICITIES.

$\frac{l}{r}$	THE INTENTIONAL PRIMARY BENDING STRESSES, IN TERMS OF THE LOAD, ARE GIVEN AT THE HEAD OF EACH COLUMN.			
	0	$0.1p$.	$0.5p$.	p.
	Pounds.	Pounds.	Pounds.	Pounds.
0	38 500	38 200	37 300	36 750
25	39 100	39 000	38 100	37 300
50	39 100	39 200	38 600	37 500
75	39 400	39 800	38 500	36 300
100	38 700	38 800	33 900	30 700

The agreement between theory and rule, indicated by Table 5, is close, except for long columns and great eccentricities, for which the rule requires a heavier column than theory.

Some opportunity for comparison between the theory and assumptions outlined on the one hand and experiments on the other is afforded by the tests of mild steel columns with pivoted ends made by Professor

Tetmajer.* Of the columns tested, the lengths of 27 did not exceed 100 radii, and they were loaded without intentional eccentricity. The ultimate load in all cases was greater than indicated by Equation 9, and less than required under ideal conditions for an elastic limit of 41 000 lb. and a modulus of elasticity of 29 000 000 lb. Table 6 is a comparison of the ultimate loads given by Equation 9 and the lowest of the ultimate loads shown by the tests. Mr. Prichard.

TABLE 6.

$\frac{l}{r}$	Equation 9. Pounds.	Tests. Pounds.	$\frac{l}{r}$	Equation 9. Pounds.	Tests. Pounds.
30	33 650	39 000 about.	70	27 600	29 000 about.
40	32 600	38 000 "	75	26 560	28 000 "
50	31 250	32 000 "	92	22 300	23 000 "

For columns of greater length than 100 radii of gyration, stiffness rather than strength is the governing consideration. For this reason, the loads allowed by Equation 9 are preferable to those allowed by the theory of column strength.

To show the relative stiffness of columns 100 radii and longer in length, when proportioned by Equation 9 with a factor of one, the deflections have been determined by Equations 4 and 8 for columns without intentional eccentricity, with a radius of gyration of one, and various lengths as shown in Table 7.

TABLE 7.

Length, in inches.	Deflection, in inches.	Ratio of deflection to length.	Length, in inches.	Deflection, in inches.	Ratio of deflection to length.
100	0.486	1:206	120	0.485	1:247
110	0.550	1:200	130	0.327	1:400

It will be noticed that the loads allowed by Equation 10 for columns up to a length of 100 radii of gyration are about one-sixth greater than those allowed by the author, but that there is a radical difference for longer columns. The objection among engineers to columns longer than 100 radii is largely sentimental. For the same load, a column with a length of 120, 130, or 140 radii, proportioned by Equation 10, in consequence of its greater sectional area, is stiffer and stronger than one of a length of 100 radii.

* *Transactions*, Am. Soc. C. E., Vol. XLV, p. 404.

Mr. Prichard. For derricks, poles, and other equipment used in building and erecting, much longer struts are used than in bridge work, and, when the loads are kept within rational limits, the flexibility of the struts, by permitting them to absorb impact, is an element of safety. In the ordinary affairs of life, long struts are used as a matter of course. The engineer who becomes alarmed at long struts in structures will bear his whole weight on a walking-stick, many times as flexible as steel, with a ratio of l to r of 200. Ample provision should be made in horizontal and inclined struts for the stresses from their own weight. The frequent neglect to make such provision in long struts has doubtless had something to do with the prejudice against them.

As regards columns, the greatest need for caution to-day is in proportioning short stiff ones, which, to an engineering public accustomed to gauge permissible unit stress by the ratio of length to radius of gyration, have an appearance of strength not borne out by their details, and, if their ends are square or fixed, they are subject to strains from imperfect butt joints, or to secondary stresses produced by the deformation of connecting floor-beams, etc. Such stresses are greater for short than for long columns, on account of their greater stiffness. In consequence of these facts, it is suggested that the average load from direct compression per square inch of cross-sectional area should not exceed 13 000 lb.

With the double requirement of Equation 10 and a 13 000-lb. limitation for direct compression, if the permissible loads are plotted to a scale for various ratios of l to r, the line indicating the maximum permissible loads will suddenly change its direction at a length somewhat less than $41r$, depending on the amount of the primary bending moment. This is a feature which the author seems to consider objectionable. It is entirely natural, however. Radically different conditions govern the strength of very short and very long columns, and the loci representing the loads under these radically different conditions will intersect sharply. If there is no intentional primary bending moment from eccentricity or transverse loading, 13 000 lb. per sq. in., unreduced, will govern for columns of shorter length than $41r$, and Equation 10 for columns of greater length, but if there is an intentional bending moment, both requirements should be applied to determine the governing one.

One of the assumptions from which Equation 10 was developed was that of frictionless hinged ends. When there is no primary bending moment, any friction on the pins, according to strict theory, will fix the ends; hence, it is not surprising that friction in pins is very potent in increasing the resistance of columns to direct load in carefully devised and conducted tests. Such friction, however, is a very poor reliance in practice, as it may be overcome by a little eccentricity or shock.

Mr. Prichard.

Under ideal conditions, a column with strictly fixed ends has the same strength as a column of half its length with the same cross-section and pivoted ends. In practice, however, the assumption of fixed ends is never wholly realized. The appearance of having the ends fixed is frequently deceptive, as compression members on opposite sides of a joint may deflect in opposite directions in such a way that the point of contrary flexure comes very near to the center of the joint, which condition is equivalent to pivoted ends. For this reason, no easement in the reduction for length, as given in Equation 10, is recommended in designing new columns with seemingly fixed ends. In determining the safe strength of a column in an existing structure, however, if it is evident that the ends are well fixed, it might be assumed that the column is as stiff as it would be if it were about three-fourths as long and had frictionless hinged ends. For such an assumption, Equation 9 would become:

Allowed compression per square inch in any fiber from combined direct compression and intentional primary bending moment for columns with ends fixed equals

$$\frac{35\,000 - 0.85\,\dfrac{l^2}{r^2}}{\text{Factor required.}}$$

In comparing this discussion with the writer's paper,* entitled "The Proportioning of Steel Railway Bridge Members," it will be noticed that the greatest compression now recommended is one-fifteenth less than in the paper referred to, in addition to which a limitation for direct compression to 13 000 lb. per sq. in. is now recommended. This change is the result of a further consideration of the subject, in the light of the Quebec Bridge disaster and the general discussion regarding columns which followed it, including the paper by Mr. Worcester. It should be stated, however, that it has always been the writer's practice in designing columns to give close attention to the details and to the make-up of the section, and to make a liberal reduction in the allowed unit stresses when the unsupported width of a plate exceeded 32 times its thickness. The following requirement as to the unsupported width of plates is suggested:

If the unsupported width, w, of any plate in a column is more than 32 times its thickness, t, the permissible stress, as given by Equation 10, shall be reduced by multiplying it by the following expression:

$$\text{Permissible stress, as given by Equation 10,} \times \frac{16\,000 - \dfrac{2\,w^2}{t^2}}{14\,000}$$

Mr. Horton.

Horace E. Horton, M. Am. Soc. C. E. (by letter).—Mr. Worcester's compilation of results of tests on full-sized wrought-metal compres-

*_Proceedings_, Engineers Society of Western Pennsylvania, July, 1907.

Mr. Horton. sion members is very interesting and instructive, and is opportune. There is an awakened interest in the subject at this time.

While the writer approves unhesitatingly Mr. Worcester's criticism on using excessive unit stress on members with short radii lengths, he knows no physical reason for limiting compression members to a length of 100 radii.

Mr. Worcester has chosen to make his platting of tests for steel on the basis of four-fifteenths of the ultimate strength. For obvious reasons, the writer uses the same. Mr. Worcester has used 12 000 lb. per sq. in. as his unit value in compression, and the writer naturally uses the same stress, with the reservation that the unit stress (tension) is $1\frac{1}{3} \times$ the compression, that is, 16 000, in this case.

The diagram, Fig. 2, gives all the tests of steel members shown by Mr. Worcester, also tests of six members* by J. A. L. Waddell, M. Am. Soc. C. E., and seven tests† by Mr. C. P. Buchanan, and, further, six tests by the Chicago Bridge and Iron Works, on 8 by 8 by $\frac{7}{16}$-in. angles.

On this diagram a straight line is drawn through the center of gravity of the group of tests, and is expressed by $11\,300 - 35\frac{l}{r}$, also the formula for loading, as indicated by C. L. Strobel, M. Am. Soc. C. E., in his paper, "Experiments Upon Z-Iron Columns,"‡ wherein was first laid down the necessity of "sawing off" the unit stress for short radii length, in this case to 8 000 lb. per sq. in., and also the first appearance of the straight-line formula, which was $10\,600 - 30\frac{l}{r}$.

The writer has platted Mr. Worcester's formula, based on 12 000, and, as a protest against the attempted "amputation" for radii lengths of more than 100, there is also platted the Hodgkinson-Gordon-Rankine formula, as given by Mr. Worcester, based on 16 000, with a lower divisor of 8 000. It will be noticed that the platted lines come tangent at 85 radii, and the two curves come somewhat above the center of gravity of the tests. However, as these values are high for short radii lengths, clearly indicating the necessity of "sawing off," the writer offers the Hodgkinson-Gordon-Rankine formula, based on 12 000, with a lower divisor of 12 000. When "sawed off" at two-thirds of the unit stress (that is, 10 666 lb. per sq. in. as the ultimate value in compression), and intersecting the curve at 40 radii length, it looks both sane and safe.

The curve produced as platted, clearly indicates what is intended, a value well below the average of the tests, and the value reduced so that one may believe it to be reliable as the radii lengths are extended, even to 200.

* The record of these tests appeared in *Engineering News*, January 16th, 1908.

† *Engineering News*, December 26th, 1907.

‡ *Transactions*, Am. Soc. C. E., Vol. XVIII, p. 103.

Mr. Horton.

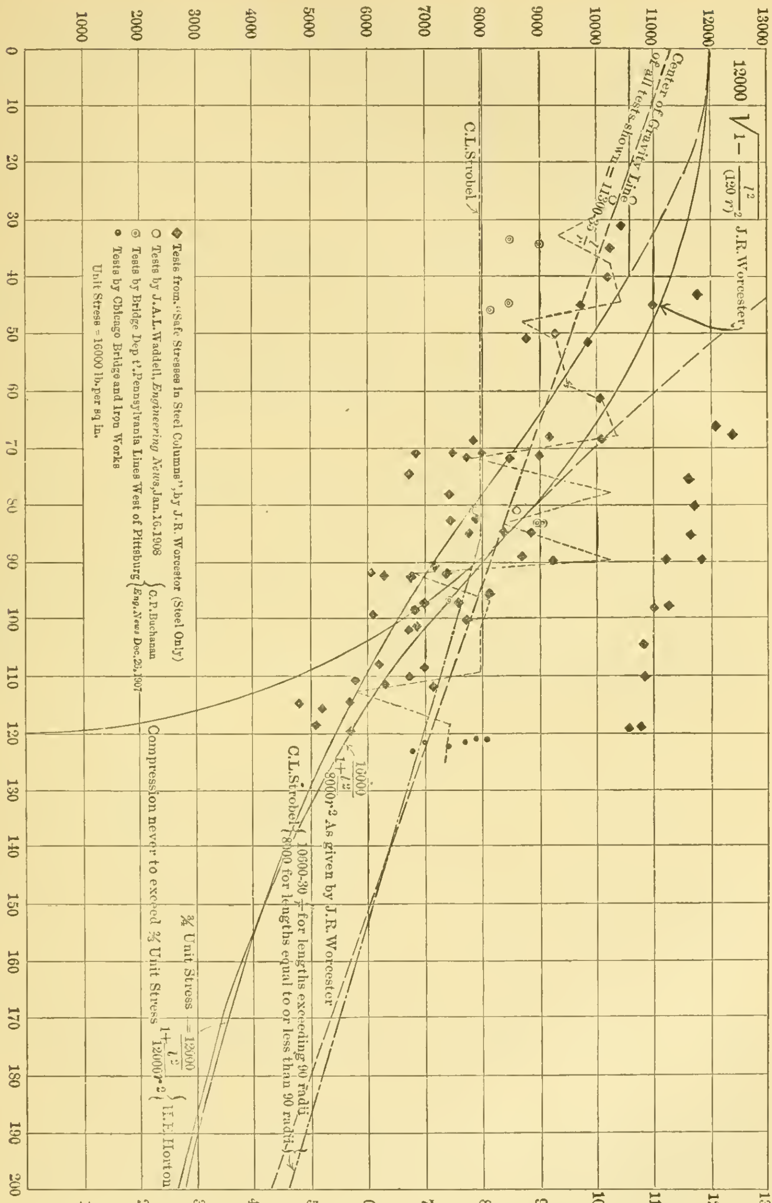


FIG. 2.

Mr. Horton. The limitation to 100 radii length of compression members can only be urged because of our want of knowledge, but practice and experience show that the greatest hazards are with short radii lengths, due to want of proper proportion and cohesion of parts, and the tendency toward using material which is too thin.

Unfortunately, engineering discussion as to the unit value of compression members has been almost entirely on formulas, and not on the physical column.

As to the physical compressive member, Mr. Buchanan gives a report, with full details of tests to destruction, of nineteen full-sized bridge members as built for actual use in structures—twelve of iron and seven of steel. The first noticeable thing is that radii length has no significance, in fact, members having a length of 83 radii were as strong as any tested, and much stronger than many of less than 40 radii, and members of 97 and even 120 radii were a good average in the whole group of tests.

The average ultimate strength of seven steel columns is 31 900 lb. per sq. in. The average crippling strength is 23 800 lb. per sq. in.; the average elastic limit is 19 700 lb. per sq. in.

The ultimate strength, crippling strength, and elastic limit, in the foregoing tests as reported, indicate a value of scarcely more than 50% of the value of the steel in tension. This is startling, with our knowledge of specifications permitting the use of steel in short radii lengths for approximately the same stress as in tension. Mr. Buchanan's tests are given in Table 8.

TABLE 8.

Average of Buchanan's tests.	$\frac{l}{r}$	Average of T. H. and J. B. Johnson's formula. Crippling load.	Actual crippling load.	Below estimate.
4 tests, Z-bar columns..........	96	28 537	21 700	6 837
15 tests, trough and channel columns	43.6	33 125	20 730	12 395

The four Z-bar columns lack, on an average from the computed crippling load, essentially half as much as did the fifteen trough and channel sections (the last with less than half the radii length), and yet they actually stood a greater load.

From the photographs of the members taken after the tests, it is seen that four Z-bar column struts yielded as a whole by flexure. The fifteen other members yielded by some order of wrinkling or failure in individual parts. It is further noticeable that the sections of the Z-bar columns were much thicker, relatively. It is clearly apparent that compressive members which fail by wrinkling fail at less load per unit than those which fail by flexure.

In the photographs of the material, after the tests by Mr. Waddell referred to above (all of the same cross-section), it is noticeable that members 81 radii long failed by flexure while those 27 radii long failed by wrinkling. From the Buchanan tests there is abundant evidence to conclude that the best results are obtained when the member yields by flexure. From Mr. Waddell's tests there is evidence that, for the best results at 27 radii, the material must needs be thicker than for 81 radii. Here is a suggestion, to be enlarged on later. Mr. Horton.

While the radius of gyration has use, as indicating the value of a strut, there is much to show that there are many other conditions of as great importance as the radii length.

The radius of gyration will modify and hold in check any disposition to use material of undue thickness, but the radius of gyration has to be held in check unless too thin material be used. The radius of gyration of a transverse element of the column may be used as such a check.

The composite nature of the compression member directly reduces its unit value, as compared with tension, a very material amount. This is directly traceable to the possible rivet efficiency connecting parts, and it is undoubtedly a fact that rivets are driven much too far apart. Two or three times as many rivets would surely give better results. Rivet connections between multiple plates, or plates and angles, forming a compression member, to reduce the tendency to wrinkling, are clearly different from tension connections, and the efficiency has to be considered locally.

With material half as thick as the rivet diameter, an efficiency of 50% may be obtained by pitching the rivets at 2.3 diameters; but, with material the thickness of the rivet diameter, and rivets pitched at 2 diameters, an efficiency of connection of 30% is all that is possible. In practice, rivets are generally driven with three times as much pitch as here indicated, and the assertion may be made that the efficiency of the connection of parts by rivets scarcely exceeds 12 per cent.

The cross-section of the compression member is unquestionably of great significance. The proportions of the material in the flanges and its width and thickness undoubtedly have paramount importance.

The compression member, with ever-increasing tendency in the evolution of design, has developed with one or more open sides on which lattice bars are used.

The proportions of such lattice bars, their connections to the columns, and their relation to a force acting through the compression member form a very material and important element, second to none in the design. At the present time, there are in the technical press many letters from correspondents, with elaborate formulas in which E represents the modulus of elasticity, and e the eccentricity. As e, eccentricity, is arbitrarily assumed, the writer prefers to assume a percentage of the compression through the column, and call it shear.

Mr. Horton. The difference between the eccentricity discussed and the shear outlined is as follows: Eccentricity is an assumption without reference to the magnitude or amount of the force acting on the member, while the shear is a direct percentage of the force acting on the member. One leads to the discussion of how accurate the workmanship of the column may be, or is. The other asserts the fact that there must be some relation between the force acting through a compression member and its disposition to "side-step." This uncertainty is not caused by faulty workmanship, but comes from a want of research and knowledge.

In all the years past the whole discussion and the specifications for compression members have absolutely ignored both shear and eccentricity as items to consider, except in what has appeared within a very short period, and there is no evidence that our workmanship has especially deteriorated in the immediate past, but there is reason to hope that our knowledge of design may be enlarged.

Figs. 3 to 15 are given in order to indicate to the eye the relation of various sections expressed by the radius of gyration; each section has the same cross-section, namely, 12 sq. in.

Fig. 3 is a solid, 3.46 in. on a side, radius of gyration = 1.

Fig. 4 is a hollow square, 5 in. on a side, metal $\frac{5}{8}$ in. thick, radius of gyration = 2.

Fig. 5 is a hollow square, $7\frac{3}{4}$ in. on a side, metal $\frac{13}{32}$ in. thick, radius of gyration = 3.

Fig. 6 is a hollow square, $10\frac{1}{2}$ in. on a side, metal $\frac{19}{64}$ in. thick, radius of gyration = 4.

Fig. 7 is a hollow square, $12\frac{3}{4}$ in. on a side, metal $\frac{15}{64}$ in. thick, radius of gyration = 5.

There are changes in the radius of gyration of from 1 to 5, with the same cross-section, with a diminishing thickness of the material, and an increasing unit value of the material by all compression formulas.

Figs. 8 to 15, inclusive, are interesting as indicating 12 sq. in. of section, in quite familiar shapes, with a radius of gyration of 5.

Appended to compression formulas it is quite usual to find a limitation of thickness to width, of 1 to 30, and Figs. 8 to 15 can only be objected to on this limitation, and not as to the radius of gyration.

According to Mr. Worcester's curve, as platted for working loads on columns 10 ft. long, Figs. 7 to 15, inclusive, may be worked for 11 800 lb. per sq. in.; Fig. 6 for 11 700 lb.; Fig. 5 for 11 300 lb.; Fig. 4 for 10 400 lb. per sq. in.; and Fig. 3 for zero. With this conclusion the writer does not agree. Figs. 4 or 5, undoubtedly, will carry the largest load of any of the sections, 3 to 15, inclusive, at 10 ft. long, while Fig. 3 will undoubtedly be a close second.

Mr. Horton.

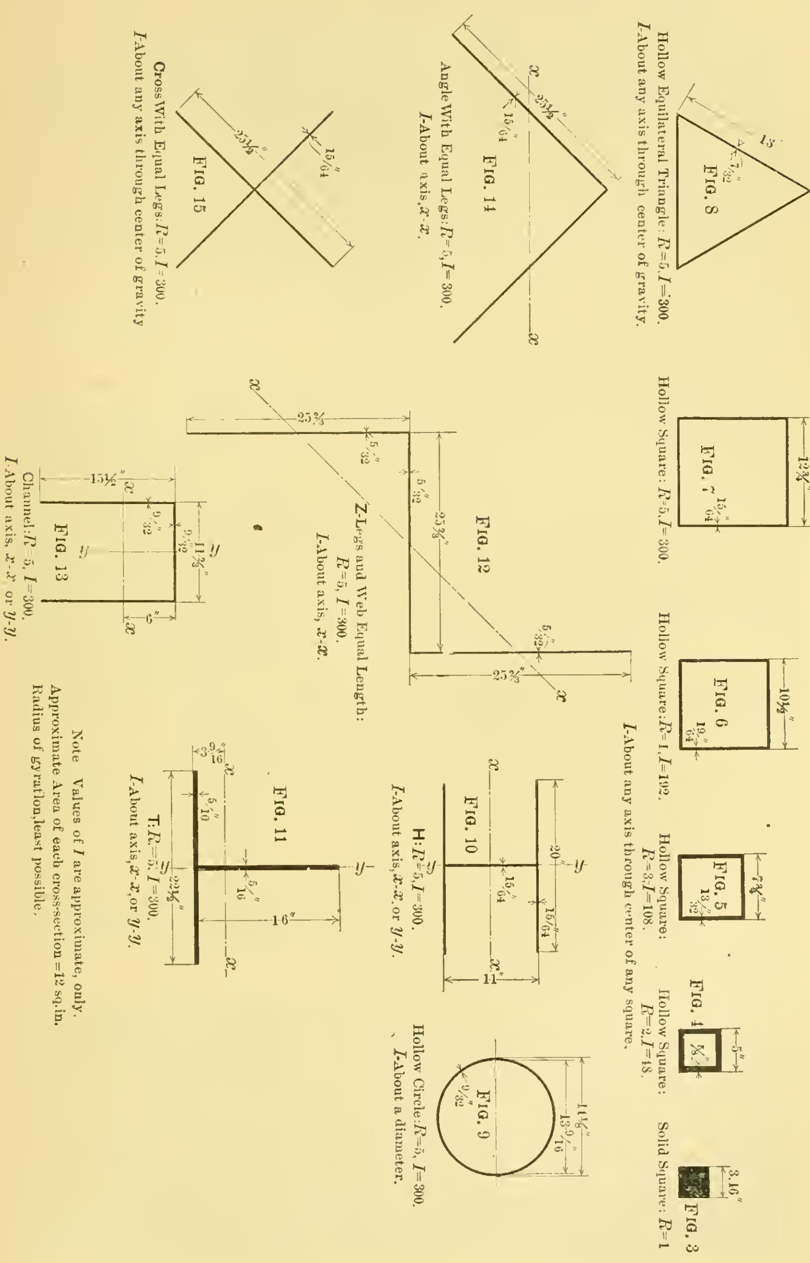


FIG. 3
Solid Square: R=1
3.16"
FIG. 4
Hollow Square: R=2, I=48.
5"
FIG. 5
Hollow Square: R=3, I=108.
7¾"
13/32"
FIG. 6
Hollow Square: R=4, I=192.
10¼"
19/64"
I-About any axis through center of any square.
FIG. 7
Hollow Square: R=5, I=300.
12¾"
15/64"
FIG. 8
Hollow Equilateral Triangle: R=5, I=300.
I-About any axis through center of gravity.
13
7/32"
FIG. 9
Hollow Circle: R=5, I=300.
I-About a diameter.
14⅛"
13 9/16"
9/32"
FIG. 10
H: R=5, I=300.
I-About axis, x-x, or y-y.
20"
11"
15/64"
FIG. 11
T: R=5, I=300.
I-About axis, x-x, or y-y.
16"
22⅝"
3 9/16"
5/16"
FIG. 12
Z-Legs and Web Equal Length: R=5, I=300.
I-About axis, x-x.
25⅝"
5/32"
FIG. 13
Channel: R=5, I=300.
I-About axis, x-x or y-y.
15½"
11⅞"
6"
9/32"
FIG. 14
Angle With Equal Legs: R=5, I=300.
I-About axis, x-x.
25⅛"
15/64"
FIG. 15
Cross With Equal Legs: R=5, I=300.
I-About any axis through center of gravity
25⅛"
15/64"
Note Values of I are approximate, only.
Approximate Area of each cross-section =12 sq. in.
Radius of gyration, least possible.

Mr. Horton. The writer would extend the radius of gyration to the elements making up the cross-section of a column, thereby limiting the thickness of the material. The radii length on a transverse section of a plate should never be more than the radii length of the column of which the plate is a part, or if it is, such parts should be used at a decreased unit stress, found by substituting the value of $\frac{l}{r}$ thus obtained in the general formula.

Angles should be one-fifth the size of the transverse dimensions of a member, and not less than the thickness of the plates.

In the case of columns with projecting portions, such as angles, Z's, etc., $\frac{l}{r}$ (where l = projection and r = radius corresponding to thickness) must be doubled and substituted in formulas.

Where a part is made of several thicknesses riveted together, the transverse radius of such a part will be taken as the radius of the same as though solid and divided by the square root of the number of pieces used.

The radii length of a lattice panel or the pitch of the lattice, with the radius of gyration of neutral axis parallel with central line of the web, of a built channel or similar section, should never exceed the radii length of the entire member. The lattice need not exceed two diameters of the rivet. The radii length of the lattice between the connections should not exceed the radii length of the member on which the lattice is used.

The lattice should have the ability to carry shear, assuming the column to be supported at its two ends or in the center:

1.—At the unit strains allowed in the column itself, an assumed uniform load equal to 10% of the load sustained by the column;

2.—At a unit stress of half the above, the weight of the column itself.

The writer wishes at this time to emphasize his faith and belief in proportion—the "Rule of Three" of our ancestors. It is the fundamental basis of comparison in all things.

Table 9 is an outline for five 2-built channel lattice columns.

Each column is in exact proportion, by the ratio of 2, in all its three (and more) dimensions, to the next of the series. It follows at once that the cross-section of the columns will be as the square, and, for the same radii length, their weight as the cube. The writer has outlined for the center of this group of five columns a rationally proportioned 12-in. 2-built channel column having a section of 23 sq. in.; he has also doubled it, and doubled it again. He has also divided the 12-in. 2-built channel column in each of its dimensions by 2 and by 4, and in this tabulation by direct proportion there are five 2-built chan-

Mr. Horton.

nel columns. There is every reason to believe that the 3-in. 2-built channel column, at $\frac{1}{4\,096}$ part of the weight for a proportional radii length of the 48-in. 2-built channel column, can be investigated with reasonable certainty as to any in the group of columns that are in direct proportion in all their elements, that is, size of rivets, size of lattice, and pitch of rivets; and it is in this way that research can be carried out at comparatively trifling cost. With the testing machines already available, the truth can be developed as to any or all of the moot questions as to value of cross-sections and radii length.

TABLE 9.

Built channel columns.	Section.		Area, in square inches.	Radius.	Rivets.	Double lattice.	R. of gyration, neutral axis, parallel to center line of web.	Length, for $\frac{100\,l}{r}$.
	Plates.	Angles.						
3-in.	$2 - 3 \times \frac{1}{8}$	$4 - \frac{3}{4} \times \frac{3}{4} \times \frac{1}{8}$	1.44	1.08	$\frac{3}{16}$-in.	$\frac{3}{8} \times \frac{1}{8}$	0.22	9 ft.
6-in.	$2 - 6 \times \frac{1}{4}$	$4 - 1\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{4}$	5.75	2.17	$\frac{3}{8}$ in.	$\frac{3}{4} \times \frac{1}{4}$	0.43	18 ft.
12-in.	$2 - 12 \times \frac{1}{2}$	$4 - 3 \times 3 \times \frac{1}{2}$	23	4.33	$\frac{1}{4}$-in.	$1\frac{1}{2} \times \frac{1}{2}$	0.86	36 ft.
24-in.	$2 - 24 \times 1$	$4 - 6 \times 6 \times 1$	92	8.66	$1\frac{1}{2}$-in.	3×1	1.73	72 ft.
48-in.	$2 - 48 \times 2$	$4 - 12 \times 12 \times 2$	368	17.32	3 -in.	6×2	3.55	144 ft.

Table 10 is a second compilation for five columns having the same areas and dimensions as the columns in Table 9.

TABLE 10.

Built channel columns.	Section.		Area, in square inches.	Radius.	Rivets.	Double lattice.	R. of gyration, neutral axis, parallel to center line of web.	Length, for $\frac{100\,l}{r}$.
	Plates.	Angles.						
48-in.	$8 - 48 \times \frac{7}{8}$	$4 - 4\frac{1}{2} \times 4\frac{1}{2} \times 1$	368	14.82	1-in.	$4 \times \frac{3}{4}$	1.38	124 tt.
21-in.	$8 - 24 \times \frac{7}{16}$	$4 - 2\frac{1}{4} \times 2\frac{1}{4} \times \frac{1}{2}$	92	7.41	$\frac{1}{2}$-in.	$2 \times \frac{3}{8}$	0.69	62 ft.
12 in.	$8 - 12 \times \frac{7}{32}$	$4 - 1\frac{1}{8} \times 1\frac{1}{8} \times \frac{1}{4}$	23	3.71	$\frac{1}{4}$-in.	$1 \times \frac{3}{16}$	0.35	31 ft.
6-in.	$8 - 6 \times \frac{7}{64}$	$4 - \frac{9}{16} \times \frac{9}{16} \times \frac{1}{8}$	5.75	1.85	$\frac{1}{8}$ in	$\frac{1}{2} \times \frac{3}{32}$	0.17	16 ft.
3-in.	$8 - 3 \times \frac{7}{128}$	$4 - \frac{9}{32} \times \frac{9}{32} \times \frac{1}{16}$	1.44	0.93	$\frac{1}{16}$ in.	$\frac{1}{4} \times \frac{3}{64}$	0.09	8 ft.

The columns in this group will not require a testing machine, because when we have divided down from 48-in. 2-built channel columns to the 12-in. 2-built channel columns, 23 sq. in. in area, and find four 12-in. plates massed together making 12 by $\frac{7}{8}$ in. of metal combined

Mr. Horton. with $1\frac{1}{8}$ by $1\frac{1}{8}$ by $\frac{1}{4}$-in. angles with 1 by $\frac{3}{16}$-in. lattice, all secured by $\frac{1}{4}$-in. rivets, common sense will indicate that the columns in this group should not be used.

The "Rule of Three" may be accepted as an agent, to assist in approaching the testing machine with columns of a size and cost so that we may hope for extended research. The "Rule of Three" may also be accepted as an agent to assist our common sense, as shown in the second compilation.

In the foregoing, the writer has attempted to point out the desirability of using all the rivet section possible in combining the parts of a composite compression member.

All research which is available indicates that the thickness of the material in the rectangular compression member has most to do with its efficiency, thick material being required for short radii length, and reducing in thickness as the radii length increases.

The piling together of relatively thin plates in multiple, with a few tack rivets, and assuming that the mass is homogeneous is dangerous.

Some comprehensive proportion of stress through the compression member must be accepted as shear, and must be provided for; if, on a 12 or 15-in. 2-channel strut of medium size, practice dictates lattice of a weight equal to, say, 30% of the scantling weight of the member, the "Rule of Three" will indicate that these same relations must be extended to large or small members.

It is not formulas that are needed to extend our knowledge of the compressive member, but comprehensive research by physical tests.

Mr. Shearwood. F. P. SHEARWOOD, M. AM. SOC. C. E. (by letter).—Mr. Worcester's curve appears to be more rational than any of the others he has plotted; still, in common with all column formulas, his assumes that the flexural stresses only result from the tendency of the member as a whole to bend, and no reduction is allowed for the secondary bending from the unsupported component parts and other unavoidable bending stresses which occur in many of the compression sections now in use, and especially in those having radii of gyration relatively large in comparison with their areas.

Strict adherence to a specified column formula has perhaps done very much to force designers to use compression members which are undesirable in nearly every way except that they meet the requirements of the formula economically as regards material.

All, or nearly all, specifications have called for the unit stresses in columns to be determined solely by the ratio of their length to their radius, the latter to be calculated from the moment of inertia of the section, without regard to whether the lattice (if used) is capable of developing it, or whether, in so doing, secondary stresses are induced.

The latticed double channel section with flanges turned out, so frequently used for compression members of truss bridges, is a good illustration of the incompleteness of the ordinary column formula. This section is generally used because, with a given width from out to out of chord gusset plates, it will give a strut having the largest radius, and therefore the highest permissible unit stress; but, if investigated, it will be found that the following stresses are almost inevitable in such a section, and of these the column formula takes no account, and they are practically unprovided for: Mr. Shearwood.

I.—Stress due to the flexure of the unsupported parts between lattice-bar connections, which is coincident with that due to the flexure of the column as a whole;

II.—Stress due to the eccentricity of the end connections, since the center of gravity of either channel is usually some considerable distance from the center of the gusset plates;

III.—Stress due to the eccentricity of the lattice-bar connections; for it is usually impracticable to arrange the bars so that they will intersect on the center of gravity of the channel;

IV.—Serious but less determinate stresses are probably induced at or near panel points, where, owing to the necessary connections, it may be impracticable to provide the last few feet of an important compression member with either tie-plates or lattice bars; and, even when center diaphragms are provided, the continuity of the lattice system is broken up, resulting in unknown bending moments in the member.

V.—In nearly all bridges, the loads are applied more or less on the inside of the trusses, thereby inducing longitudinal shear in the several members, which in turn must stress the eccentrically connected lattice bars, and increase the local stresses, as in III.

VI.—Lattice bars, having of necessity to be placed at an angle to the direct stress of the member, create a distortion and bending when the main member is under strain.

Most of the foregoing defects are absent or are minimized in members having plate diaphragms, such as **H**-shapes, which are symmetrical about every axis. Secondary bending is largely eliminated, and all metal resists stress in direct lines to the applied loads. They are well adapted to transfer any uneven application of load, but, unfortunately, owing to their relatively small radius, they cannot be made to figure as economically as flimsy latticed members.

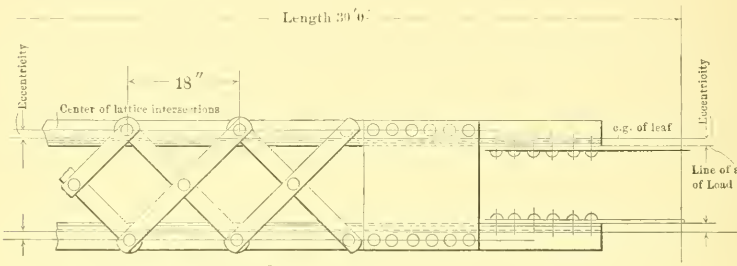
It seems to the writer that columns with their several parts tied together with solid plates should have more favorable consideration than those which are occasionally tied together with redundant and stress-inducing lattice bars.

Mr. Worcester, in common with many other authorities, apparently attributes no advantage to fixed ends over pin ends.

It would seem reasonable that members with fixed ends should have their lengths multiplied by some factor (say 0.7) when the allowed unit stress and limiting lengths are computed.

Mr. Shearwood.

Fig. 16 is given as an example, and shows the direct unit stress allowed by the straight-line formula of the American Railway Engineering and Maintenance of Way Association, in which only the flexure of the column as a whole is considered, and also that permissible if the flexure of the unsupported portions of the individual leaf and bending from the eccentricity of the lattice bars are also provided for.



Radius of gyration = 5.9"

" " " of single leaf = 0.92"

Area of section = 33 sq. in.

Allowed stress by straight-line formula = $16000 - 70\frac{l}{r} = 11720$ lb. per sq. in.

Permissible stress if some of the other stresses due to flexure are provided for.

Unit stress		16000
Deduct for flexure of Column as a whole	4280	
" " " " unsupported leaf	1370	
" " bending stress from eccentric lattice connection	700	6350
		9650 Permissible Stress

FIG. 16.

It seems probable that the disregarded secondary flexural stresses in latticed columns have caused the tests on short lengths to disagree with column formulas which are based on the compression value of the metal. In devising a new column formula, the many inherent weaknesses of latticed forms should be taken into account. Such a formula would have the advantage of discouraging the use of exaggerated forms with redundant metal, and encouraging the use of members with continuously connected component parts.

Mr. Rights.

L. D. RIGHTS, ASSOC. M. AM. SOC. C. E.—This paper brings forward a subject which is of interest, not only to engineers who work with structural steel, but to all constructors who use columns of other material.

Believing that there is a feeling among a number of engineers that the "factor of ignorance" in regard to steel columns has an insufficient margin, the author considers all the available tests, reduces them to equivalent working values, and plots the results. He then breaks away from any attempt to assume a theoretical formula, and introduces a curve which agrees fairly well in its middle portion with the average of the tests, and has the limiting values of 12 000 lb. at one end and 120 $\frac{l}{r}$ at the other. Mr. Rights.

As shown by the author's diagram, Fig. 1, most of the formulas now in common use indicate considerably higher values than those given by the proposed curve, and the question at once arises whether engineers are warranted in making such a radical reduction. In the light of present knowledge, the speaker does not feel that it is advisable to take such a step. Many of the tests which the author has plotted are from twenty to twenty-five years old; some of them are on plain shapes, and many of them, as indicated, are for iron. The present practice has been built up from these same tests, and is an attempt, perhaps in a makeshift way, to accommodate itself to the improved conditions of manufacture and details, which have changed materially since the tests were made. It is the speaker's belief that engineers would hardly feel justified in recommending this increased expense to their employers or clients.

Although the speaker cannot agree with the author as to the large reduction proposed, nevertheless, he feels that some enthusiasts, over-confident in the supposed knowledge concerning the present state of manufacturing, have increased the working values beyond safe limits, and he would suggest that there is a middle ground. He would like to offer as a temporary formula, and more or less of a compromise, the straight line produced by $15\,000 - 75 \frac{l}{r}$.

For the initial point, 15 000 lb. seems to be a satisfactory value. A very large proportion of the steel now used has an average ultimate stress of 60 000 lb., with an elastic limit of 33 000 lb., and the values suggested, if properly reduced, would come within what is, at present, considered a safe limit. It will be noted from the diagram, Fig. 17, that this straight line agrees with the author's plotted tests in the middle portion fully as well as the proposed curve, and that it would be practically tangent to the proposed curve at this position.

While it is desirable to cut off the column formula at some higher value of $\frac{l}{r}$, the question also arises as to where this point shall be. Some engineers favor 100, others, 120, others, 125, and all of them probably have had occasion at some time to consider columns at even a higher value of $\frac{l}{r}$ than 125. When such high values become absolutely necessary, the engineer uses the old formula with discretion. It is the speaker's belief that, instead of attempting to saw off the curve at some arbitrary point, it should be made fool-proof by giving it safe values above 200 $\frac{l}{r}$. It will be seen that the proposed straight line

Mr. Rights. would end at $200\,\frac{l}{r}$, and would give fairly low values between this and $120\,\frac{l}{r}$.

In the light of our present knowledge, or ignorance, the straight-line formula would seem to be adequately accurate for all practical use, and the speaker feels that engineers could not do better than adopt such a simple formula until more is learned about the subject.

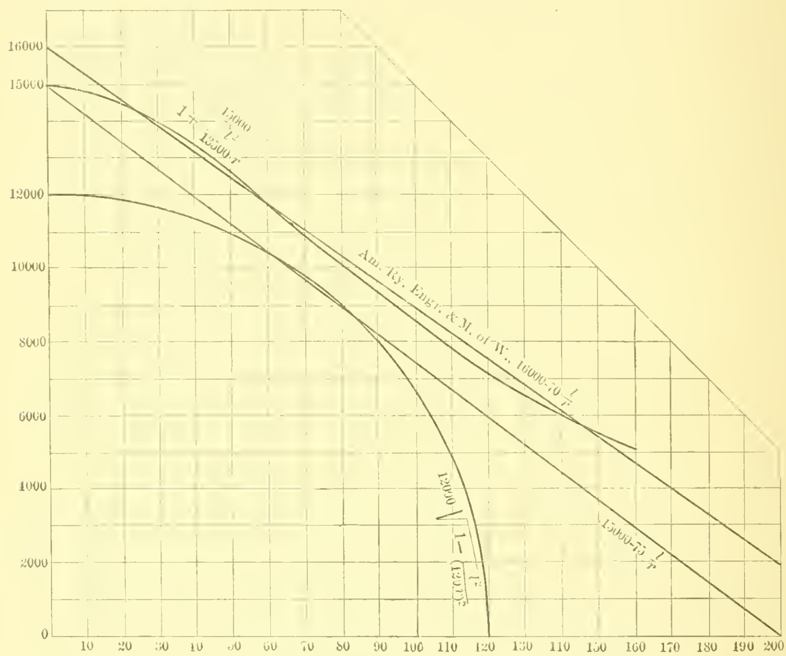


DIAGRAM OF COLUMN FORMULAS

FIG. 17.

The speaker's suggestion would be that engineers either stick to their present formulas, using them in a conservative manner, or adopt some simple straight line, as suggested herein. New tests are needed, not new formulas, and, until these tests become available, it would be better for engineers to work conservatively along the lines they have been taught.

Mr. Carpenter. A. W. CARPENTER, ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Worcester apparently overlooked the fact that full-sized tension members do not develop the strength of specimen test pieces, and his comparison of the ultimate strength of full-sized columns with the ultimate tensile strength of test specimens, therefore, is hardly on a fair

Mr. Carpenter.

basis. In proof of the statement that full-sized tension members do not develop the strength shown by test specimens of the same material, the writer would cite the tests* by J. E. Greiner, M. Am. Soc. C. E., on built-up tension members; the "Tension Tests of Steel Angles with Various Types of End-Connection,"† by Frank P. McKibben, M. Am. Soc. C. E.; and any bridge engineer's records of tests of full-sized eye-bars.

Table 11 is a summation of the results of these tests in form to bring out the point desired.

TABLE 11.

Description of tests.	Specimen tests.			Full-sized tests.		Ratio of full-sized to specimen tests.	
	Material.	Yield point, in pounds per square inch	Ultimate strength, in pounds per square inch.	Yield point, in pounds per square inch.	Ultimate strength, in pounds per square inch.	Yield point.	Ultimate strength.
J. E. Greiner's tests of built-up tension members: Nos. 3 to 10 of Series A; Nos. 11 to 18 of Series B: Average of 16 tests of full-sized members..................	Mild steel.	38 250	58 200	45 030	50 650	1.19	0.87
J. E. Greiner's tests of single-angle tension members connected by both legs: Average of 4 tests.	Mild steel.	37 580	57 280	27 450	51 050	0.73	0.89
McKibben's tension tests of single angles connected by both legs: Average of 12 tests..............	Mild steel.	35 040	56 480	Not given.	46 750	......	0.83
Average of 70 tests of full-sized eye-bars reported in Mr. Greiner's paper	Medium steel. (?)	37 342	63 582	31 270	57 745	0.84	0.91
Average of 24 tests of eye-bars 10 by $1\frac{1}{16}$ to $1\frac{5}{16}$ in., made under the writer's direction................	"	40 187	61 630	30 440	59 160	0.76	0.96

Note.—All test specimens unannealed, except those of the last item.

It should be stated that, of Mr. Greiner's tests on built-up members, Nos. 1 and 2 of Series A and all of Series C were excluded from Table 11 for the reason that the members all had defective (intentionally so designed) end connections; likewise, only the angles which were connected by both legs in the angle tests mentioned were considered; therefore, the results are the most favorable possible toward the best development of strength.

The high ratios of ultimate strength of eye-bars to that in specimen tests must be considered badly offset by the low yield-point ratios. The yield-point ratios for Mr. Greiner's angles are certainly not favor-

* *Transactions*, Am. Soc. C. E., Vol. XXXVIII, p. 41.

† *Engineering News*, July 5th, 1906.

Mr. Carpenter able to the tension side of the argument. The other tests by Mr. Greiner were made on members built of small sections, which probably accounts for the high yield-point values. Following the usual laws, lower results for yield point and (in lesser degree) for ultimate strength would be found in the members of truss bridges and other structures, which are built of thicker sections.

It would seem that, excluding eye-bars, it would be unsafe to call the average ultimate unit tensile strength of full-sized tension members more than 0.85 times the corresponding strength of test specimens of the same material; or, assuming 60 000 lb. as the ultimate unit tensile strength of structural-steel test specimens, the corresponding average strength of full-sized tension members would be 51 000 lb. Now, put Mr. Worcester's Diagram of Column Formulas on the basis of $\frac{16\,000}{51\,000}$ for steel and $\frac{16\,000}{\frac{5}{6} \times 51\,000}$ or $\frac{16\,000}{42\,500}$ for wrought iron, and the value he has chosen as the unit stress for the ratio, $\frac{l}{r} = 0$ (12 000 lb.), will become 14 000 lb. (practically).

Mr. Worcester entirely disregards the effect of end conditions on the columns tested, treating columns with flat and hinged ends alike. This seems to be rather unsatisfactory, since the theoretical influence of the end conditions on the strength of columns is well backed up by tests, and, undoubtedly, conditions arise which justify a distinction in this regard. The writer is wholly in favor of a single formula for bridge work, and that based on hinged ends, since this condition is closely approximated in pin-connected members, and the strengthening effect of greater fixity of ends in members with riveted connections is offset in unknown degree by secondary stresses and unavoidable eccentricity of loading. It would seem preferable to base a curve on, or compare formulas with, full-sized tests of columns the end conditions of which are alike. For building work and special cases, in which the condition can be unquestionably realized, a formula for columns with fixed ends would seem to be entirely proper.

Mr. Worcester mentions the tests of Tetmajer, Marshall, and Christie as "full-sized." In his excellent paper, entitled "The Practical Column under Central or Eccentric Loads,"* Mr. J. M. Moncrieff gives separate diagrams covering all the important series of tests of columns made, up to the date of the paper, and apparently includes all the tests cited by the author. According to these diagrams, the tests of Tetmajer, Marshall, and Christie were on very small "full-sized" members, generally, such as bars 1 in. square, small angles, and other shapes and tubes. It seems that some distinction should be made between these (especially the solid bars of insignificant size) and large

* *Transactions*, Am. Soc. C. E., Vol. XLV, p. 334.

Mr. Carpenter.

columns, such as those tested by Bouscaren, Strobel, and the Watertown Arsenal. It was noted that the particularly low result of 28 000 lb. ultimate strength for a column having $\frac{l}{r} = 30$ was obtained in the series of "55 tests at Watertown Arsenal of 3-in. square bars (cold-straightened), mostly on pins 1½ in. in diameter, eight being on pins from ⅞ in. to 2¼ in. in diameter," all of wrought iron. It would seem that such tests should be given very little weight in this consideration. The writer has failed to note any test of a properly-constructed, centrally-loaded large column which gives any such low result. The author's statement regarding a factor of only 2 between the ultimate strength of columns and a working stress of 16 000 lb. per sq. in. in compression seems to be misleading, because, if the 16 000 lb. be considered the constant for reduction in one of the usual column formulas, such tests as have been made on large columns show an average factor of considerably more than 2—perhaps 2.5 for mild steel—and it has been pointed out that the average factor in tension is about 3.2. The range of variation from the average is thought to be about the same in tension as in compression. A careful study of tests of large steel columns leads the writer to think that it would not be far wrong to take, as the value representing the ultimate strength of well-proportioned and properly-detailed columns, in the numerator of the Gordon-Rankine or other equivalent column formula, 41 000 lb. for mild steel and 36 000 lb. for wrought iron. These values would require, for equal factors of safety based on ultimate strength, approximately the following comparative values:

For tension in steel, 16 000 lb. per sq. in.
For compression in steel, 13 000 lb. per sq. in.
For tension in wrought iron, 13 000 lb. per sq. in.
For compression in wrought iron, 11 000 lb. per sq. in.

In spite of the author's remarks, it seems difficult to get around the fact that engineers do and must design with the elastic limit in view, and not the ultimate strength, and that the structure is unsafe and possibly ruined when the elastic limit (or more properly perhaps, the yield point) is passed, in tension as well as in compression. Also, there is considerable strength beyond the yield point in compression, which, as far as it goes, is just as valuable as the tensile strength beyond that limit. There appears to be a lack of data on the elastic strength of columns. Such as the writer has been able to find, indicate that the elastic strength will be found below that of test specimens, but not more so than with eye-bars in tension. There also appears to be much greater danger from imperfect workmanship and injuries to material, in the case of columns, than in tension members, for which reason the writer agrees with the author, that a lower unit

Mr. Carpenter. should be used in compression than in tension, and thinks that perhaps the ratio derived from the ultimate strength values proposed, will be satisfactory, that is, a compression unit of about eight-tenths of the tensile unit.

It is at this point that the writer would ask, speaking from the viewpoint of a bridge designer, why reduce the compression value? Why not raise the tension value? Was not the 16 000-lb. unit chosen with a view to increased loads, and has not the test of years proven that railway bridges can carry, with absolute safety and without appreciable deterioration, much higher stresses than the equivalent of the 16 000 lb.? If the question of maximum loading is settled, the writer sees only extravagance in designing a steel railroad bridge for the tension unit stress of 16 000 lb., the usual allowance being made for impact and workmanship of the high standard generally required.

Neither experience with columns in structures, nor study of tests, convince the writer that there is any cause for alarm in the use of the 16 000-lb. compression constant in working formulas for steel columns, unless it be that one cannot depend on having columns properly proportioned and properly detailed. An analysis of most of the large columns which have been reported as showing unsatisfactory strength will show that the columns were defective in design, as compared with the requirements of good modern practice. The writer thinks that the principal trouble, if any, will be found in the column details, and that if the same attention is given to the concentric application of loads and to rivet connections as in tension members, ample provision is made for the full transmission of stress to all parts of members through details, and the ratios of width and length to thickness are kept down to the limits of conservative modern specifications, so that columns will have some body and not be "built of sheet iron," and there will be no failures nor cause for alarm with the 16 000-lb. compression constant in columns of ordinary size and construction.

It seems to the writer that, instead of being cut off on a horizontal line for very low values of $\frac{l}{r}$, say less than 20, a formula line should theoretically rise abruptly to the compressive limit of the material at $\frac{l}{r} = 0$. This, of course, would make a complicated formula, and, as such short columns are unusual, it may be as well to omit this extra complication. It will be noted that Mr. Worcester omitted to plat values for columns having $\frac{l}{r} < 20$, although the series of tests he mentions includes a large number of such with values which would rise to the limits of his diagram.

In conclusion, the writer would state that he is opposed to a formula that is "chopped off" at the "long" end. Such a formula may be all right to design with, but it is "no good" for use in determining the strength of existing structures. A formula which best represents the true strength of a column, assuming its design is correct and its physical condition is up to the average, seems to be the proper one, and he does not know of any formula that fulfills this condition as well as the Gordon-Rankine formula, in the form: Mr. Carpenter.

$$\frac{C}{1 + \frac{1}{18\,000}\left(\frac{l}{r}\right)^2}$$

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

EFFECT OF EARTHQUAKE SHOCK ON HIGH BUILDINGS.

Discussion.*

By Messrs. Guy B. Waite and E. G. Walker.

Mr. Waite. Guy B. Waite, M. Am. Soc. C. E. (by letter).—In the paper entitled "Wind Bracing for High Buildings,"† the writer assumed a horizontal wind pressure of 30 lb. per sq. ft. as acting against the entire windward side of the structure. The stresses induced by the wind force were assumed to be resisted by the construction acting as a cantilever. The building was assumed to be plumb. No increased bending moment, caused from an overhang, by wind pressure was thought necessary. Provided the weight of the building was sufficient to counteract the overturning moment due to wind, the mass or weight of the building did not enter into the discussion.

The resistance of the structure to the horizontal component of wind pressure was discussed without reference to whether it weighed 100 lb. or 1 000 000 lb. Mr. Chew's first conclusion, that the stresses produced by earthquakes are similar to those caused by wind, is misleading. While the force from wind pressure is definite, and is distributed throughout the structure, the force from an earthquake is indefinite and unmeasurable, and is distributed throughout the structure only by acceleration taken at one place—the foundation.

To compare the two forces, it may not be improper to liken them to the working of horizontal engines under a given pressure: the force

*This discussion (of the paper by R. S. Chew, Assoc. M. Am. Soc. C. E., printed in *Proceedings* for January, 1908), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

†*Transactions*, Am. Soc. C. E., Vol. XXXIII, p. 190.

of the wind being similar to small pistons with long strokes, while the force of the earthquake is similar to a piston having an indefinitely large area with a very small stroke. In the former case, the engine force is distributed against the side of the building, is limited in amount, and is substantially all taken up by the structure; in the latter case, the immeasurable force from the unlimited piston area is simply carried through the foundation of the building, the only force taken up by the building being due to the vibration of the foundation. Mr. Waite.

On the leeward side of a building the force of the wind is practically nil, while the force of the earthquake is substantially the same on each side of the foundation. The wind force is all taken by the building, whether it be a heavy or a light structure. The vibration of the foundation of a building, from earthquake (other things being equal), will be the same, whether it be heavy or light; the momentum only will vary with its weight.

The amount of the vibration of the superstructure of the building caused by earthquake will depend on the rapidity and length of stroke of the so-called piston, the elasticity of the material in the construction, and the design of the building. For instance, if the foundation of the building were divided into two horizontal parts, with roller or ball bearings between the parts, the earth vibration could pass through without materially affecting the upper structure, whether heavy or light, the only vibration being due to the friction of the bearings.

If the vibrations were sufficiently rapid, and the columns sufficiently long and elastic, probably little vibration would be felt, whether the building were light or heavy.

While some structural resistance is absolutely necessary in the case of wind—the force being definite and positive—the structural resistance required in the case of earthquakes will depend on circumstances and design, the force itself not being against the building.

Now, with a properly designed building, having a given mass and a certain given vibration of the foundation, but little vibration may be caused to the superstructure, while, with more unfavorable designs, vibration enough to wreck the building might be caused, even if the best of wind bracing were used.

Buildings may be definitely braced against wind pressure, but they cannot be definitely braced against earthquakes.

A pile of brick may be laid so that it will resist wind pressure, but will fall on account of the acceleration caused by an earthquake; this, however, is no reason why a sober fat man cannot stand up as well as a lean man under both earthquake and wind pressure. In Nature we see resistances increasing with the size and weight of objects; if this idea be carried out in buildings, we will but obey the mathematical laws which govern the stability of all things composed of matter. The heavier the building the more horizontal resistance it will naturally

Mr. Waite. have; but the writer does not agree that the resistance should increase, as indicated in the rational analysis from which the author draws his conclusions.

It is generally considered that additional weight helps to distribute wind pressures, and that the force of the wind is largely used up in frictional and other internal work on the mass, to the relief of specifically designed resistances.

When a force is set up in a building, from the vibration of the mass composing the foundation, why should not much of this force, which would otherwise be communicated to the braces and connections of the superstructure, be lost in doing the internal work referred to above?

As buildings can be properly designed to resist wind, and can only be designed to escape the vibration of earthquakes, it is believed that they should be constructed so that their parts will withstand wind pressures, and that they will then be amply provided to withstand earthquakes.

Wind pressures are very frequent, while earthquakes are very rare. A building will have need for resistance to wind pressures several thousand times for one possible resistance to earthquake.

There is no evidence to show that a modern wind-braced building is not strong enough to withstand the vibration from an earthquake, but there is considerable evidence to show that it is sufficient.

A building well designed to resist the force of the wind should have plenty of good-sized columns. The connections and braces of these columns to the girders should be strong and positive, and there should be the maximum depth of connections of cross-beams and girders to columns. All constructions composing floors, walls, partitions, etc., should be capable of distributing stresses and of withstanding vibrations.

It seems to the writer that reinforced concrete fulfills these conditions better than any other known material. In reinforced concrete the columns and girders have a monolithic connection throughout their height, and, with proper design, can take stress in both directions. The concrete, being a filling between the reinforcing steel, can take any amount of vibration which will be conveyed by an earthquake. The reinforcing steel is run into the columns, making a stronger connection than possible in steel and ordinary fire-proof construction.

If there be any part of a building in which concrete properly reinforced cannot be designed as light as, and perform the function of resisting stresses better than, any other fire-proof material (in addition to which it preserves the steel), the writer is not aware of it.

Mr. Walker. E. G. Walker, Jun. Am. Soc. C. E. (by letter).—The writer has read with very great interest Mr. Chew's paper and the analysis with which he endeavors to arrive at the facts of the resistance of a steel-framed building to earthquake shocks. At first sight it would appear that this subject is not one which is susceptible of much calculation, but, when the nature of an earthquake disturbance is considered more closely, it at once becomes apparent that a rational analysis may easily be made. Mr. Walker.

As Mr. Chew remarks, the effect of an earthquake is a wave motion. The motion usually commences with small elastic earth-vibrations of short periodicity, followed later by the shock proper, the period of which will be much greater, from 1 to 2 sec., after which the vibrations will become slower and smaller until a quiescent state is again reached.

This being the nature of the force acting on any structure during an earthquake disturbance, the writer does not think that the author's analysis, though correct and in order as far as it goes, is sufficiently extended to take true cognizance of the initial conditions of the problem. Mr. Chew treats his building as an elastic structure, the foundations of which are subjected to an impact by a seismological wave, but he neglects the fact that this wave is followed by others at intervals which, for a short period, may be regarded as regular.

When an elastic body is struck a blow, it tends to vibrate with its own natural period of vibration, and the amplitude gets less and less until the body comes to rest, the maximum distortion, and therefore, also, the maximum stresses, occurring just after the impact. This is the state of affairs Mr. Chew assumes.

The writer submits, however, that to treat the problem on this basis is insufficient. The building should be considered, not as merely subjected to an impact, but as acted upon by a force the intensity of which varies according to a regular periodic law. The building is then compelled to execute forced vibrations, and, if it should happen to have a natural period which synchronises with that of the applied force, a resultant vibration of very large amplitude would be set up, causing extreme stresses. On the other hand, and this, presumably is a more common case, if the natural period of the building and that of the impressed force are different, there would be a continuous variation of stress in all members of the structure though the range would not be so great.

The mathematical treatment of the problem, on these lines, though a little complicated, only follows the orthodox method of investigations into the ordinary problems relating to vibrations. The writer had intended to present a solution in some of the simpler cases, such as those dealt with in the paper, but, unfortunately, the time at his disposal has been insufficient to take up the question in detail. However, it should be possible, by treating the structure as a vertical cantilever acted on by a periodic force, to arrive at a law of displacement for any portion, and thus to get a value for the maximum deflection, Δ, at any

Mr. Walker. point, as well as an accurate knowledge of the range, or extreme values of deflection. The step from this to a calculation of the stresses induced is easy and straightforward.

With regard to the author's first conclusion, it seems to the writer that the stresses produced by a shock will be similar to those caused by wind only in cases where the wind pressure is produced by gusts at fairly regular intervals. The increase or otherwise of stresses mentioned in his second conclusion would be brought out in an analysis on the lines the writer has mentioned, and, in calculating the scantlings of a new structure, the extreme fiber stress allowed could be settled in accordance with the range of stresses found.

This subject of the stresses induced in an elastic structure by seismological disturbances is a very interesting and important one, from both practical and theoretical standpoints. Up to the present, rules and formulas have been mainly the outcome of observation and experiment, rather than of deduction from the theoretical investigation; but there is no reason why the latter method should not be used; so that, with a knowledge of the seismograms which have been recorded from time to time, it should be possible to deduce, with a fair degree of accuracy, the probable stresses which would be induced by a shock, and to provide suitably for them. The writer only regrets that he has been unable to devote the time necessary to work out an analysis on the lines he has endeavored to indicate.

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PAPERS AND DISCUSSIONS.

THE USE OF REINFORCED CONCRETE IN ENGINEERING STRUCTURES.

An Informal Discussion.*

By Messrs. M. S. Falk, Rudolph P. Miller, Eugene W. Stern, and H. C. Turner.

M. S. Falk, Assoc. M. Am. Soc. C. E.—A considerable number of reinforced concrete structures have of late been described with enthusiasm before this Society and in the technical press; and many, if not all, of the published descriptions make it appear that these structures have been a complete success from the time of their inception, causing no trouble to designer, owner, or contractor. Mr. Falk.

As a rule, these descriptions cover the completed structure only, and omit references to the difficulties and dangers encountered during construction.

Candid statements of facts in relation to the use of reinforced concrete are absolutely necessary at the present time; such statements must be accurate, and should conceal nothing, so that they may serve as guides to others who propose this construction for similar classes of work.

Plates XL and XLI illustrate the construction of two buildings, entirely of concrete, which were built during 1907, are now in use, and, to any observer, would appear to be eminently satisfactory. In neither case will the respective owners repeat their experiences, since in both instances they have learned that different methods would have afforded structures which could have been erected more quickly, at less cost, and would have been fully as permanent.

* Continued from February, 1908, *Proceedings*.

Mr. Falk. Plate XL and Fig. 1, Plate XLI, show an ice storage house, the outside dimensions of which are 58 by 92 ft., and with a clear inside height of 42 ft. 8 in. from the top of the basement floor to the underside of the roof slab. The columns supporting the roof are 18 by 12 in. in cross-section, and are embedded at intervals of about 11 ft. in the curtain walls, which are 12 in. thick for the exterior and 10 in. for the interior walls. The building, which is to store cakes of manufactured ice, contains three chambers running the full length of the structure, the only entrance to each being through a small ice chute in the front of the building. There are no windows. At first study, any engineer would claim such a structure to be ideal for reinforced concrete; forms for vertical walls and for one roof slab only were required. The history of the case, however, refutes this.

The building was planned by an architect, who called for proposals, requiring the bidding contractors to design the reinforced concrete work subject to his approval, although he himself wrote the specifications under which the work was to be built, and prepared preliminary plans showing his ideas as to reinforcement. One of the requirements was that the walls and columns were to be designed to withstand an assumed horizontal thrust due to the pressure of the ice. Consequently, the columns were designed, by the contractors to whom the work was awarded, as vertical beams loaded at their centers with the ice thrust. This explains (Fig. 1, Plate XL) why the reinforcing rods in the columns were placed in two lines parallel to the exterior faces of the columns, instead of being spaced more uniformly throughout the cross-section. The rods forming this reinforcement ran, for the most part, the full height of the building, and, as they were not self-supporting, it was necessary to build a wooden structure to hold them before any concreting work was done. This structure is shown in Fig. 2, Plate XL.

The rods in the columns were hooped together at short intervals, not only by outside wires, but also by wires crossing through the center; moreover, in order to space the corner column rods away from the forms, the contractor inserted plate-washers on each corner rod.

The curtain walls were also reinforced with horizontal and vertical rods spaced and wired as shown in Fig. 1, Plate XLI. It is evident that the reinforcement acted as a screen through which the raw concrete was forced to pass.

One clause of the architect's specifications read as follows:

"The centering for columns shall not be over half the height of the building before concreting is commenced and for enclosing walls not over 10 ft. in height, unless otherwise approved."

Although the contractor should have known better, he blindly attempted to follow this clause. The final results of the work, taken in

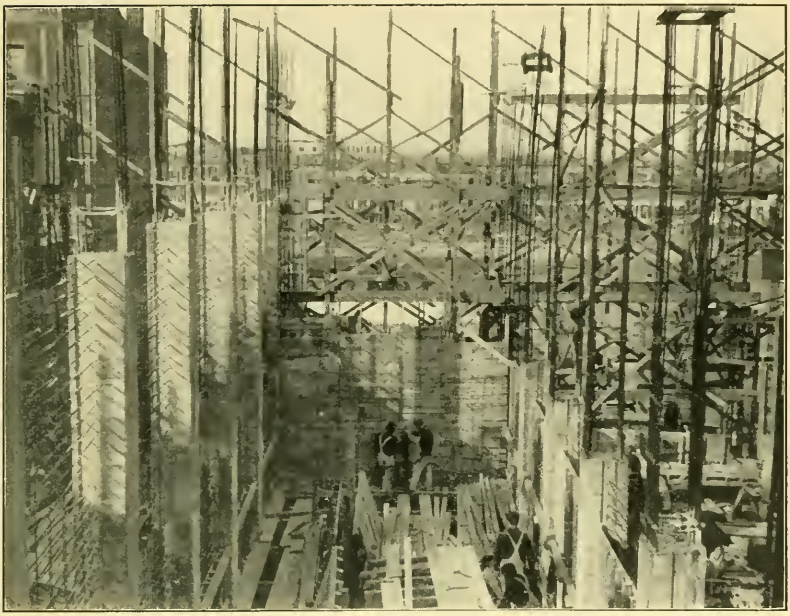


Fig. 1.—Reinforced Concrete Structure for Ice Storage.

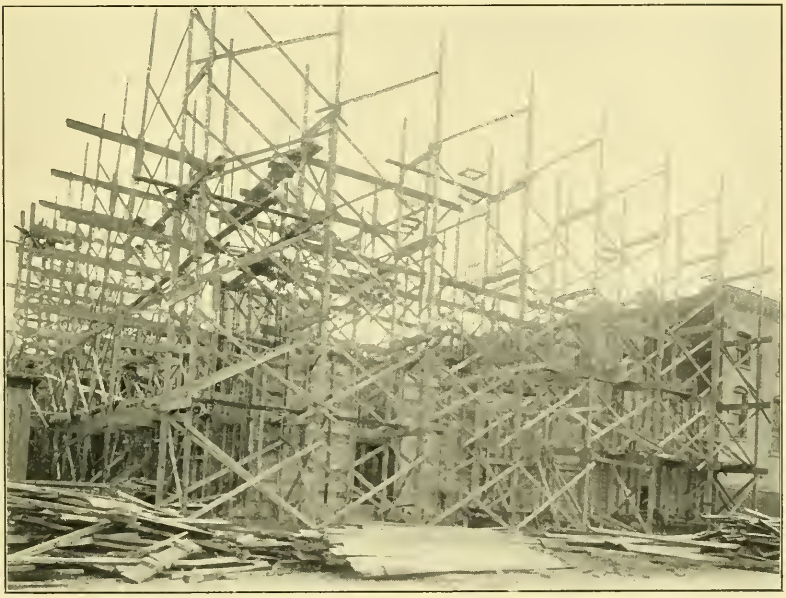


Fig. 2.—Scaffolding to Support Reinforcing Rods.

connection with the design, are shown clearly in the photographs, and require no explanation, except that when the forms for the lower portions of the walls were stripped, the owner, mistrusting both contractor and architect as to the safety of the work, called for engineering advice. Mr. Falk.

The structure was completed, after much difficulty, strictly according to plan; dangerous defects were repaired so that no failure may be expected, and surface blemishes were plastered so that anyone not familiar with the actual construction might believe the building to be an example to be followed.

The building shown by Fig. 2, Plate XLI, was originally designed by an architect as a frame building to be finished in cement stucco; but a reinforced concrete contractor convinced the owner that it would cost but little more to make the building entirely of concrete, and he was given the order to proceed. In fairness to the architect, it should be stated that he was not consulted as to the building after the original plans had left his hands.

When the structure had reached about half way to the second story the owner began to suspect the character of the work which was being done, and decided to complete the building by day's work in charge of an engineer. No difficulties out of the ordinary were encountered until the roof was reached.

The building is 40 ft. square, and there are four interior columns. The roof is a concrete slab, sloping at about 45° from the horizontal, and is supported on the side-walls and on two concrete beams running the length of the building and carried by the concrete columns. The concrete in the main portion of the building had been poured very wet; but when this mixture was placed on the sloping roof forms it refused to stay in place. Therefore, wooden planks had to be placed on top of this concrete in order to hold it down. This method, however, was exceedingly difficult, as the roof was a dangerous place for the workmen. The concrete was changed to a drier mixture, but still required the use of the outside forms. As it was impossible to lay very much of the roof in one day, there were many joints. After the concrete had set so that workmen could move about without injury to it, a surface coat of mortar, in which was incorporated a so-called water-proof compound, was placed. This coat was colored with red oxide of iron, so that the final surface showed a pleasing red. The surface coat was plastered smooth, and it seemed as though all water would be easily shed. The first rain storm, however, showed that the roof leaked almost like a sieve. It must be remembered that this work had been done by day's labor, and not by contract, and that there had been absolutely no incentive for any but the best workmanship.

The speaker consulted several water-proofing companies, asking them to water-proof the roof without destroying the color effect which had been obtained, but not one of these companies would take the work

Mr. Falk. and guarantee it for more than one year. The use of pitch or similar water-proofing material was not permitted on account of the color, nor does the speaker believe that any plastic material would stay on this roof. It was finally decided to apply alum and soap, as in the Sylvester process, and from its application up to the present time the roof has shed the rain. It has not yet passed through both a summer and a winter, and it will be interesting to note what effect the temperature will have on a thin slab of this kind. The speaker would not advise anyone to use a reinforced concrete roof of this kind.

Mr. Miller. RUDOLPH P. MILLER, M. AM. SOC. C. E.—In the speaker's experience, along the line of building construction, the success of reinforced concrete in engineering work is greatly dependent on thorough and intelligent inspection. Many a good design has been completely defeated because of the lack of proper superintendence. The materials being used at the present day in this kind of work are generally reliable, but their improper handling has often been responsible for poor results. It is desired to call attention here to two defects that have been of too frequent occurrence, which can be avoided by a little foresight in the design and by intelligent supervision in the construction: First, the displacement of the reinforcement when the concrete is placed; and second, the formation of cavities in the concrete construction due to complicated reinforcement.

It would seem unnecessary to call the attention of engineers to the danger of the displacement of reinforcing rods or bars in reinforced concrete beams. Concisely stated, if the displacement is upward, there is a loss of strength proportionate to the amount of displacement; if the displacement is downward, the fire-resisting qualities of the construction are impaired, and ability to resist fire is one of the main claims of superiority of reinforced concrete construction. Judging from experience, however, it seems to be important to call the attention of engineers to the necessity of making provision for preventing the displacement of the reinforcement. It is the speaker's opinion that, no matter how carefully bars or rods are placed in the moulds, or what precaution is taken in the pouring of the concrete, there can be no assurance that the reinforcement is in its proper position when the work is completed, unless some means have been used to prevent a movement. The only certain method that has come to the speaker's attention is that used in the so-called "Unit" systems, in which all the reinforcing bars or rods in a beam (and it is equally applicable to column construction), including the stirrups, are secured by heavy wire clamps or other devices in such a way that their relative positions cannot alter. By using washers or spacers the resultant frame can be secured in the forms against a bodily displacement, and held at a proper distance from the outer surface of the finished concrete.

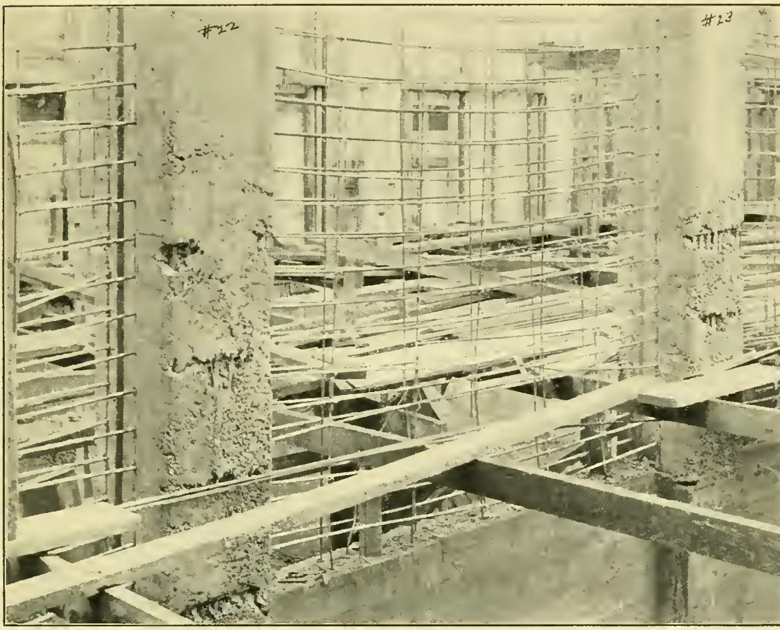
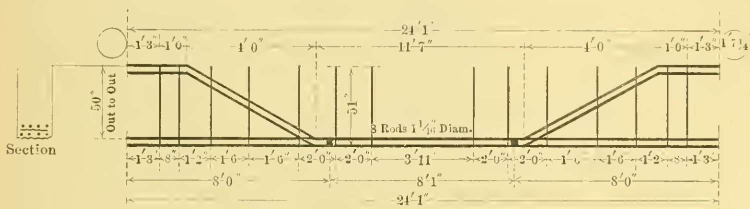


FIG. 1.—REINFORCEMENT OF WALLS AND COLUMNS.

FIG. 2.—REINFORCED CONCRETE BUILDING.

Besides assuring the correct position of the reinforcement, the use of the unit frames greatly simplifies the superintendence of the construction. It requires but a glance (comparatively speaking) to see whether all the reinforcement is in place in the form and whether the proper frame is in each form. The frame having been built up from detailed drawings, previously prepared, the danger of the omission, occasionally, of a bar or rod, of the substitution of a bar of less cross-section, or of the use of too short a bar, is practically eliminated. (See Fig. 1, Plate XLII.) Mr. Miller.

The frames themselves may be fabricated at the shops and shipped to the job; or, if the operation is sufficiently large to justify it, there may be a temporary shop on the premises. The particular advantage in this is that the forms can be inspected and checked before they are put in place. A sample detailed drawing from which the frames are made is shown in Fig. 1.



No. of Pieces	MATERIAL	LENGTH	WEIGHT	REMARKS
4	Rods 1 1/16" Diam.	24′ 1″		
4	" " "	27′ 4″		
	Single Sockets			
2	Double Sockets			
4	Stirrups 1″ x 1/8″	9′ 2″		Holes punched 1″ from each end
6	" "	8′ 10″		
4	" "	8′ 9″		
4	Ties 1″ x 1/8″	12″		
16	Clamps with Bolts			See Standard Sheet No. Q.

FIG. 1.

Fig. 2, Plate XLII, shows another and quite satisfactory method of securing the reinforcement in position when the style of columns used is such as to admit of it. This is a photograph of one of the column-girder connections in the McGraw Building, New York, recently described* by William H. Burr, M. Am. Soc. C. E.

The second detail of construction which seems to have escaped attention is the avoidance of too complicated a reinforcement. In the disposition of the steel, care must be taken that the several elements are not so closely spaced or so arranged as to prevent the concrete from pouring between and around them and thus producing cavities. The size of stone used in the aggregate should be considered in connection with the spacing of the rods, or *vice versa*. When complicated rein-

**Proceedings*, Am. Soc. C. E., for October, 1907.

Mr. Miller. forcement cannot be avoided, the size of the stones should be reduced to suit the condition, or the stone should be eliminated, and mortar should be used. All this applies particularly to column construction and other work where the concrete must fall through considerable height. The speaker has seen a column, the cross-section of which was not more than 20% of its embedded area, because of the cavities formed by the sieve-like action of the reinforcement. An instance of what is meant, though not as serious as the case referred to, is shown in Fig. 3, Plate XLII.

Mr. Stern. EUGENE W. STERN, M. AM. SOC. C. E.—No structural material in recent years has temporarily won such enthusiastic partisanship, or caught the public eye to such an extent, as reinforced concrete. It may be that the reason for this is that it appeals so much to the imagination of the layman.

It is useless to consider all the claims that have been made for it; but one in particular should be flatly contradicted, which is that but little special knowledge is required in the art of working in this material, and therefore that it can be largely done by unskilled labor. In view of the many fatal accidents which have resulted from the improper use of this material, this claim is not as strongly urged as it once was.

A matter of interest in connection with the construction of reinforced concrete work is that contracting firms, or those who exploit patented or deformed bars, are largely responsible for the designs which go into buildings to-day. They submit their own plans, based on the use of these bars or some special method of construction, under some kind of guaranty as to carrying capacity, almost always without any charge for their services. It is the speaker's experience that this method leads to trouble, very often to a lawsuit. The client's interests are supposed to be looked after by the contractor, but, when any trouble happens, his interests, of course, are forgotten.

It is more than ever necessary, in the use of this material, that the structural design and supervision of the work should be placed in the hands of competent professional engineers who represent the owner's interests only.

In no other material of construction is such extreme care necessary, and such intelligent, constant and painstaking supervision required in every part of the process.

Reinforced concrete is a valuable material for use in structures, and has a large field. It has its limitations, however, and often, owing to the enthusiasm of its advocates, has been used where other materials would have answered the purpose better.

As a facing for buildings it has not proven a success, for the reason that it is difficult to obtain a pleasing surface, and ordinarily is

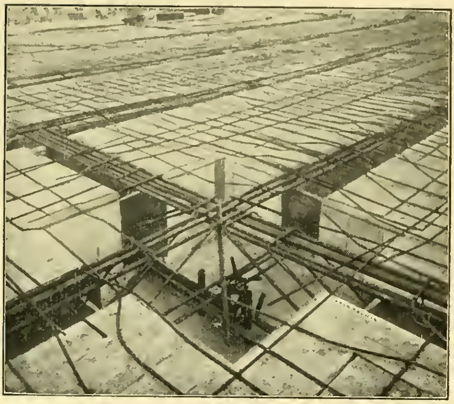


FIG. 1.—UNIT SYSTEM OF FRAMES.

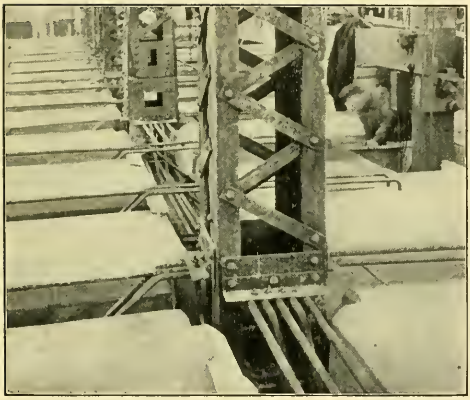


FIG 2.—METHOD OF SECURING REINFORCEMENT.

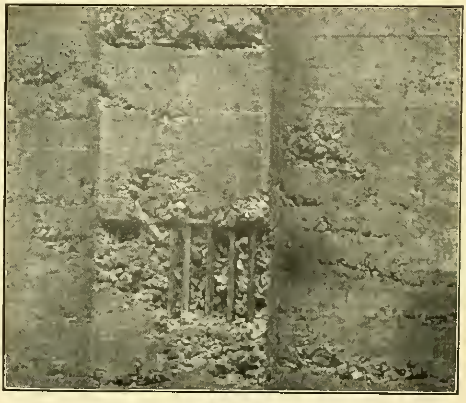


FIG. 3.—CAVITIES IN COLUMN.

more expensive than brick. In the construction of high buildings, the Mr. Stern. speaker believes that it is not as suitable for columns and girders as steel-frame construction. For low buildings, or factory and mill buildings, occupying large areas in outlying districts, where there is plenty of room to handle the material, it has proved a very desirable substitute for mill-constructed buildings.

Among the faults of reinforced concrete work is its tendency to crack, due to the shrinkage of the concrete. The speaker has had to deal with a number of reinforced concrete buildings, and none of these has been free from cracks in various places. A recent case was interesting: In a building there were two rows of columns longitudinally, dividing it into three bays. In the center, between the columns, there were heavy girders, but, in the outside bays, for structural reasons, there were in places light girders. The heavy girders, in shrinking, drew the columns slightly together and the outside girders cracked in the top flange.

The illustrations shown by Messrs. Falk and Miller are interesting in showing what actually happens in practice. The speaker, however, has seen much larger voids in columns than any of those illustrated. In one case, where deformed bars having prongs were used, there were voids in the columns which practically occupied the whole area of the column. There was no attempt on the part of the contractor to scamp his work, but the interlacing of the prongs formed a screen which held up the stone and prevented it from becoming well consolidated in the mixture, with the above result.

The use of reinforced concrete for railroad bridges does not seem to be a proper application, for the reason that constant vibration would tend to cause cracks ultimately, and separate the reinforcement from the concrete.

Where conditions would be favorable to the rusting of steel, reinforced concrete is not suitable, unless cracking can be prevented, as otherwise the reinforcement will ultimately rust out.

Mathematical investigations have been carried to an extreme degree of refinement in reinforced concrete construction, and designs have been worked out on paper for huge structures that stagger the imagination of any but the most enthusiastic. Only recently, a design for a bridge over Spuyten Duyvil Creek, New York City, has been prepared by its Department of Bridges, involving the construction of a reinforced concrete arch of 703 ft. span.

Reinforced concrete is anything but an academic proposition. It is eminently a practical one. Theorists assume for their computations certain conditions, some of which may be possible, and some of which may not be possible, to obtain in the practical operation of construction. Many things in the practical use of this material have yet to be understood, and these can only be learned by experience.

Mr. Stern. Has the state of the art, in the use of this comparatively new material, progressed to such an extent as to warrant the conclusion that, to-day, it is a perfectly safe and legitimate proposition to undertake to build structures, which in magnitude and boldness of conception far exceed anything in existence of similar type? Is it not sounder engineering to progress slowly along well-tried-out lines?

There is wide difference of opinion as to what the working stresses ought to be, particularly in compression. The building codes of the various cities in America are not by any means uniform in this respect, the allowable unit stresses in compression varying from 350 lb. upward, and some engineers have recommended as high a stress as 750 lb. per sq. in.

There have been a great many tests on concrete cubes, the data obtained from which are valuable in this discussion. The highest results have been obtained, of course, where the specimens have been kept in moist sand, or submerged under water; but tests made under such ideal conditions, which rarely obtain in practice, should not be used as a basis for deciding what should be the working stress of concrete in compression, unless these conditions approximate those under which the structure itself is built.

Among the many tests made at Watertown Arsenal, the speaker would refer especially to a series of tests on 12-in. cubes, prepared by the authorities at the Arsenal, the results of which are given in their Annual Reports for 1899 to 1904. These blocks were allowed to set in air, and were stored in a dry, cool building throughout the period of the tests, which were made after periods ranging from 3 months to 5 years. The conditions under which the blocks were stored would be almost identical with those to which reinforced concrete work in building construction would be exposed, and these tests, therefore, would give results more in harmony with actual conditions than those in which the blocks were immersed in water or kept moist in sand for a number of months.

The average of	10	tests,	after	3	months,	was	1 958 lb. per sq. in.
" "	" 16	"	"	4	"	"	2 244 " " " "
" "	" 16	"	"	1	year,	"	3 330 " " " "
" "	" 15	"	"	2	years,	"	2 610 " " " "
" "	" 15	"	"	3	"	"	2 610 " " " "
" "	" 10	"	"	4	"	"	2 960 " " " "
" "	" 2	"	"	5	"	"	2 630 " " " "

Disregarding the 3-month and 4-month tests, the average of 58 tests, after 1 to 5 years, was 2 870 lb. per sq. in. These blocks were all made of 1 part Alpha Portland cement, 2 parts sand and 4 parts broken trap rock, varying in size in the different specimens from ½ in. to 2½ in.

It will be noticed that the 2-, 3-, 4-, and 5-year tests show substantial reductions in strength from the 1-year tests, and the records show, also, that there was a considerable loss of weight, varying from ½ lb. to 2 lb. in each block. Mr. Stern.

There will undoubtedly be differences of opinion as to what fraction of the ultimate strength should be adopted for a safe working stress. To compensate for the great factor of ignorance which exists in the construction of concrete and reinforced concrete work, there should be an ample margin of safety. No matter what care may be taken with the sampling and storing of cement, it is practically impossible, in the process of construction, to prevent, not only some of the material losing its strength, but also to obviate defects in workmanship.

It has been considered for many years that a factor of safety of from 10 to 20 should be used in masonry. The speaker sees no reason, therefore, why a greater load than $\frac{1}{8}$ to $\frac{1}{10}$ of the ultimate strength of laboratory tests on concrete cubes should be used in practice in building construction, which would give between 290 and 360 lb. per sq. in. as a unit stress.

Professor Burr has brought up the question as to whether or not the Watertown tests quoted by the speaker were all made under uniform conditions and with the same brand of cement and other materials. As far as can be learned from the official reports of these tests, Alpha Portland cement was used throughout, and the same quality of sand and stone; moreover, the specimens were stored under the same conditions, throughout the years during which the tests were conducted.

H. C. Turner, Assoc. M. Am. Soc. C. E.—During this discussion, a number of questions have been raised which call for an answer by those who are closely identified with the construction of reinforced concrete buildings. Mr. Turner.

In answer to Mr. Stern's question regarding the preservation of the steel reinforcement in concrete structures, the following is the experience of the Turner Construction Company in razing a one-story building, erected for the J. B. King Company, at New Brighton, Staten Island, in 1902, which was taken down during the summer of 1907 to make room for a larger structure: The building had reinforced concrete walls, 9 in. in thickness to grade line and 5 in. in thickness from grade line to roof line, reinforced concrete interior columns, 11 in. square, and reinforced concrete beams, girders and roof slab. The foundation consisted of spruce piling, cut off at mean tide and capped with reinforced concrete. All steel reinforcement was found in perfect preservation except a few ¼-in. hoops in the wall columns, which were within ¼ to ½ in. of the surface. These showed slight corrosion, which would indicate that it is important to secure all steel reinforcement at least ¾ in. from the exterior surface. The steel in the footings, Mr. Turner. although alternately wet by the tide each day, was in perfect condition. In some cases this steel was within $\frac{3}{4}$ in. of the surface.

Numerous observations of a similar kind have been made by engineers, and it is now generally recognized that steel reinforcement is permanently preserved in concrete structures.

It seems unfortunate that illustrations of standard reinforced concrete work have not been shown, rather than those of generally defective work, although such illustrations are valuable in indicating the character of design and workmanship to be avoided. It must not be assumed, however, that they are typical of reinforced concrete construction. Much excellent work is being done in New York City, and in all cities in the United States. Eight and ten-story buildings are not unusual, and it is to be noted that these buildings have proven especially adaptable for heavy storage or heavy manufacturing.

As Mr. Miller has stated, it is perhaps more difficult to secure good workmanship than good engineering design. This is a matter of organization. Good workmanship should be required, and undoubtedly can be furnished. There is abundant evidence of this, and good workmanship costs but little more than poor workmanship. It is necessary to have a thorough and experienced organization of workmen; but this is just as true in any line of successful business.

Regarding safe unit stresses, there is no reason for a factor of safety of ten. Reinforced concrete buildings have a larger factor of safety than steel buildings, because of the monolithic character of the construction. Concentrated loads are distributed over larger areas because the reinforcement extends in both directions. Vibration is largely reduced. This is well demonstrated in the Ketterlinus Buildings, in Philadelphia. The two buildings are about the same size, 8 stories in height; one has a steel frame with hollow tile floors and brick walls; the other, and later, building has reinforced concrete columns, beams, girders and floors, with brick veneer walls. Both buildings are used for printing and lithographing, and are subjected to practically the same floor and machinery loads. The vibration in the concrete building is very noticeably less than in the steel-frame building, in fact, it can hardly be detected.

In the Robert Gair Company Building, in Brooklyn, there is a 16-ton embossing machine set on a 3 by 6-ft. base in the middle of a bay on the seventh floor. No deflection has occurred in the beams, and, when the machine is in operation, no vibration is perceptible, although the working loads assumed for this building were only 200 lb. per sq. ft.

Answering Mr. Miller's observations on the value of unit systems in reinforced concrete construction, the chief objection to them at present is the additional cost, which must be paid by the owners. Unit frames may relieve the architect or engineer of some anxiety and responsibility, but it is admitted that most of the important work Mr. Turner. in the United States has been done with the loose-bar system; and, with a proper organization, loose bars, so-called, can be placed and secured in the work with absolute reliability. The owner looks for results, and should certainly be entitled to the difference in cost between buildings constructed with loose-bar systems and with unit systems.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

CALVIN EASTON BRODHEAD, M. Am. Soc. C. E.*

Died April 29th, 1907.

Calvin E. Brodhead was born in Pike County, Pennsylvania, on December 27th, 1846. His family moved to Mauch Chunk, Pennsylvania, in 1851. He attended school at what was known at the time as Park Seminary, and at St. Mark's Parish School, where Felix Ansart was Principal. During vacation periods, he worked in the blacksmith shop of N. Remmel and Company, who repaired cars for the old Beaver Meadow Railroad. When the survey was made for the railroad from Bethlehem to Bath, Pennsylvania, about 1862, he found employment on the engineer corps, and chose engineering as his profession. After the great flood of 1862 in the Lehigh River, which destroyed the canal above Mauch Chunk, he entered the service of the Lehigh Valley Railroad, locating the line over Wilkes-Barre Mountain, between Penn Haven and White Haven. On this work he met the late Sidney Dillon, F. Am. Soc. C. E., and a friendship was formed which lasted until Mr. Dillon's death.

After the Lehigh Valley Railroad was opened to Wilkes-Barre, Mr. Brodhead moved farther up the line to what was known as the Pennsylvania-New York Canal and Railroad. About 1871 he was transferred from Wilkes-Barre to Bethlehem, Pennsylvania, and, as Principal Assistant Engineer, under the late Robert H. Sayre, Chief Engineer, commenced locating the Easton and Amboy Railroad. He remained with the Lehigh Valley Railroad until 1877. During the building of this line (Easton and Amboy) the construction of the Musconetcong Tunnel was directly under Mr. Brodhead's charge.

From 1877 until 1883 he was engaged in the lumber business, and then he formed a partnership with Lafayette Lentz, of Mauch Chunk, John Byron and Daniel C. Hickey, of Mt. Vernon, New York, and engaged in the contracting business. The first contract of the new firm was for about ten miles of very heavy work on the Southern Pennsylvania Railway in Fulton County, Pennsylvania. In 1885 the firm secured the contract for the Vosburg Tunnel for the Lehigh Valley Railroad. In 1887 the firm of Brodhead and Hickey succeeded Lentz and Company, and while connected with this firm Mr. Brodhead was engaged on several large undertakings, notably the Palisade Tunnel for the New York, Susquehanna and Western Railroad, and a portion

*Memoir prepared by F. H. Clement, M. Am. Soc. C. E.

of the Pittsburg, Bessemer and Lake Erie Railroad, and the Lehigh Valley Railroad. After the death of Mr. Hickey, in 1894, the firm name was changed to C. E. Brodhead and Brother, and subsequently to the Brodhead Construction Company, under which name the firm continued work until Mr. Brodhead's death. It was in the contract business that Mr. Brodhead spent the most active part of his life, and in that he was best known and most successful. He was a man of quick ideas, and was a born locating engineer, in which capacity he was frequently called in consultation.

Mr. Brodhead continued to be interested for many years in the coal and lumber business, having large interests in Kentucky. He was twice married, and three children by his first wife survive him.

Mr. Brodhead was elected a Member of the American Society of Civil Engineers on February 21st, 1872.

GEORGE THOMAS NELLES, M. Am. Soc. C. E.*

DIED NOVEMBER 15TH, 1907.

George Thomas Nelles, son of George W. Nelles and Virginia Hobbs Nelles, was born on April 15th, 1856, in Muscatine, Iowa.

His boyhood was spent in Leavenworth, Kansas, to which place his parents moved in the summer of 1857. Mr. Nelles prepared for college in the private school of the Reverend (now Bishop) John Mills Kendrick, and was graduated from the Rensselaer Polytechnic Institute with the degree of C. E. in June, 1877.

After a few months' work as instrumentman with the United States Engineer Corps at Leavenworth, he entered the service of the Kansas City, St. Joseph and Council Bluffs Railway, as Assistant Engineer in charge of surveys and relocation.

In the summer of 1878 he re-entered the Government service, as United States Assistant Engineer, on the Missouri River improvement, having in charge at various periods the work at Atchison, St. Joseph, and Leavenworth, until the spring of 1883 when he was elected City Engineer of Leavenworth, Kansas. Entering upon his duties at a time when the city was growing rapidly, he directed much public work, supervising, during his term of office, the expenditure of more than $1 250 000 in grading and paving streets and constructing sewers, culverts, and bridges.

During his term of six years as City Engineer, Mr. Nelles was also Consulting Engineer for the Western Home for Disabled Volunteer Soldiers; Chief Engineer of the Riverside Coal Company; Chief En-

*Memoir prepared by F. E. Bissell, M. Am. Soc. C. E.

gineer of the Leavenworth Rapid Transit Company; and Chief Engineer of the East Omaha Land Company.

At the organization of the Nebraska and Colorado Stone Company, in 1889, Mr. Nelles became its Secretary and Manager. The company operated quarries in Nebraska and Colorado, contracting not only to furnish stone, but, also, in many cases, for the complete erection of the structure.

Severing his connection with the Stone Company in 1891, he entered the general contracting business, constructing sewers, pavements, bridges, water-works and river and harbor improvements. The largest and most important contracts handled during the four years he spent in this work were the construction of the sewers in Denver, Colorado, and the harbor improvements in the Mississippi River at St. Louis, Missouri.

In the spring of 1895 Mr. Nelles again entered the Government service as U. S. Assistant Engineer, on the Tennessee River improvement at Chattanooga, Tennessee. During his six years of service on the Tennessee River and its tributaries, many important and difficult problems presented themselves. He made a careful study of the construction of locks and dams under the conditions of fluctuating velocity of current and volume of discharge which there prevail. His reports on all subjects assigned to him were always exceedingly full and complete. He prepared detailed tables showing the cost of construction of the lift and guard locks at Colbert Shoals, Alabama. He investigated the discharge of the Tennessee River, checking the formulas with the actual measured velocities, and determining for this stream the value of n, in Kutter's formula. His solution of the problem of the effect of a dam on a submerged discharge, and on the surface level of the upper pool, reached in his study of projects for the improvement of that part of the Tennessee River known as the "Suck," is a material addition to engineering knowledge.

Mr. Nelles studied the conditions on the French Broad River, and made plans for widening and deepening the channel; he examined and reported on the necessity of making any improvement of Powells River; made plans and estimates for the low-water improvement of the Hiawassee, Little Tennessee, and Clinch Rivers, and also reported on the feasibility of making improvements on the Holston River.

The same careful attention to details, and a comprehensive consideration of all the component parts of the subject, characterize each of these reports. They show that rare combination, complete theoretical knowledge and practical ability.

In June, 1901, Mr. Nelles was transferred to Cleveland, Ohio, as U. S. Assistant Engineer in charge of the improvements of the harbors on Lake Erie at Cleveland, Lorain, and Fairport. The same thoroughness and attention to detail, combined with indomitable energy and great administrative ability, characterized his work there.

The earnestness with which he worked, the ability which he brought to the work, and the honesty of his dealings, combined with his cheerful disposition, made him a very companionable man, both socially and professionally.

His health began to fail in 1903. Two surgical operations failed to give more than temporary relief, and he died at Cleveland, Ohio, on November 15th, 1907.

On February 15th, 1884, Mr. Nelles was married to Miss Lena Ralston, who, with one son, survives him.

Mr. Nelles was a Member and a Director of the Civil Engineers' Club of Cleveland. He was elected a Member of the American Society of Civil Engineers on October 3d, 1888, and contributed to the *Transactions* a discussion* on the paper by the late George W. Rafter, M. Am. Soc. C. E., entitled "On the Flow of Water over Dams;" and also a discussion† on the paper by the late R. C. McCalla, M. Am. Soc. C. E., entitled "Improvement of the Black Warrior, Warrior, and Tombigbee Rivers, in Alabama."

HERBERT FRANKLIN NORTHRUP, M. Am. Soc. C. E.‡

Died January 21st, 1908.

Herbert Franklin Northrup, born on a farm near Shoreham, Vermont, on October 9th, 1850, was the youngest child of Nazro and Mary Hawes Northrup.

After attending the village school he prepared for college in Kimball Union Academy, and entered Middlebury College, Vermont, in the class of '73. He next taught mathematics and English for two years at a boys' school in Flushing, Long Island. He then took a graduate course in engineering, in Sheffield Scientific School, Yale, in the class of '78.

His first engineering engagement was upon the Lake Champlain breakwater at Swanton, Vermont, in 1878, and in 1879 he was engaged on railroad maintenance work at Salem, Massachusetts. In the spring of 1880 he entered the employ of the Texas Pacific Railroad as Assistant Engineer of construction, and was located at Fort Worth, Texas. He continued in the employ of that company, in responsible positions, until the completion of its construction in 1885.

On February 2d, 1882, he was married to Miss Cornelia F. Allan, of New Haven, Connecticut.

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* *Transactions*, Am. Soc. C. E., Vol. XLIV, p. 359.

† *Transactions*, Am. Soc. C. E., Vol. XLIX, p. 284.

‡ Memoir prepared by J. J. McVean, M. Am. Soc. C. E.

He entered the employ of the Missouri Pacific Railroad Company in 1885, as Assistant Engineer of construction in Missouri and Kansas, and in September, 1886, he engaged with W. V. McCracken and Company, Contractors, as Chief Engineer on the construction of railroads in Ohio and Indiana. From August to November, 1887, he was engaged as Locating Engineer on the Duluth, South Shore and Atlantic Railway, in the northern peninsula of Michigan.

In November, 1887, the writer engaged him as engineer in charge of preliminary and location surveys for the Chicago and West Michigan Railway, from Baldwin to Traverse City, Michigan, 75 miles, which was finished in June, 1888. He was then engaged until June, 1889, upon some construction work in the East, when he again returned to take charge of the construction of the road from Baldwin to Traverse City, following which he had charge of the location and construction of an extension of about 90 miles from Traverse City to Petoskey, Michigan, which was completed in 1893. In 1893 and 1894 he was engaged with the Detroit, Bay City and Alpena Railroad, and from 1895 to 1901 was in private practice and City Engineer of Traverse City, Michigan, and designed a proposed water supply for that city. During this time he also located several miles of road for the Lake Superior and Ishpeming Railroad Company.

In 1902 he entered the employ of the Cleveland, Cincinnati, Chicago and St. Louis Railroad Company, in charge of a residency on the relocation and construction of its line for double track, and reduction of grades and curvature, where he had charge of some very difficult and heavy work, especially the construction of several large-span concrete arches.

In May, 1905, he formed a partnership with the writer, as Consulting Engineers, with office at Grand Rapids, Michigan, where he was engaged until his death.

Mr. Northrup was beloved by all who knew him. He was of a very modest and retiring disposition, amiable, a staunch friend, and a thoroughly honorable business man. Quiet and even-tempered, honest in all his dealings, he had not only the entire confidence of his employers, but also the love and friendship of his assistants.

One of his many assistants, now occupying a responsible position with the City of Buffalo, says "I knew him as a gentleman and an engineer, and nothing can be added to that. His even temper and kindly ways always left a pleasant recollection."

His death was very sudden and unexpected; after a severe fall on an icy sidewalk, he was attacked with prostatitis, necessitating an operation from which he did not rally.

Mr. Northrup was a Royal Arch Mason, a member of the Delta Kappa Epsilon College Fraternity, and was elected a Member of the American Society of Civil Engineers on January 6th, 1892.

WILLIAM ROBERTS, Assoc. Am. Soc. C. E.*

DIED DECEMBER 28TH, 1907.

William Roberts was born in Watertown, Massachusetts, on March 25th, 1835. He was a son of John Roberts, a descendant of one of the old Boston families. His parents moved to Waltham soon after his birth, and he attended the Waltham Public Schools, the private school of Daniel French, and the Allen School of Newton.

He entered the employ of the Boston Manufacturing Company, in the machine shop, as a start toward the development of his mechanical genius. Obtaining permission from the Fitchburg Railroad, he ran an engine from Waltham to Boston a number of times. He then went to Virginia, where he studied at the establishment of the Norfolk Manufacturing Company. When very young he entered the United States Navy. He was Third Assistant Engineer under Commodore Perry when he opened the Ports of Simoda and Hakodadi, in Japan, and served on the *Michigan,* on the Great Lakes in 1856, and on the steam frigate *Roanoke,* on the Coast of Central America in 1857. He was one of the officers on the steamer *Fulton* which captured Walker, the filibuster, in 1858, and he served on the *Memphis* on the Paraguay expedition in 1859.

In July, 1858, Mr. Roberts was promoted, becoming Second Assistant Engineer, and one year later he was made First Assistant. He resigned in September, 1859, but, in response to his country's call, reenlisted in the Navy in April, 1861. In 1863 he became Chief Engineer.

During the attacks on the forts and batteries at Pensacola Bay, in 1861, he was on the frigate *Niagara;* the steam sloop *Housatonic* carried him to a point off Charleston, in 1862, when she drove two iron-clad rams into port. He was attached to the frigate *Niagara* repairing at Charlestown Navy Yard, during 1863 and 1864.

After his retirement from the Navy he returned to Roberts' Crossing, Waltham, and joined his father in the manufacture of paper, and, even after his father's death, he carried on the business under the firm name, John Roberts and Son.

The manufacture of roofing paper was the principal product of the mill until his ever-active mind turned to the then new article, asbestos, and his mill was the first to produce asbestos fire-proof paper, the secret of the process being held by him for many years.

He declined the acceptance of public office, notwithstanding the many entreaties on the part of his friends. The only State positions he held were Commissioner on Prisons, and Representative to the

*Memoir prepared by Sumner Milton, Esq.

General Court. He was a staunch Republican, and was sent as a delegate to the State Convention for many years. He was a Member of the Waltham Board of Cemetery Commissioners, and a Director of the Waltham National Bank.

Mr. Roberts was a life member of Monitor Lodge, A. F. and A. M., and Post 29, G. A. R., of Waltham. He belonged to the Military Order of the Loyal Legion of the United States, also the American Society of Mechanical Engineers.

"He serves God well, who serves his creatures," truly speaks the life of William Roberts. Never was he known to refuse to help a worthy person or project. Many leave public bequests and are thought generous, but Mr. Roberts' method was to give continuously; and, as was his nature, quiet, just, liberal, honest, and philanthropic, so was his giving, and there are many individuals and institutions who miss his beneficence.

Mr. Roberts was an interesting conversationalist, having toured the world. He was especially interested in the ocean, and crossed the Atlantic on all the finest new steamers, his knowledge of mechanical engineering enabling him to note all the latest improvements in the engines. It was difficult, indeed, to ask a question on country or product on which Mr. Roberts could not give valuable information, and in such a simple manner that a child could enjoy his talk.

On October 27th, 1879, Mr. Roberts married Eva C., daughter of Hon. Gideon Haynes, and their home was always at Waltham. His married life was one of devotion, and it would be difficult to decide whether the palm should be given to him or to his companion in life.

William Roberts was elected an Associate of the American Society of Civil Engineers on June 4th, 1884.

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INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

CURVE RESISTANCE IN WATER PIPES.*

By Ernest W. Schoder, Assoc. M. Am. Soc. C. E.

The object of this paper is to present the results of some measurements which seem to throw new light on a subject that awakened considerable interest and discussion among the members of this Society some seven years ago.

A. V. Saph, Assoc. M. Am. Soc. C. E., and the writer, in their discussion of the paper† by Messrs. Williams, Hubbell and Fenkell, presented results of experimental studies on the flow of water in a line of 2-in. brass pipe with 180° curves, or return bends, of varying radii of curvature. It seemed difficult to harmonize these results‡ with one of the principal conclusions in the paper, namely:

"That curves of short radius, down to a limit of about 2½ diameters, offer less resistance to the flow of water than do those of longer radius."§

In their closing discussion, this conclusion is further defined by Messrs. Williams, Hubbell and Fenkell in these words:||

"In a given length of pipe consisting of two tangents joined by a curve of 90°, the loss of head will decrease as the radius of the curve

* This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

† *Transactions*, Am. Soc. C. E., Vol XLVII, "Experiments at Detroit, Mich., on the Effect of Curvature upon the Flow of Water in Pipes."

‡ Pages 319-322 of same.

§ Page 191 of same.

|| Page 348 of same.

is decreased, to a limit of about $2\frac{1}{2}$ diameters, and will increase as the radius is increased above that limit, the total length remaining the same."

Also:

"* * * for the range of these experiments, at least: To unite two points on two tangents intersecting at 90°, and equally distant from their intersection, by a pipe line consisting of portions of the two tangents and a curve of 90°, the line of least hydraulic resistance will be one in which a curve of about $2\frac{1}{2}$ diameters radius is used."

At the time, no critical comparison was made between the results of the loss-of-head measurements on 180° 2-in. brass pipe curves and those on 90° curves in 12-in., 16-in. and 30-in. cast-iron water mains. This seemed hardly justifiable with the limited data available. However, Mr. Saph and the writer did point out* the probability of large piezometric errors involved in the Detroit Experiments.

On this account, and because of other unsatisfactory features to be mentioned later, the writer felt the desirability of making further experiments in an attempt to clear up the situation. He is now able to present some new evidence that tends to limit the general application of the results of the Detroit Experiments, if it does not essentially contradict them.

The chief experimental problem that the writer set for himself was to find the losses of head due to each of a number of 90° curves of different radii, for as great a range of velocities as the available facilities would permit. Given two long runs of straight pipe connected by a 90° curve, one part of the problem is to measure the loss of head in a portion of the pipe line including the curve. The portion must be long enough so that the full effects on the flow of the water due to the curve are realized in the down-stream straight run of pipe. Another part of the problem is to find the loss of head in the same straight pipe used with the curves when it is uninfluenced by effects of curvature (or other disturbance) and when it is in the same condition as when used with the curves. This last is a difficult matter in any case with pipes of large size, as will appear presently. Evidence will be given to show, also, that it is often practically impossible, where the pipe lines are fixed in place underground.

The straight pipe chosen for the experiments first to be described was 6-in. wrought iron. Before the curve experiments were made, a

* *Transactions*, Am. Soc. C. E., Vol. XLVII, 1902, pages 317-321.

straight line composed of six pipe lengths was set up in the pipe alley alongside the large canal of the Cornell University Hydraulic Laboratory. The pipes were flange-connected. The total length of 6-in. pipe was 122 ft. The loss of head was measured in a length of 99.33 ft., a length of 20.04 ft. being allowed up stream from the first piezometer for the disturbances due to entry from a 12-in. header to die down. The piezometers consisted each of two diametrically opposite holes in the pipe wall into which $\frac{1}{8}$-in. T-handle cocks were screwed. A three-way connection served to join the short hoses from these cocks and the single long hose to the gauge. A water differential gauge was used to measure the loss of head. A calibrated concrete measuring tank, of 500 cu. ft. capacity, received the discharged water from a 4-in. pipe with a 4-in. regulating valve through which the 6-in. line discharged. An instantaneous diverter deflected the discharge into the tank or allowed it to run to waste, as desired. The measurements lasted from 5 to 10 min., the time being accurately taken. Fig. 1, Plate XLVI, is a photograph showing the pipe line. The results of these measurements are given in Table 2, where data from later straight pipe experiments are given also.

After this the pipes were marked, disconnected, and transported to the bottom of the Fall Creek Gorge, in Ithaca, near the hydroelectric plant of Cornell University. Here the curve experiments were performed. Fig. 2, Plate XLVI, is a photograph of this plant and Fig. 1 is a plan of the pipe line.

The pipes composing the experimental portion of the line were the same, and were set up in the same order as when tested as a straight pipe line. The curves were placed between Pipes Nos. 1 and 2, the two up-stream pipes in the experiments described above, and as shown by Fig. 1. The down-stream tangent portion thus consisted of Pipes Nos. 2, 3, 4, and 5. Pipe No. 6 was not used for the curve experiments because this would have brought the end of the pipe line out into the creek.

At the discharge end of Pipe No. 5 there was attached a brass nozzle for measuring the velocity in the pipe line. This nozzle is shown by Fig. 3, Plate XLVI. It had been calibrated previously by tank measurements in the Hydraulic Laboratory. The average discharge coefficient from 40 experiments, with pressure heads at the base of the nozzle ranging from 1.649 to 50.208 ft., was found to be 0.988.

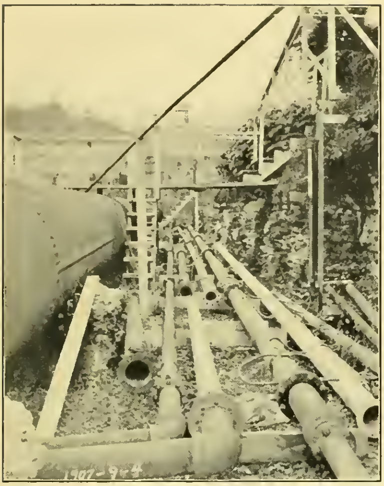


FIG. 1.—SIX-INCH PIPE LINE (AT THE RIGHT) AS SET UP FOR FIRST STRAIGHT-PIPE EXPERIMENTS.

FIG. 2.—CORNELL UNIVERSITY HYDRO-ELECTRIC POWER PLANT, AND 6-IN. PIPE LINE FOR CURVE EXPERIMENTS.

FIG. 3.—NOZZLE AT END OF 6-IN. PIPE LINE.

FIG. 4.—SIX-INCH, 90° CURVES, NOS. 1 TO 12.

The lengths of the pipes, from face to face of flanges, are given in Fig. 1; the following are the inside diameters—the means of four measurements, two at each end:

Pipe No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 6.
6.106 in.	6.086 in.	6.102 in.	6.078 in.	6.072 in.	6.083 in.

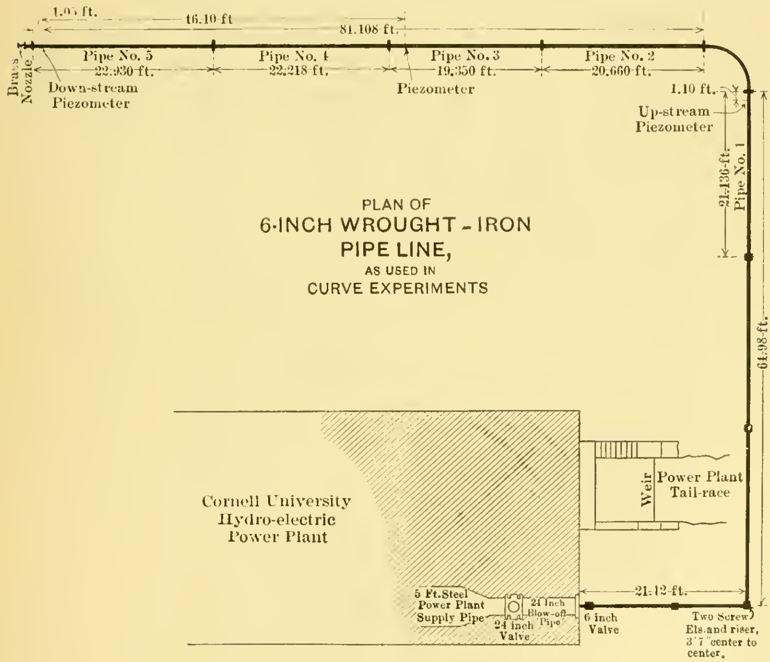


Fig. 1.

The Curves.—The curves used were made to order, except that Nos. 10 and 12, respectively, were standard "long sweep" and "short turn" 6-in., cast-iron, flanged, 90° elbows. Curves Nos. 1 to 6, inclusive, were bent from 6-in. wrought-iron pipe. Curves Nos. 7 to 12, inclusive, were of cast iron. All were 90° curves; and all were left uncoated.

The wrought-iron pipe curves had about 6 in. of straight pipe at each end. The ends had been threaded to receive standard flanges. The cast-iron curves were flanged, faced, and drilled complete, ready for setting up. Fig. 4, Plate XLVI, is a photograph of the curves stacked against the power-plant wall.

TABLE 1.—DIMENSIONS OF CURVES.

No. of curve.	Material.	Radius, in feet.	Radius, in pipe diameters.	A, in feet.	B, in feet.	$A + B$, in feet.	Length on center line, in feet.	Diameter, mean of four end caliperings.
1	Wrought iron.	10.00	20	10.54	10.52	21.06	16.77	6.09 in.
2	" "	7.50	15	8.04	8.02	16.06	12.84	6.18 "
3	" "	5.00	10	5.59	5.57	11.16	9.01	6.16 "
4	" "	4.00	8	4.54	4.52	9.06	7.34	6.11 "
5	" "	3.00	6	3.60	3.58	7.18	5.89	6.11 "
6	" "	2.50	5	3.05	3.10	6.15	5.08	6.09 "
7	Cast iron.	2.00	4	2.25	2.25	4.50	3.64	5.91 "
8	" "	1.50	3	1.75	1.75	3.50	2.86	5.95 "
9	" "	1.08	2.16	1.50	1.50	3.00	2.54	5.91 "
10	" "	0.95	1.90	1.08	1.08	2.16	1.75	5.91 "
11	" "	0.88	1.76	2.00	2.00	4.00	3.62	5.95 "
12	" "	0.67	1.34	0.67	0.67	1.34	1.05	5.93 "

The dimensions of the curves are given in Table 1, in which reference is made to the dimensions indicated in Fig. 2. The dimensions

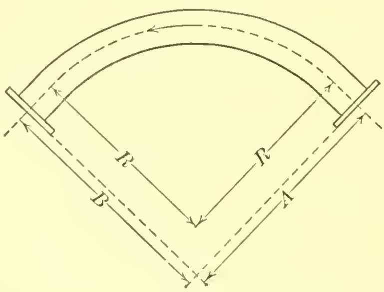


FIG. 2.

in Table 1 are for the curves as placed in the pipe line, the wrought-iron bends having screw flanges attached. The distances from the faces of the flanges to the ends of the wrought-iron curves were as follows: Curves Nos. 1, 2, 3, and 4, up-stream end, $\frac{1}{4}$ in., down-stream end, $\frac{15}{32}$ in.; Curve No. 5, up-stream end, $\frac{1}{2}$ in., down-stream end, $\frac{3}{4}$ in; Curve No. 6, up-stream end, 1 in., down-stream end, $\frac{5}{8}$ in. The inside diameters of Curve No. 6 were measured also at points $22\frac{1}{2}°$, 45° and $67\frac{1}{2}°$ from the up-stream end. At these points, $\frac{1}{4}$-in. taps for the insertion of a Pitot tube had been made on both the vertical and horizontal diameters. The measurements were as follows: at $22\frac{1}{2}°$, 5.92 and 6.09 in.; at 45°, 6.08 and 6.08 in.; at $67\frac{1}{2}°$, 5.95 and 6.15 in.

Curves Nos. 9 and 11 were made in error, the foundry making curves of quite short radius, with straight portions at each end, so that the dimensions from the center to the face were 18 and 24 in., respectively, where the writer's order called for curves with the same radii as the center-to-face dimensions. Curves Nos. 7 and 8 were made later, to correct this error.

The experimental pipe line was arranged with the idea of keeping all conditions the same except for the introduction of the several curves. Consequently, the flange joints in the portion down stream from the curves were not disturbed throughout the experiments. The entire length of 85 ft. of 6-in. wrought-iron pipe, together with the nozzle, was shifted bodily when a new curve was placed in the line. This was rather vigorous exercise for two men, but it was accomplished by a judicious tilting of the wooden horses supporting that part of the pipe line. The portion up stream from the curves was left unchanged during the experiments.

The same up-stream piezometer was used for the curve experiments as for the first straight-pipe experiments. The down-stream piezometer was a new one, of the same type, placed 1.05 ft. up stream from the down-stream end of Pipe No. 5 which adjoined the nozzle. A similar intermediate piezometer was placed 2.00 ft. up stream from the down-stream end of Pipe No. 3.

At first it was assumed that some effect of the curves might extend 100 or more diameters down stream in the straight pipe beyond.* The losses of head in all the curve experiments were measured between the piezometer just up stream from the curves and the piezometer just up stream from the nozzle, distant 168 diameters down stream from the curves. These two piezometers were connected to the two branches of a differential U-tube mercury gauge by lines of $\frac{3}{8}$-in. or $\frac{1}{2}$-in. three-ply rubber tubing. The nozzle piezometer was connected to one branch of an 8-ft. mercury U-tube gauge, the other branch of which was open to the atmosphere. These gauges were provided with blow-off cocks for the removal of air from the gauge and connections.

In experimenting, the 6-in. valve was first opened wide to establish a swift flow through the pipe line. The gauge hoses connected to the piezometers were allowed to run for a while before connecting them to the gauges, being pinched near the gauge end while connecting up.

* *Transactions*, Am. Soc. C. E., Vol. XLVII, 1902, p. 302.

Finally, the pet-cocks were opened, and all traces of air were blown off. Then the gauge readings were recorded and checked. The 6-in. valve was then closed a little, and the readings were taken again as soon as the flow had become settled. In this way ten or twelve runs were made, the last being generally a repetition of the first run with the valve wide open. If there was time, a new curve was then placed in the line. Otherwise, the change was deferred until another day, in which case one or more check runs were made at the high velocities before removing the curve and substituting another. For each set-up, the level of the center of the nozzle with reference to the nozzle mercury gauge scale was determined. The alignment of the down-stream tangent was corrected after each change of curves.

When the work on the twelve curves had been finished, Curve No. 6 was replaced in the line to find whether or not any appreciable change in the condition of the pipes had occurred during the experiments. Then a number of Pitot-tube studies were made to determine the conditions of flow at the piezometers and in the curve. Later, a 6-in. screw elbow, of the ordinary steam-fitting type, was placed in the line, and a series of measurements made on it as for the other curves. Finally, the loss of head was measured in the portion (46.10 ft. long) of the experimental section farthest down stream, Pitot tube traverses having shown normal flow in the pipe 38 ft., or 76 diameters, down stream from the curves. Hence, this portion represented straight pipe unaffected by curvature effects.

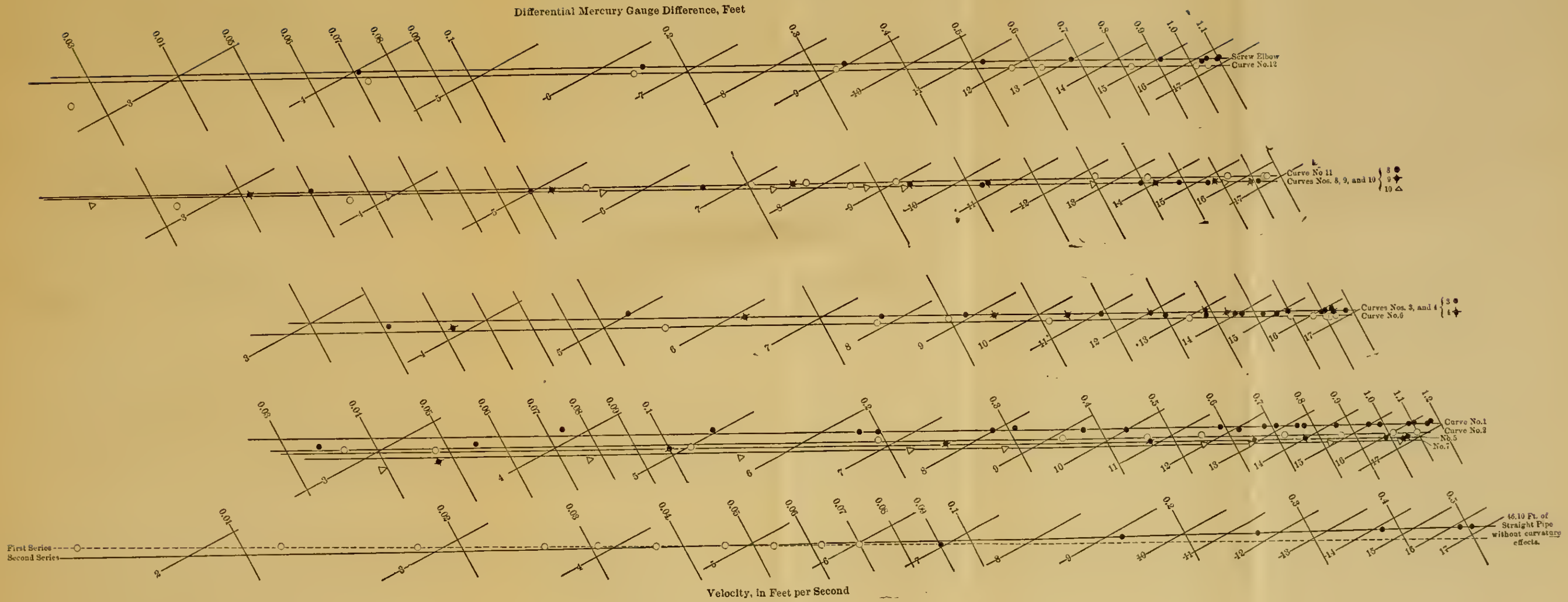
The results of these loss-of-head measurements are given in Table 2. For simplicity, there is given only the mean velocity in the pipe line as deduced from the nozzle mercury pressure gauge indications. The differential mercury gauge differences are taken directly from the checked subtractions in the field notebook. These differences, multiplied by 12.57, would give the loss of head, in feet of water, the specific gravity of mercury being taken as 13.57.

To illustrate the calculations involved in obtaining the velocity in the pipe line from the observations in the mercury pressure gauge the following is a sample:

In Experiment No. 1, Curve No. 1, October 10th, 1907, the gauge readings were: left 0.270, right 7.900. The difference is 7.630 ft. The center of the nozzle-tip level was at 1.81 on the gauge scale. Hence the top of the left mercury column was 1.54 ft. below the center of the nozzle tip. The pressure head at the base of the nozzle,

6-INCH PIPE, 90° CURVE EXPERIMENTS.

Logarithmic Plotting of Observed Differential Mercury Gauge Differences, with Mean Velocities in the Pipe line, showing the Losses of Head in the Experimental Length as varied only by the introduction of the several Curves. Also the corresponding Plotting for Straight Pipe without Curvature Effects.



therefore, was $7.630 \times 13.57 - 1.54 = 102.1$ ft. The diameter of the nozzle tip was 2.738 in., and the diameter of the nozzle base was 6.123 in., the ratio of areas being 1 : 5. Theoretically, the nozzle-tip velocity is $C \sqrt{\frac{1}{1 - \left(\frac{1}{5}\right)^2}} \sqrt{2gh} = C \times 8.19 \sqrt{h}$, where h is the pressure head at the nozzle base. As stated above, the coefficient C, for this nozzle had been found to be 0.988 by experiment. The mean diameter of the pipe line is 6.084 in. Therefore the mean velocity in the experimental portion of the pipe line was: $V = 0.988 \times 8.19 \times \left(\frac{2.738}{6.084}\right)^2 \sqrt{h} = 1.639 \sqrt{h}$. This gives, for this experiment, $V = 1.639 \times \sqrt{102.1} =$ 16.56 ft. per sec.

The results given in Table 2 were plotted logarithmically on 10-in. base paper. Plate XLVII shows these plottings assembled.

Now, in order to study the effects of the curves, it is necessary to select a uniform basis of comparison. Thus the experiments were made by introducing the various curves between two fixed lengths of pipe. For this case, the observed losses of head may be compared with each other at once by using Table 2 or Plate XLVII; but such a basis takes no account of the comparative lengths of the several pipe lines, nor does it consider whether or not the same two points are joined by the combinations of straight pipes and curves.

The writer has chosen two bases of comparison. In one, all cases are reduced to equal lengths on the center lines of straight pipes and curves. In the other, all cases are reduced to what they would be if two fixed points had been connected by two straight pipe lines with the various 90° curves between them.

For both these cases it is necessary to know the loss of head in the straight pipe when it is unaffected by curvature. Two series of experiments already described give this information.

Turning to Plate XLVII, it is seen that the results of the first straight-pipe experiments do not agree with the later ones. Each series gives a good straight line, but the two lines have different slopes and represent different laws of flow. The equations of these lines, reduced so as to represent loss of head in feet of water per foot length of pipe, are

For the first series, $H = 0.000\,669\ V^{1.87}$

For the later series, $H = 0.000\,598\ V^{1.91}$

TABLE 2.

Date, 1907.	Number of Experiment.	Observed differential mercury-gauge difference, in feet.	Velocity in pipe, in feet per second.	Temperature of water, in degrees, Fahrenheit.
CURVE No. 1.				
Oct. 10..	1	1.155	16.56	49
	2	1.085	16.00	
	3	0 990	15.27	
	4	0.772	13.41	
	5	0.708	12.81	
	6	0.625	12.06	
	7	0.380	9.29	
	8	0.285	7.96	
	9	0.198	6.59	
	10	0.118	4.93	
	11	0.055	3.46	
	12	1.158	16.58	
Oct. 12..	13	1.169	16.58	47
	14	1.104	16.10	
	15	0.956	14.95	
	16	0.858	14.13	
	17	0.756	13.24	
	18	0.680	12.53	
	19	0.590	11.64	
	20	0.436	9.96	
	21	0.308	8.20	
	22	0.187	6.38	
	23	0.074	3.82	
	24	0.034	2.67	
CURVE No. 2.				
Oct. 12..	1	1.097	16.65	47
	2	1.019	16.04	
	3	0.718	13.35	
	4	0.859	14.73	
	5	0.549	11.55	
	6	0.460	10.62	
	7	0.350	9.18	
	8	0.196	6.76	
	9	0.107	5.02	
	10	0.048	3.29	
	11	0.036	2.81	
	12	1.096	16.65	
CURVE No. 3.				
Oct. 12..	1	1.065	16.66	48
	2	0.994	16.11	
	3	0.859	14.95	
	4	0.770	14.05	
	5	0.682	13.28	
	6	0.600	12.39	
	7	0.488	11.09	
	8	0.318	8.85	
	9	0.242	7.69	
	10	0.110	4.99	
	11	0.051	3.45	
	12	1.065	16.66	
Oct. 14..	13	1.062	16.65	46
	14	1.005	16.16	
	15	0.892	15.18	
	16	0.750	13.91	
CURVE No. 4.				
Oct. 11..	1	1.071	16.62	46
	2	1.034	16.30	
	3	0.824	14.59	
	4	0.682	13.15	
	5	0.572	12.01	
	6	0.349	9.29	
	7	0.158	6.10	
	8	0.062	3.87	
	9	0.438	10.46	
	10	1.070	16.63	
Oct. 15..	11	1.070	16.63	45
	12	1.024	16.27	
	13	0.732	13.67	

TABLE 2 (*Continued*).

Curve No. 5.

Date, 1907.	Number of Experiment.	Observed differential mercury-gauge difference, in feet.	Velocity in pipe, in feet per second.	Temperature of water, in degrees, Fahrenheit.
Oct. 15..	1	1.053	16.63	45
	2	0.985	16.09	
	3	0.762	14.05	
	4	0.642	12.85	
	5	0.460	10.80	
	6	0.240	7.63	
	7	0.100	4.86	
	8	0.048	3.42	
	9	1.052	16.63	
Oct. 17..	10	1.045	16.61	

Curve No. 6.

Date, 1907.	Number of Experiment.	Observed differential mercury-gauge difference, in feet.	Velocity in pipe, in feet per second.	Temperature of water, in degrees, Fahrenheit.
Oct. 17..	1	1.036	16.65	47½
	2	1.004	16.40	
	3	0.893	15.41	
	4	0.644	13.04	
	5	0.298	8.67	
	6	0.410	10.36	
	7	0.238	7.78	
	8	0.121	5.53	
	9	0.046	3.41	
	10	1.028	16.61	
Oct. 19..	11	1.026	16.60	
Oct. 26..	12	1.034	16.61	42
	13	1.015	16.45	
	14	0.963	16.01	

Curve No. 7.

Date, 1907.	Number of Experiment.	Observed differential mercury-gauge difference, in feet.	Velocity in pipe, in feet per second.	Temperature of water, in degrees, Fahrenheit.
Oct. 19..	1	1.024	16.61	47½
	2	0.819	14.80	
	3	0.631	12.90	
	4	0.544	11.94	
	5	0.287	8.58	
	6	0.212	7.31	
	7	0.123	5.62	
	8	0.076	4.38	
	9	0.039	3.20	
	10	1.025	16.62	

Curve No. 8.

Date, 1907.	Number of Experiment.	Observed differential mercury-gauge difference, in feet.	Velocity in pipe, in feet per second.	Temperature of water, in degrees, Fahrenheit.
Oct. 21..	1	1.008	16.60	43
	2	0.858	15.26	
	3	0.782	14.50	
	4	0.694	13.64	
	5	0.331	9.26	
	6	0.419	10.50	
	7	0.172	6.56	
	8	0.100	4.99	
	9	0.050	3.44	
	10	1.016	16.62	

Curve No. 9.

Date, 1907.	Number of Experiment.	Observed differential mercury-gauge difference, in feet.	Velocity in pipe, in feet per second.	Temperature of water, in degrees, Fahrenheit.
Oct. 22..	1	1.020	16.63	42
	2	0.981	16.40	
	3	0.881	15.40	
	4	0.730	14.00	
	5	0.426	10.55	
	6	0.230	7.58	
	7	0.107	5.15	
	8	0.041	3.13	
	9	1.020	16.63	

Curve No. 10.

Date, 1907.	Number of Experiment.	Observed differential mercury-gauge difference, in feet.	Velocity in pipe, in feet per second.	Temperature of water, in degrees, Fahrenheit.
Oct. 22..	1	1.016	16.63	42
	2	0.990	16.40	
	3	0.905	15.65	
	4	0.778	14.48	
	5	0.595	12.60	
	6	0.324	9.26	
	7	0.288	8.70	
	8	0.214	7.41	
	9	0.126	5.60	
	10	0.064	3.97	
	11	0.024	2.46	
	12	1.016	16.64	
Oct. 24..	13	1.013	16.66	

TABLE 2 (*Continued*).

Date, 1907.	Number of Experiment.	Observed differential mercury gauge difference, in feet.	Velocity in pipe, in feet per second.	Temperature of water, in degrees, Fahrenheit.
CURVE NO. 11.				
Oct. 19..	1	1.050	16.59	17½
	2	0.926	15.53	
	3	0.607	12.40	
	4	0.320	8.99	
	5	0.275	8.41	
	6	0.241	7.75	
	7	0.202	7.07	
	8	0.120	5.41	
	9	0.056	3.78	
	10	0.032	2.87	
	11	1.046	16.56	
Oct. 21..	12	1.052	16.58	43
	13	0.715	13.60	
CURVE NO. 12.				
Oct. 24..	1	1.046	16.62	43
	2	1.009	16.31	
	3	0.818	14.65	
	4	0.558	12.01	
	5	0.320	8.92	
	6	0.168	6.43	
	7	0.072	4.21	
	8	0.027	2.74	
	9	0.612	12.61	
	10	1.046	16.61	
SCREW ELBOW.				
Nov. 12..	1	1.090	16.55	37
	2	1.036	16.15	
Nov. 18..	3	1.098	16.55	33
	4	1.053	16.21	
	5	0.912	15.02	
	6	0.682	12.91	
	7	0.515	11.20	
	8	0.331	8.92	
	9	0.171	6.40	
	10	0.071	4.01	
	11	1.088	16.55	

STRAIGHT-PIPE EXPERIMENTS.

First Series: Temperature of water: 68° at beginning, 70° at end.

Date, 1907.	Number of Experiment.	Observed loss of head in 99.33 ft. of pipe, in feet of water.	Corresponding differential mercury-gauge difference for 46.10 ft. of pipe.	Velocity in pipe, in feet per second.
Sept. 4..	1	1.901	0.0701	6.03
	2	1.687	0.0622	5.67
	3	1.449	0.0535	5.22
	4	1.241	0.0458	4.80
	5	1.000	0.0369	4.29
	6	0.831	0.0307	3.88
	7	0.704	0.0260	3.55
	8	0.473	0.0175	2.87
	9	0.307	0.0114	2.27
	10	0.160	0.0059	1.61

Second Series*: Temperature of water, 33° fahr.

Date, 1907.	Number of Experiment.	Corresponding values.	Observed values.	Velocity in pipe, in feet per second.
Nov 18..	1	13.70	0.506	16.56
	2	13.19	0.487	16.23
	3	10.24	0.378	14.29
	4	6.83	0.2525	11.56
	5	4.40	0.1625	9.26
	6	2.45	0.0905	6.93

* The velocities for the second series are computed for the mean diameter of the experimental section (46.10 ft. long) which was 6.075 in.

To show more clearly the magnitude of this difference, the following calculated values are given:

	Loss of head, in feet of water per foot of length:			
	$V = 3$ ft. per sec.	$V = 5$ ft. per sec.	$V = 10$ ft. per sec.	$V = 16$ ft. per sec.
First series	0.0052	0.0135	0.049	0.119
Later series	0.0050	0.0135	0.052	0.129

Thus, at the outset, comes the question which must arise in all experiments of this kind: What was the law of flow of the identical straight pipe used with the curves when unaffected by curvature, but otherwise in the same condition? On this depends the calculation of the excess loss of head caused by the curves.

There are a number of possible causes for the difference shown above:

1.—The pipe may have become rougher by rusting in the interval between the two series of experiments;

2.—Pipes Nos. 4 and 5 together may have had different hydraulic properties from Pipes Nos. 2, 3, 4, and 5 together;

3.—The different temperatures of the water may have caused a difference in loss of head.

It was intended originally to bring the straight pipes back to the hydraulic laboratory and again test them as at first for loss of head after the curve experiments had been finished, but the lateness of the season prevented this.

Later in this paper the above possible causes of differences will be discussed more fully. In order to remove any question as to mistaken judgment, the results are worked up in both ways.

The individual observations, directly, are not used in the final comparisons. From the mean lines drawn on the logarithmic diagram, where all observed values for each curve have been plotted, the gauge-difference values for velocities of 3, 5, 10, and 16 ft. per sec. have been picked off. These values are given in Table 3.

All the cases have been reduced to the length conditions existing in the set-up for Curve No. 1. For the first comparison, the observed differential mercury-gauge differences for the other curves have been increased by an amount corresponding to the additional length of straight pipe necessary to give the same length on the center line as existed between the two piezometers when Curve No. 1 was in the pipe line. For the second comparison, the added quantity corresponds to the extra length of pipe required to make the sum of the tangent distances from the point of intersection equal to that for Curve No. 1. These two cases may be called, for brevity, respectively, the equal-lengths and the two-fixed-points cases.

TABLE 3.—Differential Mercury-Gauge Differences from the Mean Lines Drawn for the Plotted Points for All the Observations, in Feet.

No. of curve.	Velocity, in Feet per Second: 3	5	10	16
1	0.0444	0.1180	0.443	1.090
2	0.0408	0.1083	0.410	1.011
3	0.0400	0.1062	0.402	0.991
4	0.0400	0.1062	0.402	0.991
5	0.0395	0.1046	0.396	0.978
6	0.0376	0.1008	0.386	0.960
7	0.0381	0.1013	0.385	0.950
8	0.0375	0.1002	0.382	0.942
9	0.0375	0.1002	0.382	0.942
10	0.0375	0.1002	0.382	0.942
11	0.0382	0.1030	0.394	0.981
12	0.0387	0.1031	0.392	0.968
Screw elbow.	0.0403	0.1077	0.413	1.030

TABLE 4.—Lengths of Straight Pipe to be Added to Reduce All Cases to Conditions of Curve No. 1.

No. of curve.	For equal lengths on center lines.	To connect two fixed points.	No. of curve.	For equal lengths on center lines.	To connect two fixed points.
2	3.92 ft.	5.00 ft.	8	13.90 ft.	17.56 ft.
3	7.75 "	9.90 "	9	14.22 "	18.06 "
4	9.42 "	12.00 "	10	15.01 "	18.90 "
5	10.87 "	13.88 "	11	13.14 "	17.06 "
6	11.68 "	14.91 "	12	15.71 "	19.72 "
7	13.12 "	16.56 "	Screw elbow.	....	20.56 "

Table 4 gives the length of straight pipe to be added for each curve for the two cases to reduce all to the conditions of Curve No. 1.

The loss of head per foot length of straight pipe is given in Table 5.

Using Tables 5 and 4, the individual corrections are calculated. The results are given in Table 6.

TABLE 5.

Velocity, in feet per second.	From the First Experiments: Mercury differential-gauge differences, in feet.	Feet of water.	From the Second Experiments: Mercury differential-gauge differences, in feet.	Feet of water.
3	0.000412	0.00518	0.000399	0.00502
5	0.001071	0.01346	0.001075	0.0135
10	0.003903	0.0491	0.00412	0.0518
16	0.00943	0.1185	0.01026	0.129

TABLE 6.—Corrections to be Added to Observed Differential Mercury-Gauge Differences to Reduce All Cases for Comparison with Conditions of Curve No. 1, in Feet.

For Equal Lengths on Center Lines: No of curve.	Velocity, in feet per second. 3	5	10	16	To Connect Two Fixed Points: No. of curve.	Velocity, in feet per second. 3	5	10	16
On Basis of First Straight-Pipe Experiments.									
2	0.0016	0.0042	0.015	0.037	2	0.0021	0.0054	0.020	0.047
3	0.0032	0.0083	0.030	0.073	3	0.0041	0.0106	0.039	0.093
4	0.0039	0.0101	0.037	0.089	4	0.0049	0.0129	0.047	0.113
5	0.0045	0.0117	0.042	0.103	5	0.0057	0.0149	0.054	0.131
6	0.0048	0.0125	0.046	0.110	6	0.0061	0.0160	0.058	0.141
7	0.0054	0.0141	0.051	0.124	7	0.0068	0.0177	0.065	0.156
8	0.0057	0.0149	0.054	0.131	8	0.0072	0.0188	0.069	0.166
9	0.0059	0.0152	0.056	0.134	9	0.0074	0.0194	0.070	0.170
10	0.0062	0.0161	0.059	0.142	10	0.0078	0.0202	0.074	0.178
11	0.0054	0.0141	0.051	0.124	11	0.0070	0.0183	0.067	0.161
12	0.0065	0.0168	0.061	0.148	12	0.0081	0.0211	0.077	0.186
					Screw elbow	0.0085	0.0220	0.080	0.191
On Basis of Second Straight-Pipe Experiments.									
2	0.0016	0.0042	0.016	0.040	2	0.0020	0.0054	0.021	0.051
3	0.0031	0.0083	0.032	0.080	3	0.0040	0.0106	0.041	0.102
4	0.0038	0.0102	0.038	0.097	4	0.0048	0.0129	0.049	0.123
5	0.0043	0.0117	0.045	0.112	5	0.0055	0.0149	0.057	0.142
6	0.0047	0.0126	0.048	0.120	6	0.0059	0.0160	0.061	0.153
7	0.0052	0.0141	0.054	0.135	7	0.0066	0.0178	0.068	0.170
8	0.0055	0.0150	0.057	0.143	8	0.0070	0.0189	0.072	0.180
9	0.0057	0.0153	0.059	0.146	9	0.0072	0.0194	0.074	0.185
10	0.0060	0.0161	0.062	0.154	10	0.0075	0.0203	0.078	0.194
11	0.0052	0.0141	0.054	0.135	11	0.0068	0.0183	0.070	0.175
12	0.0063	0.0169	0.065	0.161	12	0.0079	0.0212	0.081	0.202
					Screw elbow	0.0082	0.0221	0.085	0.211

The values in Table 6 are added to the corresponding ones in Table 3 to obtain the losses of head when the length conditions are the same as for Curve No. 1. If, from these latter losses of head, there are subtracted the losses of head in the same length of straight pipe without curvature effects, the results are the excess losses of head due to the curves. This has been done, using the values in Table 7. The units are in feet of difference on a mercury differential gauge.

TABLE 7.

Velocity, in feet per second.	From the First Experiments:		From the Second Experiments:	
	Loss per 103.02 ft. of straight pipe, the length on the center line, for Curve No 1.	Loss per 107.32 ft. of straight pipe, the sum of the tangent distances from the P. I., for Curve No. 1.	Loss per 103.02 ft. of straight pipe.	Loss per 107.32 ft. of straight pipe.
3	0.0424	0.0442	0.0411	0.0428
5	0.1104	0.1150	0.1108	0.1154
10	0.402	0.419	0.425	0.442
16	0.972	1.012	1.057	1.101

The excess losses of head, found as above described, are given in Table 8, having been reduced to feet of water by multiplying by 12.57. (On a mercury differential gauge the difference corresponds to a water difference 12.57 times as great, the specific gravity of mercury being 13.57.)

In Table 9 these excess losses of head are expressed in terms of the lengths of straight pipe that would give the same losses. In Table 10 they are expressed in terms of the velocity heads. Tables 9 and 10 have been worked out on the basis of the second series of straight-pipe experiments only. The excess losses of head, given in Table 8, are plotted on Figs. 3, 4, 5, and 6, to show the relation to the radius of curvature. The points have been connected by short straight lines, no attempt being made to draw smooth averaging curves. The results given in Table 9 have been plotted in the same manner in Figs. 7 and 8. The results are now in shape for discussion on the influence of curvature.

Fig. 5 indicates for increasing radius of curvature a decided lessening in the excess loss of head throughout the range of the experiments. Fig. 3 seems to indicate on the whole a gradual decrease as the radius of the curve increases above about 1.25 ft., or 2½ diameters. Certainly, there is no general indication of an increasing loss of head.

TABLE 8.—EXCESS LOSS OF HEAD OF STRAIGHT PIPES JOINED BY A 90° CURVE OVER STRAIGHT PIPE ALONE, IN FEET OF WATER.

For Equal Lengths on Center Lines:					To Connect Two Fixed Points:				
No. of curve.	Velocity, in feet per second.				No. of curve.	Velocity, in feet per second.			
	3	5	10	16		3	5	10	16
On Basis of First Straight-Pipe Experiments.									
1	0.025	0.096	0.52	1.48	1	0.002	0.038	0.30	0.98
2	0.000	0.026	0.29	0.96	2	−0.016	−0.016	0.14	0.58
3	0.001	0.052	0.38	1.16	3	−0.001	0.023	0.28	0.91
4	0.019	0.074	0.47	1.36	4	0.009	0.052	0.38	1.16
5	0.020	0.074	0.45	1.37	5	0.013	0.057	0.39	1.22
6	0.000	0.036	0.38	1.23	6	0.006	0.023	0.31	1.12
7	0.014	0.063	0.43	1.28	7	0.009	0.050	0.39	1.18
8	0.001	0.059	0.43	1.27	8	0.006	0.050	0.40	1.21
9	0.013	0.063	0.45	1.31	9	0.009	0.058	0.41	1.26
10	0.016	0.074	0.49	1.41	10	0.014	0.068	0.47	1.36
11	0.015	0.084	0.54	1.67	11	0.013	0.079	0.53	1.63
12	0.035	0.119	0.64	1.81	12	0.033	0.116	0.63	1.78
					Screw elbow.	0.058	0.185	0.93	2.66
On Basis of Second Straight-Pipe Experiments.									
1	0.042	0.091	0.23	0.41	1	0.020	0.033	0.01	−0.14
2	0.016	0.021	0.01	−0.08	2	0.000	−0.021	−0.14	−0.49
3	0.025	0.047	0.11	0.18	3	0.015	0.018	0.01	−0.10
4	0.034	0.070	0.20	0.39	4	0.025	0.046	0.11	0.16
5	0.034	0.069	0.20	0.41	5	0.028	0.052	0.14	0.24
6	0.015	0.033	0.11	0.29	6	0.009	0.018	0.06	0.15
7	0.028	0.058	0.18	0.35	7	0.024	0.046	0.14	0.24
8	0.024	0.055	0.18	0.35	8	0.021	0.046	0.15	0.26
9	0.026	0.059	0.20	0.39	9	0.024	0.053	0.18	0.33
10	0.030	0.069	0.24	0.49	10	0.028	0.064	0.23	0.44
11	0.029	0.079	0.29	0.74	11	0.028	0.074	0.28	0.69
12	0.049	0.116	0.40	0.91	12	0.048	0.112	0.39	0.87
					Screw elbow.	0.072	0.181	0.70	1.76

Figs. 4 and 6 show how the results are modified when the results of the first straight-pipe experiments are used. Fig. 4, especially, is of interest in comparison with Fig. 3. Both are based on the equal-lengths comparison. In Fig. 3 the excess loss tends to approach zero for the long-radius curves, but, in Fig. 4, the average excess appears to be nearly constant for the longer radii. If this latter constant

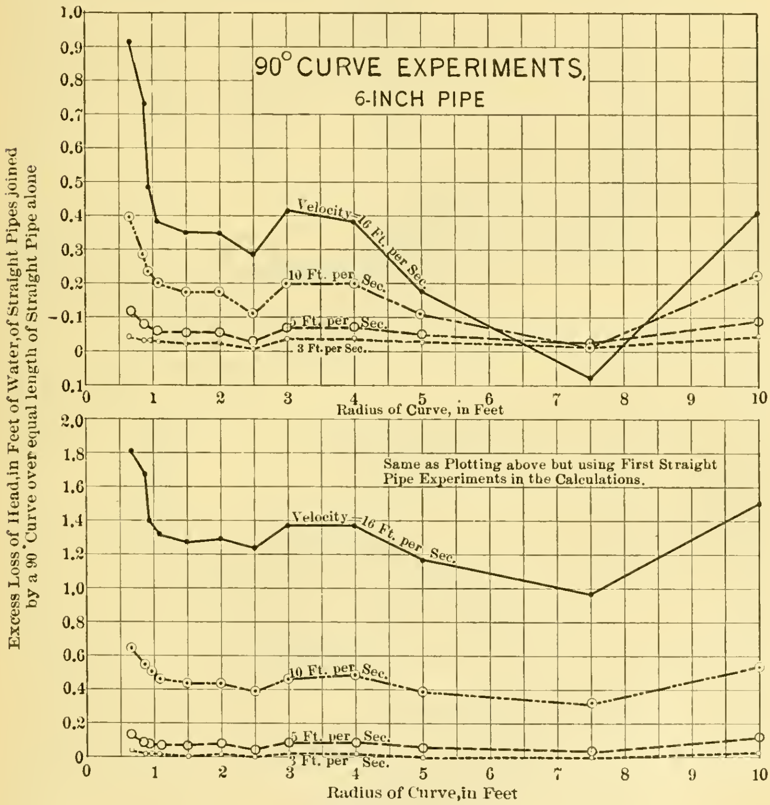
TABLE 9.—LENGTHS OF STRAIGHT PIPE, IN FEET, TO GIVE LOSS OF HEAD EQUAL TO THE EXCESS LOSS DUE TO THE CURVES. ON THE BASIS OF SECOND STRAIGHT-PIPE EXPERIMENTS.

For Equal Lengths on Center Lines:					To Connect Two Fixed Points:				
No. of curve.	Velocity, in feet per second.				No. of curve.	Velocity, in feet per second.			
	3	5	10	16		3	5	10	16
1	8.4	6.7	4.4	3.2	1	4.0	2.4	0.2	−1.1
2	3.2	1.6	0.2	−0.6	2	0.0	−1.6	−2.7	−3.8
3	5.0	3.5	2.1	1.4	3	3.0	1.3	0.2	−0.8
4	6.8	5.2	3.9	3.0	4	5.0	3.4	2.1	1.2
5	6.8	5.1	3.9	3.2	5	5.6	3.8	2.7	1.9
6	3.0	2.5	2.1	2.2	6	1.8	1.3	1.2	1.2
7	5.6	4.3	3.5	2.7	7	4.8	3.4	2.7	1.9
8	4.8	4.1	3.5	2.7	8	4.2	3.4	2.9	2.0
9	5.2	4.4	3.9	3.0	9	4.8	3.9	3.5	2.6
10	6.0	5.1	4.6	3.8	10	5.6	4.7	4.4	3.4
11	5.8	5.8	5.6	5.7	11	5.6	5.5	5.4	5.3
12	9.8	8.6	7.7	7.0	12	9.6	8.3	7.5	6.7
					Screw elbow.	14.3	13.4	13.5	13.6

TABLE 10.—EXCESS LOSSES OF HEAD DUE TO CURVES, EXPRESSED IN TERMS OF VELOCITY HEADS. ON BASIS OF SECOND STRAIGHT-PIPE EXPERIMENTS.

For Equal Lengths on Center Lines:					To Connect Two Fixed Points:				
No. of curve.	Velocity, in feet per second.				No. of curve.	Velocity, in feet per second.			
	3	5	10	16		3	5	10	16
	$\frac{V^2}{2g} = 0.1297$	$\frac{V^2}{2g} = 0.3882$	$\frac{V^2}{2g} = 1.553$	$\frac{V^2}{2g} = 3.975$					
1	0.30	0.23	0.15	0.10	1	0.14	0.08	0.01	−0.04
2	0.11	0.05	0.01	−0.02	2	0.00	−0.05	−0.09	−0.12
3	0.18	0.12	0.07	0.05	3	0.11	0.05	0.01	−0.03
4	0.24	0.18	0.13	0.10	4	0.18	0.12	0.07	0.04
5	0.24	0.18	0.13	0.10	5	0.20	0.13	0.09	0.06
6	0.11	0.08	0.07	0.07	6	0.06	0.05	0.04	0.04
7	0.20	0.15	0.12	0.09	7	0.17	0.12	0.09	0.06
8	0.17	0.14	0.12	0.09	8	0.15	0.12	0.10	0.07
9	0.19	0.15	0.13	0.10	9	0.17	0.14	0.12	0.08
10	0.21	0.18	0.15	0.12	10	0.20	0.16	0.15	0.11
11	0.21	0.20	0.19	0.19	11	0.20	0.19	0.18	0.17
12	0.35	0.30	0.26	0.23	12	0.34	0.29	0.35	0.22
					Screw elbow.	0.52	0.47	0.45	0.44

tendency represents the truth, then a remarkable dilemma is presented. As the curvature of a pipe becomes less and less the external conditions approach nearer to those of straight pipe. The natural inference is that the loss of head also approaches straight-pipe values, unless, indeed, it be argued that the slightest deflection from straight pipe immediately causes a considerable excess loss of head.* Fortunately, there exist experimental data which will assist in the consideration of these points. These will be given presently.



FIGS. 3 AND 4.

Fig. 6 shows the same tendency as Fig. 5, although the decrease in loss of head with increasing radius of curvature is less decided on the basis of the first straight-pipe experiments.

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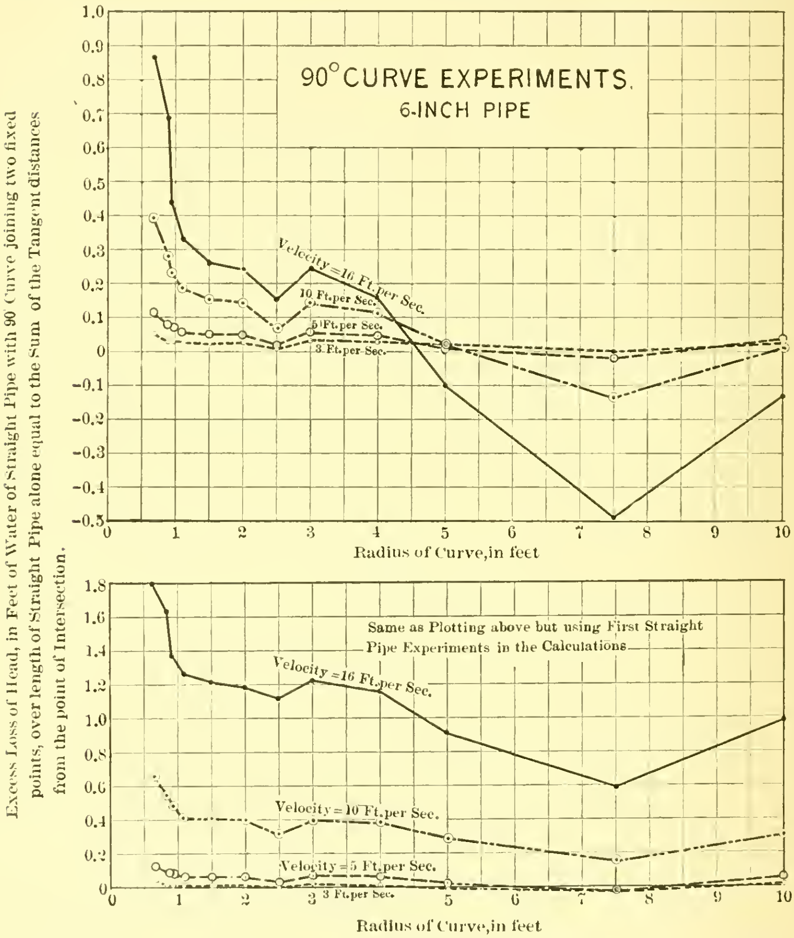
* Detroit Curve Experiments, *Transactions*, Am. Soc. C. E., Vol. XLVII, 1902, Conclusion *J*, page 191, and pages 186–187.

As to the difference found between the results of the two straight-pipe series, the following may be stated. Conditions of temperature were favorable for some rusting during September, after the first straight-pipe series and before the first curve experiments were made.



FIGS. 5 AND 6.

During the curve experiments, the data show no indication of increasing roughness, or of any effect of changes in temperature of the water.* All the straight pipes had been used in a steam-heating main

*This is remarkable. It has been observed, for smooth brass pipes of all sizes between $\frac{1}{5}$ in. and 5 in. in diameter, that the loss of head is increased about 4% for a decrease of temperature of the water of 10° fahr. Rougher pipes, such as galvanized iron and wrought iron, show no effect due to temperature changes.

for some years, and all seemed to have a uniform internal appearance. The first series had no velocities greater than 6 ft. per sec., while the second series had velocities as high as in the curve experiments. A

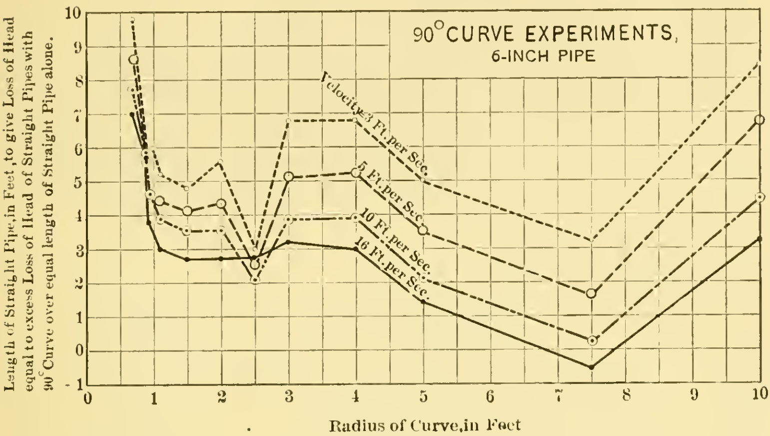


FIG. 7.

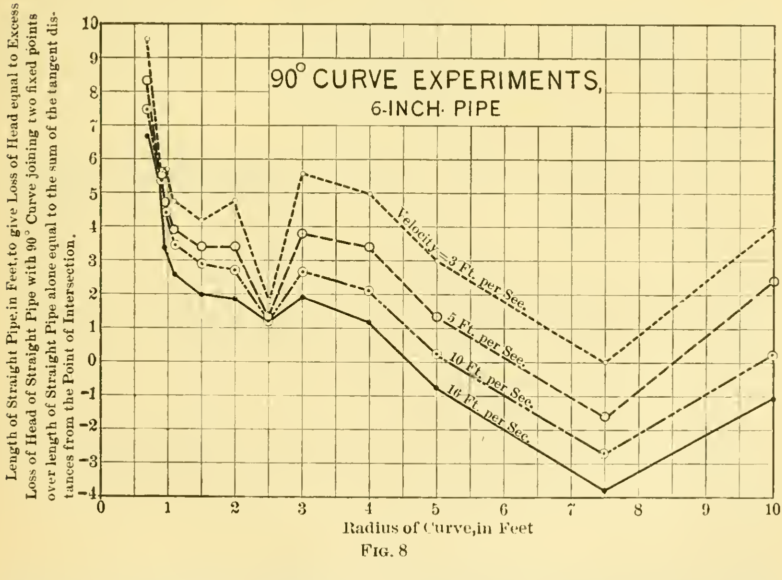


FIG. 8

separate measurement of the loss of head in Pipes Nos. 2 and 3 when uninfluenced by curvature was not made, and it is impossible to decide as to their hydraulic properties as compared with Pipes Nos. 4 and 5.

It is thus clear that, without additional evidence, it is not possible to reject one of the straight-pipe series and accept the other, or to feel safe in using average values.

In relation to this matter, the writer desires to present the data from measurements on an 8-in. cast-iron water main. This main supplies raw water to the Cornell University Filtration Plant.* It had been laid and in use for three years before the experiments in the fall of 1906. Before laying, the inside diameter of each length had been calipered. At the time of laying, each pipe length was set accurately to line and grade with a transit. After laying, and before covering

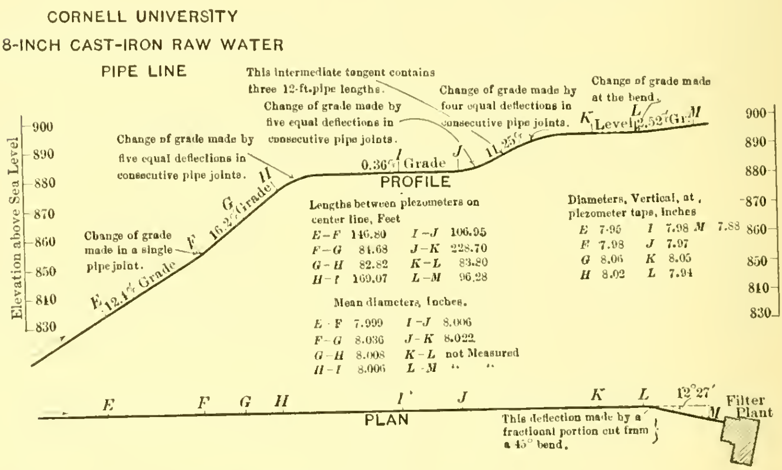


FIG. 9.

the pipe, the piezometer holes were drilled and tapped, and the diameters measured at these points; the ½-in. brass piezometer cocks were inserted so as not to project inside, and the lengths between the piezometers were measured and checked in the ditch.

The plan and profile of the pipe line are shown by Fig. 9. There are eight experimental sections, four of which contain deflections, and four of which are straight, preceded by considerable lengths of straight pipe.

The piezometer taps were placed on top of the pipe and 1 ft. up stream from the joints, except that those before deflections were placed 2 ft. up stream from the joint where the first deflection occurred.

The flow in the pipe line was measured at the filter plant by a

*Designed by G. S. Williams, M. Am. Soc. C. E. See *The Engineering Record*, April 9th, 1904.

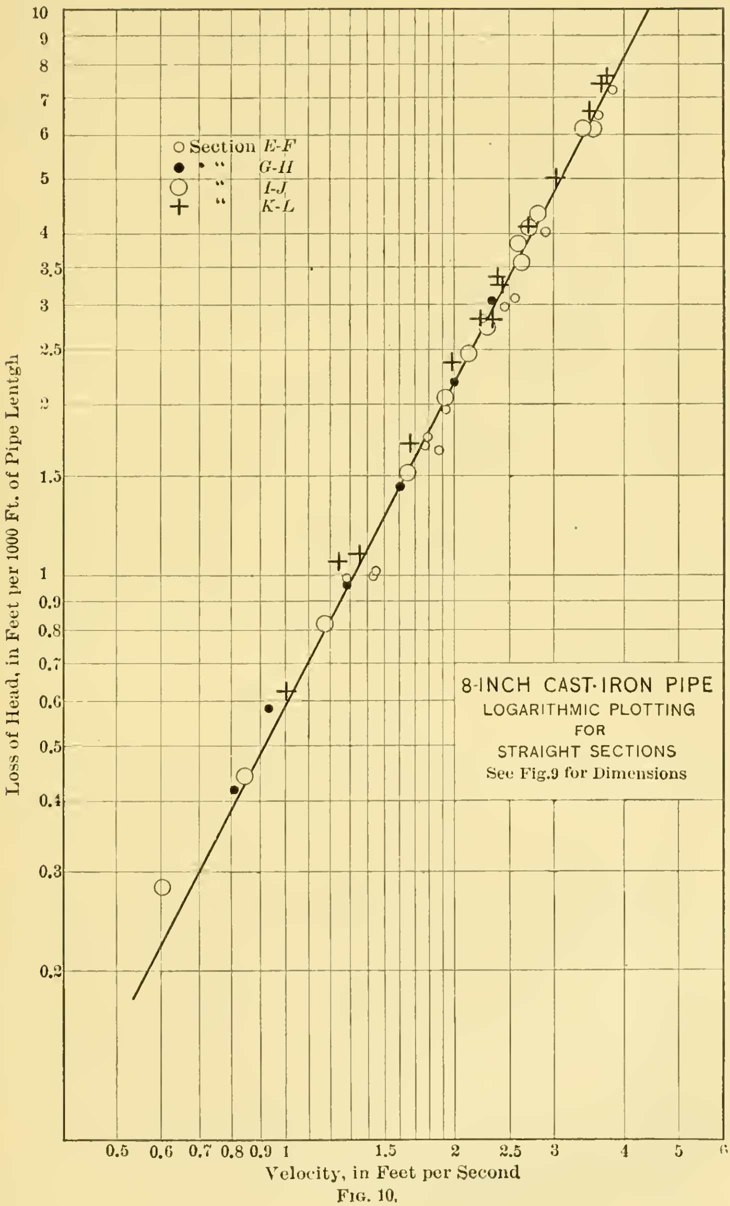

10
9
8
7
6
5
4
3.5
3
2.5
2
1.5
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
Loss of Head, in Feet per 1000 Ft. of Pipe Lentgh
Section E-F
" G-H
" I-J
" K-L
8-INCH CAST-IRON PIPE
LOGARITHMIC PLOTTING
FOR
STRAIGHT SECTIONS
See Fig.9 for Dimensions
0.5
0.6
0.7
0.8
0.9
1
1.5
2
2.5
3
4
5
6
Velocity, in Feet per Second

FIG. 10.

Venturi meter which had been accurately calibrated in place by volumetric measurements. A differential water gauge was used with the meter. The losses of head were measured with a portable differential water gauge mounted on a tripod. The gauge was set up on the ground midway between two piezometer wells, and pressure connections were made with small three-ply rubber hose after thoroughly blowing off to remove all air.

The flow was controlled by a valve at the filter plant. It was possible to shut this valve down entirely, and thus get no-flow conditions and a check on the gauge readings. The electric motor-driven two-stage centrifugal pumps at the lower end of the pipe line allowed this procedure without any trouble.

After changing the valve setting and allowing the flow to become steady, simultaneous readings were taken on the meter and the loss-of-head gauges. The results are shown graphically in Figs. 10 and 11. For the straight sections the observed losses of head have been reduced uniformly to loss per 1 000 ft. It will be seen on Fig. 10 that the straight sections differ among themselves. Thus Section *E-F* has less and Section *K-L* has greater loss of head than the average of the four sections; in fact, Section *K-L* has about 15% greater loss of head than Section *E-F*. The equation of the mean line for the four sections of straight pipe is

$$H = 0.586\ V^{1.91}$$

where H is the loss of head, in feet per 1 000 ft., and V is the velocity, in feet per second. (The corresponding values of C, in $V = C\ \sqrt{R\ S}$, are: at 1 ft. per sec., 101; at 4 ft. per sec., 107.)

Fig. 11 shows the difference between this average law of flow for the straight portions and the hydraulics of the sections containing deflections. It is remarkable that Section *F-G* with a single 3.8° deflection, Section *H-I* with a curve composed of five 3.17° deflections, and Section *J-K* with a reverse curve composed of one curve with five 2.18° deflections and another with four 2.81° deflections, all show, on the whole, less loss of head than the average of equal lengths of straight pipe. Section *L-M*, with a short-radius bend giving a deflection of 12° 56′, is the only one showing a greater loss of head.

The writer does not argue from this that such easy curves or deflections are more favorable for the flow of water than straight pipe.

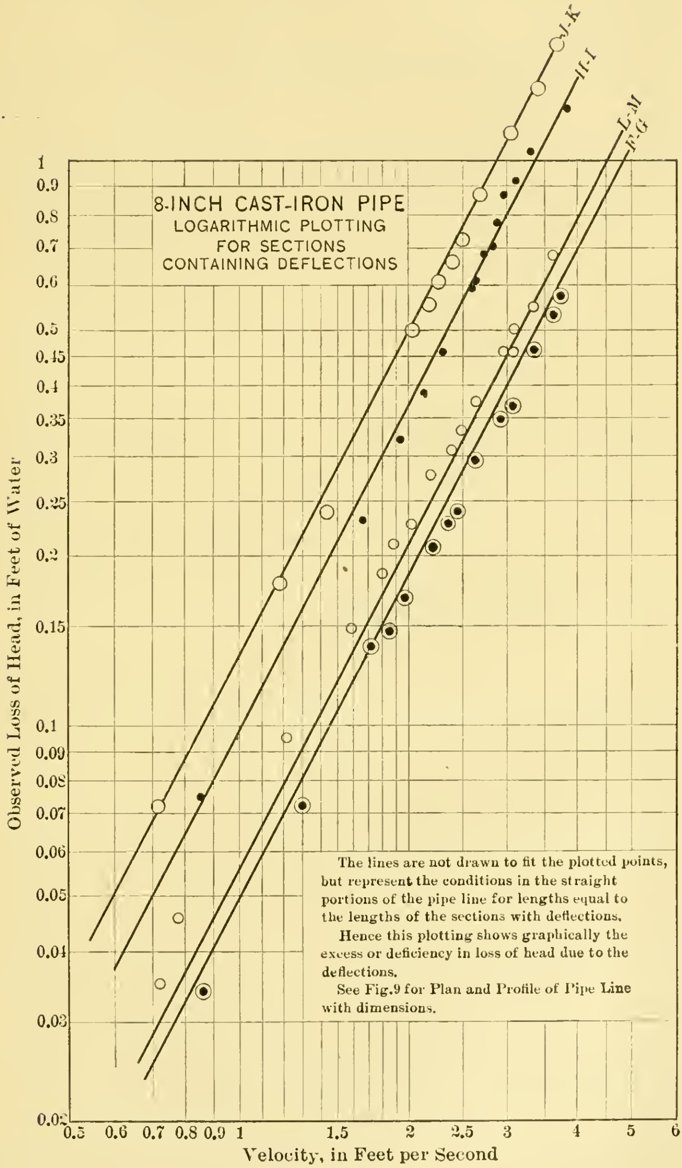

8-INCH CAST-IRON PIPE
LOGARITHMIC PLOTTING
FOR SECTIONS
CONTAINING DEFLECTIONS
J-K
H-I
L-M
F-G
1
0.9
0.8
0.7
0.6
0.5
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
Observed Loss of Head, in Feet of Water
0.5 0.6 0.7 0.8 0.9 1 1.5 2 2.5 3 4 5 6
Velocity, in Feet per Second
The lines are not drawn to fit the plotted points, but represent the conditions in the straight portions of the pipe line for lengths equal to the lengths of the sections with deflections.
Hence this plotting shows graphically the excess or deficiency in loss of head due to the deflections.
See Fig. 9 for Plan and Profile of Pipe Line with dimensions.

FIG. 11.

but he does see the indication that any difference is very small and may be less than the difference between two straight sections in the same pipe line, as occurs in the case above recorded.

The 6-in. wrought-iron pipe experiments also give some information on the question of the effect of slight deflections in otherwise straight pipe. The first series, of October 10th, 1907, was made with a decidedly zigzag appearance of the down-stream tangent, that is, the joints were not in a straight line, although the individual pipe lengths themselves were straight. On October 12th the series was repeated, but with the down-stream tangent carefully aligned. No difference in results is noticeable.

Viewed in the light of the foregoing, it is easy to decide that the first 6-in. straight-pipe experiments do not apply to the later curve experiments because the deduced excess loss of head does not continue to approach zero for the long easy curves. In this respect, the difference between Figs. 3 and 4 is noteworthy.

Now, all of this contradicts the findings of Messrs. Williams, Hubbell, and Fenkell in the Detroit Experiments. The writer cannot imagine that radically different laws apply to the cases investigated by him and by these experimenters.

One difference in conditions is to be noted, however. The long-radius curves in the 30-in. Detroit main were made up of several pieces, while the writer's 6-in. curves were all one-piece bends. As to the probable small effect of the joints, the writer's 8-in. pipe experiments, with small deflections, give some idea; but there are other possible causes for the divergence of the findings. The smallness of the measured losses, with the comparatively low velocities available in the Detroit 30-in. main, would tend to magnify excess losses due to other effects than curvature. Thus, in Figs. 3 and 5 it will be seen that a very different appeal to the eye is given by the line for a velocity of 10 ft. per sec. than by the line for a velocity of 3 or even 5 ft. per sec.

There remains, also, for the Detroit Experiments, the possibility of relatively large errors due to several causes. These errors were considered by Messrs. Williams, Hubbell, and Fenkell in their closing discussion, and a table was presented* in which corrections—as large as 50% in one case—were made to the results given in the main part of the paper. These corrections materially alter the appearance of the remarkable Fig. 90 of the paper.

*Table No. 89, page 360, *Transactions*, Am. Soc. C. E., Vol. XLVII, 1902.

If, now, in the Detroit Experiments, to the causes for incorrect deductions above mentioned there be added the effect of using for comparison the results of experiments on short sections of straight pipe that might have had quite different hydraulic properties from the straight pipe in the curve section, it is easy to see that the combination of circumstances may have led to conclusions not at all general in their applicability, and perhaps even wrong for the case in hand; but, as to this matter, the writer is quite content with suggesting the salient arguments.

After all, however, the engineer will be particularly interested in the magnitude of the excess losses of head due to curves. Are they seriously large in an extreme case?

Figs. 7 and 8 show that the excess loss of head for the shortest 90° curve is equal to the loss in 7 to 10 ft. (or 14 to 20 diameters) length of straight pipe. When the radius is 2½ diameters the excess loss of head is equal to the loss in 5 to 10 diameters length of straight pipe. In this relation Fig. 12 is interesting. It is seen that, for the smooth brass 180° curves, the excess loss of head is rather less than the loss of head in 7 diameters length of straight pipe. The expression, $0.15 = \frac{V^2}{2g}$, seems to give a fair average value for the range of these brass-curve experiments.

Table 9 shows that the 6-in. screw elbow gives an excess loss of head equal to the loss in about 27 diameters length of straight wrought-iron pipe. The writer has also the record of some accurate measurements on the loss due to 3-in. and 4-in. screw elbows, from which it appears that the losses are equal, respectively, to the losses in 25 and 27 diameters length of straight wrought-iron pipe.

When, therefore, the Detroit Experiments, after thorough revision, indicate for a long easy curve in 30-in. pipe an excess loss of head equal to the loss in 50 diameters length of straight pipe, not only does the loss seem to be too large when compared with the loss in a screw elbow where sudden enlargement and contraction are present in addition to extremely short-turn curvature effects, but the whole trend of the results is directly the opposite of what is shown by the writer's experiments on 6 and 8-in. pipes.

Now, it may be that Nature changes her methods somewhere between pipes of 8 and 30 in. in diameter, as regards the effects of curvature. It must be so, if both the Detroit Experiments and those of the writer have been interpreted correctly. The evidence seems to stand as follows: In Detroit, with a small range of low velocities (the greatest about 3 ft. per sec.) the 30-in. pipe line shows an increas-

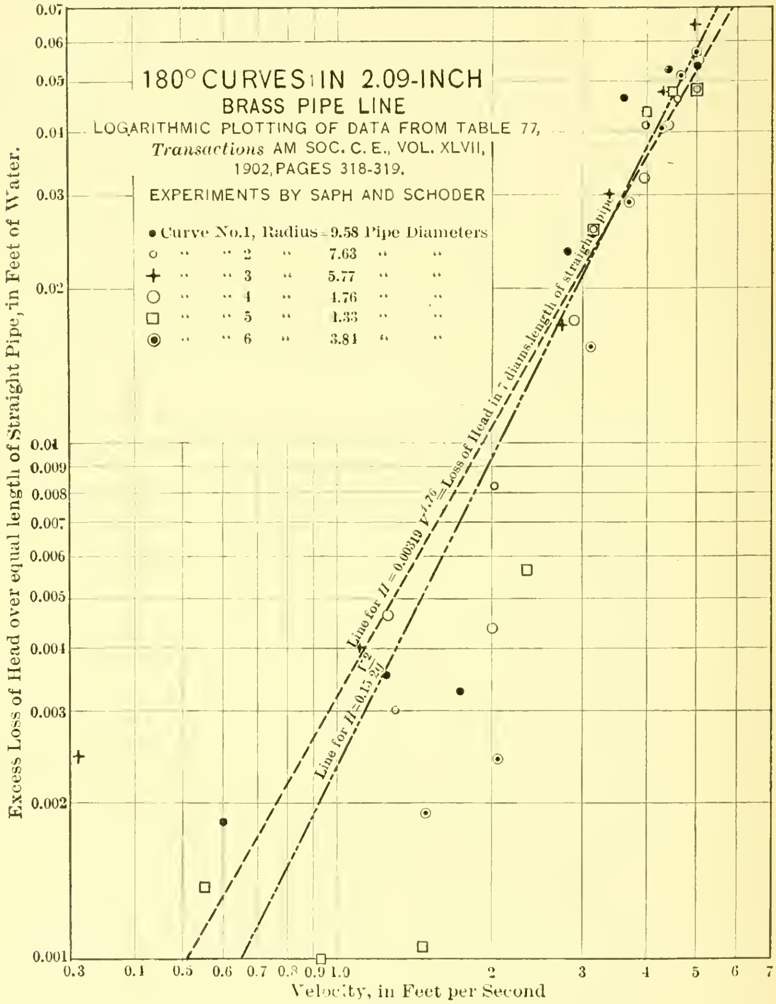


FIG. 12.

ing loss of head with increasing radius of curvature for curves with radii between 5 and 25 diameters. (The other pipe lines, 12 and 16-in., have only short-radius curves, and the results are not consistent.) The writer's experiments on 6-in. pipe, with velocities up to 16 ft. per sec., show the opposite. His 8-in. pipe experiments indicate no measurable excess loss of head for bends composed of a series of small deflections at consecutive joints in ordinary cast-iron pipe, and his 6-in. pipe also shows no appreciable excess loss for several small deflections. The quantitative results for the long Detroit 30-in. 90° curves are far in excess of the 180° 2-in. brass curves, the writer's 6-in. 90° curves, the 3, 4 and 6-in. screw elbows, and, as above stated, are contradicted by the no-excess results of the 8-in. long, easy curves.

If, then, for any reason, an engineer wishes to use a long, easy curve, or a series of small deflections, in the joints between straight pipes, the writer's experiments indicate just what most hydraulic engineers have assumed, namely, that there is practically no difference between the loss of head due to a long, easy curve and that due to an equal length of straight pipe.

No calculations on the basis of the loss of head per foot length of curved portion have been made, because all the excess loss of head probably does not occur in the curve, an unknown part of the loss taking place in the down-stream tangent in the region where the abnormal flow returns to normal. Partly for this reason, also, it was not deemed wise to attempt any correction on account of the smaller diameter of the 6-in. cast-iron curves, or, indeed, on account of the variation in the diameters of any of the curves from the mean diameter of the straight pipe. Besides, we have no precise knowledge concerning the effects of slight sudden enlargements or contractions, such as are involved in these experiments.

The writer has found this an interesting study, and would gladly have extended the experiments to other sizes of pipes, but the cost, in time and money, is rather large. He would suggest the desirability of similar studies on other small sizes, and of many further experiments on curves in existing large pipe lines.

The writer desires to acknowledge his indebtedness to Professor W. B. Gregory, who worked with him throughout the 6-in. pipe experiments, and in subsequent Pitot tube investigations in the course of which it was shown that normal flow prevailed at all the piezometers in the 6-in. pipe.

After preparing this paper, the writer's attention was called to a record of experiments on 90° bends in 3 and 4-in. pipes.* These ex-

* Paper No. 3679, "Loss of Pressure in Water Flowing through Straight and Curved Pipes," by Arthur William Brightmore, M. Inst. C. E., *Minutes of Proceedings*, Inst. C. E., Vol. CLXIX, 1906-1907, p. 323.

periments covered a right-angled elbow and right-angled bends having radii equal to 2, 4, 6, 8, 10, 12, and 14 diameters in 3-in. pipe, and the same, excepting the last two, in 4-in. pipe.

The arrangement resembled that by the writer except that the down-stream piezometer was located rather close to the curves, being 6 ft. 8 in., or 27 diameters, distant for the 3-in. pipe, and from 5 ft. to 6 ft. 7¼ in., or from 15 to 19 diameters, distant for the 4-in. pipe. The length of straight pipe up stream from the up-stream piezometer was 7 ft. for both the 3 and 4-in. pipes.

Fig. 13 is a reproduction of Mr. Brightmore's plotting of the results of his experiments. There is a striking similarity between the

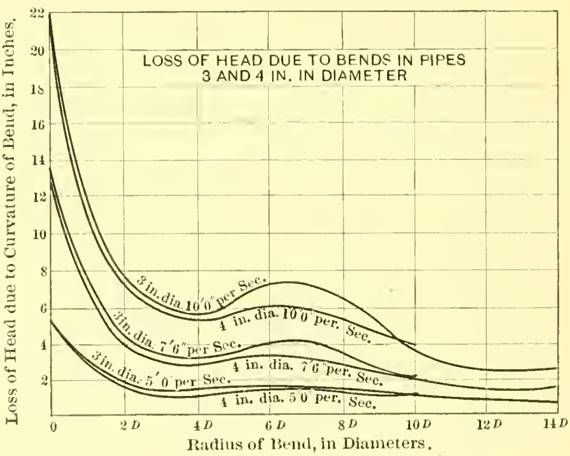


FIG. 13.

shape of his curves and those by the writer in Fig. 3 (which corresponds to Fig. 13). The hump in the curves between 6 and 8 diameters appears in both Figs. 3 and 13. The quantitative results are not readily compared. For Mr. Brightmore's 4-in. pipe the straight pipe (rusted cast iron) had a coefficient of 47.5 in the formula, $V = C\sqrt{R\,S}$. For the 3-in. pipe (galvanized) the coefficient was 65 to 70 for V, ranging from 3 to 11 ft. per sec. In the writer's experiments, the coefficient for the 6-in. wrought-iron pipe was 119 to 125 for V ranging from 3 to 16 ft. per sec. Mr. Brightmore purposely allowed the pipes and curves to become rusted, but, he states, not tuberculated. The foregoing figures indicate that he was working with much rougher pipes than the writer used, and the quantitative values shown on Fig. 13 indicate the same in comparison with Fig. 3.

It is evident that further experiments are desirable before precise laws can be stated, although the qualitative results by Mr. Brightmore and the writer agree in indicating a decreasing loss of head for an increasing radius of curvature.

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PAPERS AND DISCUSSIONS

NOTES UPON DOCKS AND HARBORS.

By Luther Wagoner, M. Am. Soc. C. E.

To be Presented September 2d, 1908.

During the year 1907 the writer visited the principal ports of Europe and the United States for the purpose of procuring data for the preparation of a report upon the future needs of San Francisco in the matter of port improvements.

For the foreign work he had exceptional facilities for observing completed works and receiving information concerning new or projected work. As a result, he obtained a large quantity of technical literature, maps, plans, and photographs, as well as notes, and, believing that some of the data obtained may be of general interest to the profession, he has prepared the following paper.

Comparison of European with American Harbors.

A striking difference in the ground plans of port works is at once apparent to the visiting engineer. The development by piers or jetties, like that of San Francisco or New York, has no parallel. Generally speaking, the European idea is one of enclosed basins, with or without locks, as tidal conditions may require. In the Mediterranean, where the tide ranges from 10 to 20 in., there is usually no protection

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

from the sea, so a mole or breakwater is built and behind it a safe harbor is created by dividing up the protected area into basins separated by solid filled piers, usually from 300 to 400 ft. wide. Upon these piers there is first a space of from 20 to 25 ft. for cranes and one railway track. Next are the sheds, from 75 to 130 ft. wide; then there is an open space between the sheds for two railway tracks, and a wagon road between them. At Naples it was necessary to build such a wall in 110 ft. of water; at Genoa, in depths of from 50 to 70 ft., and at Marseilles from 60 to 75 ft.

At cities like Antwerp, Rotterdam, Bremen, and Hamburg, on tidal rivers, the problem has been one of obtaining the desired area by dredging out basins and enclosing them with quay walls, leaving sufficient space for railway connections, sheds, warehouses, and roads, the whole being arranged so as to facilitate business. In this matter it is specially noteworthy that railway connections have been provided at all points, and, generally speaking, freight can be transferred directly from the ship to the car or *vice versa,* thus avoiding delay and extra handling.

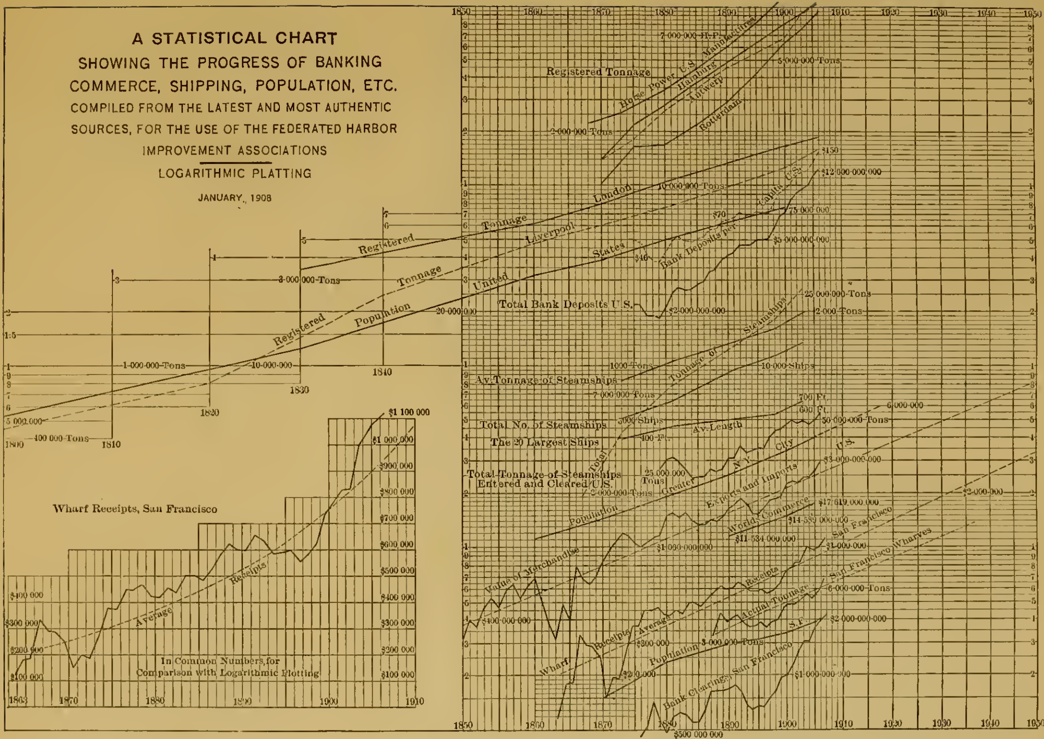
Liverpool, which is essentially a receiving and forwarding port, has perhaps the best arrangement of railways and stations. In a length of 6 miles of water front there are ten or more great railway freight stations, all being just at the rear of the docks. In point of efficiency of belt-railway service, there is nothing at present in the United States that is comparable with that of any first-class European port. Within the City of Philadelphia there are more railway lines connecting manufacturing establishments with the various roads than in any other city of the same area, but, on the other hand, the railways own and control about 80% of the available water front. New York has no belt-railway, and it is doubtful if it will ever have one, as the price of land required for it is prohibitory. San Francisco is fortunate, in this respect, as the State owns the water front, along which there is a street 200 ft. wide, and upon which there is a State belt-railway. If the recent plans for the port improvements are carried out, the new street along the water front will have a width of 350 ft., upon which there will be ample room for belt-roads, warehouses, and railway freight stations, as well as street cars and other vehicular traffic.

In Europe the systematic planning of new work is especially noteworthy. For example, in Antwerp, a broad, comprehensive scheme of port enlargement has been carefully prepared, and is of such magnitude that it will require many years for its execution. The Board of Control has acquired the lands, and planned the roads and rail connections long in advance of actual needs, thus permitting its orderly execution without regard to any vested interests. As the city grows up around the new port, it will not have to make expensive changes. Similarly, Rotterdam has planned a gigantic basin where it is intended to dredge 650 acres to a depth of 40 ft. and deposit the soil on the low lands below. It is estimated that the completion of this work may require from twenty to thirty years. All the larger ports in Europe are planning and executing systematic extension of their facilities to hold their present and secure a share of expected increased trade. Coincident with such work, much attention is given to making the city attractive, a place where one would like to live, and where a visitor would like to go again. They have, in general, a good administration, and are able to select and keep employed men of ability to administer the public utilities. As a rule, they look further ahead than Americans; in other words, they think more before taking action.

Statistical Chart.

Among the duties imposed upon the writer was the request to try and forecast the amount of the future commerce of the port of San Francisco, and plan improvements ample for such purpose, say fifty years hence. The method of investigation and the results are shown upon the diagram, Plate XLVIII. After a preliminary study of the data, it was found that, owing to the rapid increase in the quantities platted, and the natural irregularities of the subject, the ordinary method of showing the time relation was not suitable for prediction purposes, and the logarithmic method was used. In the diagram, Plate XLVIII, the year 1700 is zero, 1800 is 100, and 1900 is 200, and the logarithms of these numbers were used for the time scale. Four vertical scales of 1 to 10 were drawn, so as to cover all the data used without confusion of lines. In such a diagram an inclined line denotes an exponent; for example, the average line drawn through "Value of Merchandise Exports and Imports, United States," is the graphical representation of

$$\text{Average value} = 0.0000244\ (\text{Year}, 1700)^{6.06}$$


A STATISTICAL CHART
SHOWING THE PROGRESS OF BANKING
COMMERCE, SHIPPING, POPULATION, ETC.
COMPILED FROM THE LATEST AND MOST AUTHENTIC
SOURCES, FOR THE USE OF THE FEDERATED HARBOR
IMPROVEMENT ASSOCIATIONS
LOGARITHMIC PLATTING
JANUARY, 1908
Registered Tonnage
Horse Power U.S. Manufactures
Hamburg
Antwerp
Rotterdam
London
Liverpool
United States Population
Registered Tonnage
Total Bank Deposits U.S.
Bank Deposits per Capita U.S.
Tonnage of Steamships
Av. Tonnage of Steamships
Total No. of Steamships
The 20 Largest Ships
Av. Length
Total Tonnage of Steamships Entered and Cleared U.S.
Total Greater N.Y. City Population
Exports and Imports U.S.
World's Commerce
Value of Merchandise
San Francisco Receipts
San Francisco Wharves
Actual Tonnage
Wharf Receipts Average
S.F. Population
Bank Clearings San Francisco
Wharf Receipts, San Francisco
Average Receipts
In Common Numbers for Comparison with Logarithmic Plotting

For Hamburg's tonnage, the exponent is about 10, and for the population of the United States the exponent is about 4. The degree of accuracy of this method of forecasting can be readily seen by going back, say, to 1870 or 1880, and projecting ahead to the present time. The application of logarithmic platting to such purposes is believed by the writer to be new.

It is commonly held by experts that the production of pig iron and steel is a good financial barometer. The production of iron and steel (not shown on Plate XLVIII), the horse-power used in the United States manufactures, and the total bank deposits in the United States are three curves which can be almost exactly superimposed by moving them vertically into position; in other words, they have a common exponent or law of increase.

Some very interesting conclusions may be drawn from this diagram. For example, up to the year 1900, London and Liverpool tonnage were moving at a common rate of growth. London then needed port improvements, but expended its energy in discussion (and a very thorough one it was), while Liverpool deepened and extended its docks; the result can be plainly seen upon the diagram after 1900. The diagram also shows that the exponent of increase of population in the United States is about 4; exports and imports of the United States, about 6; world's commerce, about 6; United States bank deposits, about 10; the horse-power, and manufacture of pig iron, steel, etc., about 11 to 13; consequently, per capita, this means a rapid increase of business for the engineer, because the increase in such activities, referred to a time relation, is measured, per capita, by the difference in exponents.

The data relating to steamships are quite interesting. They were compiled from a valuable report,* by Elmer L. Corthell, M. Am. Soc. C. E. Attention is called to the very rapid and uniform increase in the average tonnage of vessels, about 841 tons in 1873, 1 955 tons in 1903, and now about 2 300 tons.

The draft (loaded) of a vessel can be expressed by an equation in this form:

$$\text{Draft} = K\ (\text{length} \times \text{breadth})^{\frac{1}{3}},$$

in which K ranges from 1 for small vessels to 0.88 for those of the largest type. There is no doubt that K would be uniformly taken

* To the Tenth Congress, Milan, 1905, Permanent International Association of Navigation Congresses.

about 1, if the depth of water in ports permitted it. It is quite probable that vessels will go on increasing in general dimensions, because it is more economical to do transoceanic business in large vessels of great depth than in small ones. Naturally, there must be an almost general work of deepening ports and docks before there can be any great advance in draft. The average ship is a tramp, and does business at any place, and, for the present, not many ships of great draft can be operated, and those only between ports in which there is the requisite depth of water and dockage; but it may confidently be predicted that the natural economic law of draft will involve the universal deepening of at least all the world's greater ports.

Very notable is the relation between the exponent of growth of population of the United States—about 4—and that of the trade of the United States and of the world's commerce—about 6. This means per capita: An increase as the square of the time since the year 1700, or, for every dollar's worth of trade per capita now, about $1.50 of trade 45 years hence.

Still more marked is the relation, if population be compared with such activities as the production of steel and pig iron, horse-power used in manufactures, and bank deposits. All these activities have increasing exponents, the difference being from 6 to 9, which means that, per capita, in the United States these activities are increasing as the 6th to the 9th power of the time, measured from the year 1700, all of which points to an ever-increasing demand for competent technically trained men to direct such service.

Concrete Piles.

Concrete piling has been used in Ravenna, Venice, Boulogne, Rotterdam, Hamburg, Southampton, and a number of places. Speaking generally, the experience has been satisfactory, although in some cases serious difficulty has been encountered in driving such piling. Some trouble was observed in Hamburg, where concrete piles, from 30 to 36 ft. long, were being driven for the launching ways of a shipyard, many of the piles being badly broomed and broken. In this case, they were driven in a sand fill, without the use of a water-jet.

Southampton is the best place at which to observe such work, as experience there covers a longer time, and the piles are of greater dimensions than elsewhere. At the time of the writer's visit, 15 by 18-in.

piles, 60 ft. long, were being driven as sheet-piling at the front of a quay wall which had failed by sliding outward. These piles were driven about 30 in. in front of the wall by using a jet and a steel bonnet lined with a cushion of sawdust. The penetration was 28 ft., a heavy steam hammer being used for driving. Embedded in each pile near its center there was a $\frac{3}{4}$-in. iron jet pipe with an elbow about 2 ft. from the top. At the bottom this pipe was screwed into a cast-iron point; the nozzle opening was $\frac{3}{8}$ in. The pressure was 200 lb. and the quantity of water used was from 5.5 to 6 cu. ft. per min. At least 60 days was allowed for seasoning. No trouble was experienced in driving, and no brooming or breakage was noticed.

At this place there is a coal wharf on reinforced concrete piles which also has diagonal braces of concrete and a concrete deck. This wharf has been subjected to an unusual amount of buffeting and hard service, and is quite elastic. Two piles which were broken off under water by a collison were repaired by divers. The broken parts were removed, then an iron tube was inserted and caulked in place, after which the tube was pumped out and filled with concrete.

Extensive improvements being under way for the London & South Western Railway (the owner of the docks), their engineer, Mr. Shields, had just completed a careful inspection of all concrete work. He reported that all work below water was in sound condition, but that a few remarkable cases of rusting and exfoliation had occurred above the water, for which no satisfactory explanation could be found. In this the writer saw a very strong resemblance to the failure of expanded metal in concrete floors, reported to the Structural Association of San Francisco in 1906. As a final result, it was decided that for any new work at Southampton, there should never be less than 2 in. of concrete covering the reinforcement, both above and below water.

At Paris the writer was informed by E. T. Quinnette de Rochemont, M. Am. Soc. C. E.,* that although the length of experience with reinforced concrete piles was not greater than from 5 to 7 years, he was favorably impressed, and that the Corps of the *Ponts et Chaussées* would use them freely if the conditions required it.

Young's million-dollar pier at Atlantic City is an example, on a large scale, of the use of concrete piles. These piles have a riveted case of $\frac{3}{16}$-in. steel to which is secured a concrete point. They were hoisted into position and filled with concrete to a depth about equal to the

* *Inspecteur Général des Ponts et Chaussées.*

water; they were then lowered and put down by jetting about 15 ft. into the sand, care being taken to have the concrete filling at all times above the surface of the water. There are no reinforcing rods, and, when the outer galvanized cylinder fails, the structure must depend upon the tensile strength of the concrete to resist the lifting action of the waves, which, owing to the exposed position of the pier, may be quite severe during a storm.

The Wear of Concrete.

Most of the quay walls observed in Europe are faced with rubble or ashlar. In Belgium, Holland, and Germany they have a rubble facing of hexagonal basalt blocks about 2 ft. deep, to prevent wear. In the Albert Dock, London, a concrete non-faced quay wall has been in use about 30 years, and, having been subjected to much buffeting from lighters as well as ships, it has worn away about 2 in.

Careful inspection was made as to a possible action at or near the water line due to freezing, or wave action, or both, but nothing noteworthy was seen. However, at Baltimore there is a noticeable exception, for, on certain bridge piers, and for a vertical range of 18 in., the concrete has disintegrated to a depth of several inches about at the ordinary water line. Aside from the affected part of the concrete, which was covered with a vegetable growth more dense than in the lower unaltered part, nothing unusual was observed. The concrete above and below the affected zone is good.

Various theories have been advanced to account for the decomposition. The most plausible one is that with a small tidal range there is a destructive action by the waves lapping the affected zone, and this, perhaps, is assisted by ice action. None of the theories suggested, when weighed and considered in reference to similar structures elsewhere, appears to the writer as tenable. Believing it worthy of investigation, the matter has been reported to the United States Geological Survey, with a request for an investigation and report.

Concrete Caissons.

In Europe extensive use is being made of hollow concrete caissons, both plain and reinforced, for breakwaters and quay walls. The structure is towed into position and sunk, after which the hollow cells are filled. Some of those used for breakwaters weigh more than 5 000 metric tons.

At Rotterdam, caissons 131.2 ft. long, and having a width of 32 ft. at the base and 16 ft. at the top, and 43 ft. high, were being used. A middle division wall through the length, and nine cross-walls, divide the caisson into twenty cells. Four such caissons were built at the same time in an improvised dry dock. The first step was the preparation of a base, about 2 ft. thick and 32 by 131.2 ft., well reinforced, and in this were embedded the vertical rods for the walls. The external side walls, about 14 in. thick over the base, were carried up with a batter. When the caisson walls were up to about five-eighths of their final height, the gates were opened and the caissons were floated out to a place in the harbor where they were secured to mooring piles. There they were completed, meanwhile being afloat for one or two months.

As there are streaks of peat in the soil at Rotterdam, the bad parts are removed by a dredge, and then the dredged cut is filled with sand at least 6 ft. deeper than the base of the caisson. Then the caisson is towed into place, and, by means of a tongue and groove on the ends, the floating mass is brought into alignment, the free end being controlled by tackle. Next, by opening valves, the caissons are sunk on the prepared bed of sand, after which the water is pumped out of the front row of cells and these are filled with concrete; the rear row of cells is filled with sand to save expense.

In the older construction, the site was dredged, then a brush-mattress facing was placed, and this was allowed to stand for one or two years to secure thorough settlement of the mass on and to the rear of the mattresses. Afterward wood piles were driven through the mattress and, by using a special diving bell, were cut off and capped below low water; then they were decked with wood or reinforced concrete upon which was built the quay wall. The floating, reinforced concrete caisson method was stated to cost less per linear foot than for piles decked with concrete and more than for piles decked with wood. Practically, the cost may be said to be the same, with the decided advantage of a nearly monolithic wall.

Where rock is convenient, this method might be used with advantage: Having made the dredged cut, next place along the front line of the caisson one or more rows of piles, which might be driven to, or cut off, say, 2 ft. below, the grade of the bottom of the caisson; and then rock fill to grade; the object of the piles covered by rock would be to prevent any rotation of the concrete block around the outer toe due to a thrust from the shore side.*

Compressol.

In Paris the writer witnessed a demonstration of the Compressol method of preparing foundations. By a sort of pile-driver, a heavy conical weight is dropped repeatedly upon the soil, and, when the desired depth is obtained, small stones are dropped into the hole, and, by special forms of conical weights, are forced down and out into the soil, after which concrete is rammed into place by the same means. A completed pile will generally be 1 m. in diameter and have a bulb at the base.

One contractor in Belgium has eighteen of these machines at work. There are many places where such a system might be used; its special value would appear to be in a firm soil of loam requiring piling, and where the pile heads would be above the permanent water plane. Whether, in point of economy, it presents any advantages over some of the patented American systems is not known to the writer. It is asserted, by those advocating the Compressol system, that, owing to the thorough compression given to the ground, both laterally and vertically, combined with the mushroom-shaped base of the pile, it is capable of sustaining two or three times as much load as piles used in American systems. Certainly there must be considerable merit in it, otherwise it would not be used so extensively.

Wood Piles Driven at an Angle.

At Bremen, and notably at Bremerhaven, it is the practice to construct much of the new work in the dry. The area to be enclosed is stripped, by land dredges and cars, to 3 or 4 ft. below low water, and is kept dry by pumps. Along the proposed line of quay wall, two single-rail tracks are laid, about 33 ft. apart; these serve to carry a pile-driver which traverses a carriage supported by the two rails. The pile-driver is arranged to swivel in two directions; thus the driver can be placed with great precision and dispatch, and piles can be driven at any desired batter. It is the practice to select long piles, and first drive each tenth bent of piles. The ways are marked with a metric

* A good illustrated technical description of the work at Rotterdam may be found in *De Ingenieur*, July 20th, 1907; The Hague, Holland. This has not yet been translated into English.

scale, and an attendant records the position of the pile at each fifth or tenth blow. From these data piles of suitable length are selected for the intervening nine bents, and a similar record of driving is kept. Should one or more piles in a group settle too much during the last ten blows, a longer pile is driven in the bent or in the adjoining bents to give additional bearing power. The bents, when driven, batter about 1 on 5, like the letter A. At the cross of the A, two strong timbers are bolted to the framed piles, and longitudinal wales are bolted to the piles and side pieces, and a 6-in. wood floor completes the foundation. Upon this foundation a quay wall of rubble-faced concrete is built, and is bonded to the rear piles by tension rods, after which the area is opened and excavated to the full depth by dredges. A construction of this kind resists most effectively the thrust from the landward mass of earth, the outer piles being compressed and the rear piles in tension.

Mr. Claussen, Dock Engineer at Bremerhaven, says that he considers 15 tons per pile a safe load when used in tension. A number of long walls, such as the sides of a dry dock, quay walls, and locks in use for ten years, were remarkably straight, and offered strong evidence as to the value of this system of construction. This could not be used if limnoria or teredo were present, unless the piles in the outer row were covered by a concrete wall; but it might have application for mooring bits, etc., where the piling is protected, and perhaps would be advantageous where it is required to erect a temporary bulkhead and load the ground landward for a year or more, so as to consolidate it before commencing the permanent construction of a quay wall.

Cranes.

In nothing is the difference between the United States and Europe so marked as in the non-use here and the general use there of power cranes, usually hydraulic, but often electrically operated.

It is quite common for a merchant to visit Europe and, having noted the many excellent things to be seen, quite naturally think that among the improvements required in the United States are cranes.

In Europe several thousand cranes are installed; upon an average there is one crane to 283 ft. of quay wall, and their usage is quite variable.

At Marseilles, in 1903, thirty-four hydraulic cranes, having a capacity of 2 750 lb., worked 121 days per year for each crane, on a 9-hr. basis, and averaged 25.2 loads per hour. Ten cranes of double power, but working at one metric ton, worked 115 days; and working at 3 tons, 16 days each per year.

The whole number of loads was 1 116 980, which gave an income of 166 312 francs, or 2.92 cents per load. The average income of a crane was about $745 per year, which includes the power and the crane operator. This port has perhaps a more intensive use of cranes than those farther north, where generally only one crane out of four or five is observed to be working.

At London it is alleged that cranes do not pay interest upon their cost, but the ship owners insist that the dock owners have them and do not use them, except perhaps for a small part of a cargo.

It is possible that the crane idea is a survival from the days of sailing ships, when they were first introduced and were really required; next they were copied by other places, and by sheer inertia dock owners persist in having them. The people of the United States are quick to seize and appropriate a good idea, and the fact of the non-use of cranes, compels a strong belief that the appliances used—the ships' tackle and the stevedores' hoists—are ample. It has been asserted that a difference in the nature of the business done in the different countries is responsible for the general use of cranes abroad; but this does not appear to be a reasonable view of the matter.

The writer does not wish to give the impression that cranes are not useful; on the contrary, he believes that a partial adoption of the plan in the United States—to the extent, at least, of having wharf cranes which would serve to lift anything in excess of the capacity of a ship's tackle—might prove useful.

At Liverpool many cranes are supported upon the top of the warehouse front wall and a rail upon the peak of the roof. This is a very excellent disposition, because the crane is always out of the way.

Dock Strikes.

There is and has been much trouble at many ports in Europe from labor strikes, and, upon the whole, these are probably worse than any that have happened in America.

It is difficult for a stranger to form an accurate estimate of this subject, because he hears various versions of the cause and nature of the trouble.

The writer questioned the officials of the ports visited, and, as a check, obtained the views of the marine underwriters and sometimes the Jesuit Fathers, who are usually in a position to estimate the troubles impartially. The general unrest appears to have below it a raising of the standards of living, for the cost of living and the wants of the laborer have increased faster than his wages. On the other side, the employers assert that, in view of the serious competition between ports, a small amount of extra cost will cause a diversion of business, therefore they oppose an increase of wages.

The nature of the work is irregular, there are periods of great activity followed by lessened opportunity for work. Genoa has been greatly troubled in the past by strikes, and the present port governing board, which is closely modelled on the lines of the Liverpool Dock Trust, has assumed that it has the power to settle such questions by creating a permanent force of laborers who perform any sort of service, from discharging cargo to road making, if required. The board takes on extra men for short periods to cover emergency cases. The laborers receive less pay than men engaged by private employers, and it is said that, on account of a strong union organization, they do less work. The outside criticism was, that the laborers practically dictated hours and terms to the dock board. The experiment is an interesting one, and its outcome will be a matter of interest.

During the writer's visit, a mild strike was in progress at Le Havre, and at Antwerp a severe one which required the importation of some 2 000 English strike breakers, who, for their protection from assault, were housed on vessels in the harbor. In August the strike culminated in burning the timber yards and required calling out the troops in order to save the city from fire and quell the rioters.

To the writer, the wages seemed to be too low, but unless concerted action were taken by all the competing ports, it would be difficult to effect a raise. It does not appear that the form of government has any very decided influence upon strikes or violence arising therefrom. The striker is usually a voter, and some one in authority may need his vote. About the only safe deduction that can be made is that a small aggressive minority comes pretty near getting all it desires.

Average Data.

The following average data concerning foreign ports, compiled from a paper,* entitled "Results of Investigation Into Cost of Ports and Their Operation," by Mr. Elmer L. Corthell, and corrected by the writer in a few particulars, presents in a summarized form information of considerable interest:

The ports included are London, Liverpool and Birkenhead, Glasgow, Bristol, Hamburg, Rotterdam, Le Havre, Dunkirk, Bilbao, Antwerp, Bremen and Bremerhaven, the Tyne Ports, Marseilles, Amsterdam, Lisbon, Bombay, and Buenos Ayres.

Total cost of port improvements to 1906	$764 388 000
Registered tonnage, entered and cleared in one year (about 1905-1906)	185 652 000
Goods dealt with in one year, 1905-1906, in long tons of 2 240 lb., approximately	143 000 000
Gross revenue in one year (about 1905-1906)	58 206 000
Expenses " " " " " "	29 003 000
Net revenue " " " " " "	29 203 000
Gross revenue per registered ton " "	31.4 cents
Gross revenue per long ton " "	40.6 cents
Quayage length, in feet	1 192 000
Quayage length, in miles	227
Approximate length of rail, in miles	930
Length of rail divided by length of quayage	4.11
Ratio of area of sheds to quayage (44 467 770 sq. ft. of sheds to 1 067 120 ft. of quay)	41.1 to 1
Average weight of goods dealt with per year per linear foot of quay, in long tons	120
Percentage of gross income on capital cost	7.615%

One crane to each 283 ft. of quay wall.

General Reflections on Commerce.

The greatest factors to-day in the material and moral development of the world are transportation and commerce. By their agency, people and their products are moved from a region of a lesser to one of a greater use and demand. They are the greatest of all the civilizing

* *Proceedings* of Permanent International Navigation Congresses, Brussels, 1907.

agencies, because they promote an exchange of thought as well as of commodities.

The growth of modern commerce is closely interwoven with the development of the steam engine, railways, and electricity. It is a question of power and its applications, and its present enormous dimension is largely the work of the engineer. Its growth has been phenomenal, and is ever increasing; it is far more rapid in its rate of increase than that of population, which means increased wants upon the part of the people, and increased ability to buy and to enjoy. Its future is a question of great philosophical interest; but, until the people of the world are raised to the general level of intelligence of the more favored nations, it is reasonable to believe that its march will continue, and that day is so far distant that it does not immediately concern the present age. Until then, the signs point to an ever-increasing scope in the functions of the engineer.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE FLOOD OF MARCH, 1907, IN THE SACRAMENTO AND SAN JOAQUIN RIVER BASINS, CALIFORNIA.

Discussion.*

By Messrs. Luther Wagoner, H. H. Wadsworth, and George L. Dillman.

Mr. Wagoner. Luther Wagoner, M. Am. Soc. C. E. (by letter).—This paper is an extremely valuable contribution, and presents the main facts of the flood with great clearness. The opening statement, that it was one of the most destructive floods that has ever occurred in California, while probably correct in a financial sense (and due to the fact that there was more property to be damaged than at previous floods), implies that it was about the greatest flood on record. The authors say:

"It is doubtful if any combination of causes or conditions will ever produce a larger rate of delivery of water to this valley for a 4-day period than occurred during the flood of March, 1907."

The writer believes that it would be unsafe to accept this statement as a basis for planning reclamation and flood prevention, unless it is qualified by a large factor of safety. It is generally believed that the flood of 1862 was greater in volume of water discharged into the basins and bay. In 1890 the writer, while engaged upon plans for the La Grange Dam on the Tuolumne River, found a well-preserved record of the 1862 flood near the present dam and 70 ft. above the bed of the stream. The record was in the shape of rounded pieces of wood and

* Continued from April, 1908, *Proceedings*.

bark, fir, pine, tamarack, and juniper, showing that these pieces came from the higher regions. They were found in a talus of loose rock, and were doubtless carried into the void spaces by eddies and lodged there. Almost opposite, and across the river, a similar deposit was found, and at almost the same level. This led to a search along the river gorge above, where several similar records were found. Levels were taken, and connected with several cross-sections, and these, combined with the known high water at La Grange, about 1 mile below the dam, where the channel is wider and more regular, led to the conclusion that the maximum discharge was 130 000 cu. ft. per sec. This was based on the slope and Kutter's formula ($n = 0.040$), and the dam was planned to be able to discharge that volume over it; this corresponds to a run-off of 86.7 cu. ft. per sec. per sq. mile. Mr. Wagoner.

In 1895 the writer found a similar record on the middle fork of the American River near Volcanoville, from which, by the same methods, a run-off greater than 100 cu. ft. per sec. per sq. mile was deduced. (The original notes of both the foregoing records were destroyed in the San Francisco fire, two years ago.) Estimates have appeared in print in which the flood flow was given for the whole basin of the American River at 250 000 cu. ft. per sec., and even more. The record given above, applied to the whole water-shed above Fair Oaks, would give a discharge of about double the authors' 93 000 cu. ft. per sec.

While it may be true that a flow of 782 000 cu. ft. per sec. for 4 days may not be exceeded, there are two points to be considered. The flow from the San Joaquin region might occur as in 1862, and in combination with a 1907 flood on the Sacramento, in which case the quantity would be greatly exceeded. Again, suppose the rivers were leveed in accordance with the plans of the Engineering Commission of 1904, it would not require a 4 days' sustained flood to overtop the levees, and the probabilities are always in favor of the shorter but perhaps more intense run-off.

There is an average difference of a month in the melting of the snow upon the Columbia and Snake water-sheds, yet in 1900 it melted on each at the same time, with the result of backing up the Willamette and flooding Portland to a depth of several feet. This flood was sustained for more than two weeks.

The writer concurs in the conclusions of the authors, that relief from damage by floods must be sought in storage to relieve the peak of the discharge. Storage of débris is equally important, if the bed of the stream and navigation interests are to be preserved, and future studies should be upon the lines of effecting both water and débris storage.

It is the writer's belief that this can best be accomplished by loose-rock dams, backed with earth and waste on the up-stream face and with the down-stream face secured to the mass by suitable anchors, so

Mr. Wagoner. as to allow the passage of floods over the crest of the unfinished dam during construction. When completed there would be an ample spill-way around the dam, so that it could never be overtopped. Such dams would have to be of great height, from 400 to 500 ft., or even greater, because they would usually be located in gorges, and it might require from 300 to 400 ft. of permanent elevation to create a sufficient reservoir area, after which the increase of storage would be rapid. The only serious objection to such a type is the cost, but it can be shown that in the end it would be economical, because the desired regulation could thus be obtained (and, incidentally, the storage of débris), as well as power and irrigation.

It is beginning to be recognized that the proper treatment of this subject is a serious matter, and that the cost may reach $100 000 000 or more. From the analysis presented in the paper, it appears that there should be a storage of about 3 000 000 acre-ft. in order to give the required relief on the Sacramento water-shed alone. Such storage could be valued as follows: (*a*) Relief of peak load and flood prevention; (*b*) storage of débris; (*c*) preservation of the channel of the river; (*d*) irrigation; and (*e*) power. When all these possible uses are admitted and properly valued, it can readily be seen that a high cost per acre-foot of storage is permissible.

Mr. Wadsworth. H. H. Wadsworth, M. Am. Soc. C. E. (by letter).—Although disastrous to the agricultural interests of so large an area, and to all the transportation lines of the valley, in some respects it may be said that this flood occurred at a very opportune time. The reclamation of the overflowed lands of the Sacramento and San Joaquin Valley has become a subject of vital importance to the future development of California, and the newly-awakened interest in the improvement of the navigation of these streams, and the movement toward the proper correlation of the various interests affected by the flow of the streams, from their sources in the mountains to their discharge through the Golden Gate, make the new standard set by this flood of great importance.

The country is to be congratulated that the Geological Survey had a sufficient number of gauging stations established to obtain so many data in regard to the flood; and the authors are to be complimented on having presented them so promptly in such shape as to be of practical use, and show the magnitude of the problems involved.

A press of work, in connection with surveys looking to the regulation and improvement of these rivers, prevents the writer from discussing at this time more than one or two of several points or questions which have occurred to him, and these only very briefly.

As stated by the authors, previous estimates of flood flow of the Sacramento River have been greatly exceeded during this flood. In the case of the Yuba River, the maximum flow which has been assumed since the inception of the project for restraining débris in the bed of the stream, and which was used in designing the several works forming part of the project, is 125 000 cu. ft. per sec., or 25% in excess of the maximum observed flow at the gauging station of the Geological Survey. Before the failure of the dam, known as "The Barrier," this structure served as a weir for measuring the flow of the river, though its coefficient was very uncertain after it became backed up with tailings to its crest. The high-water marks left by the river during the night when the failure occurred indicated that the assumed maximum was nearly if not quite reached. Based on observations and estimates of flow at this point and at a few points on mountain streams carrying the drainage from comparatively small areas, Table 29 was prepared by the writer as a guide in determining the required capacities of spillways or canals to carry flood water around or away from dams erected for the storage of mining tailings. Mr. Wadsworth.

TABLE 29.—ASSUMED MAXIMUM RUN-OFF, SIERRA NEVADA STREAMS, BASED ON THE FLOOD OF MARCH, 1907.

Drainage Area, in Square Miles.	Maximum Run-off, in Cubic Feet Per Second.		Drainage Area, in Square Miles.	Maximum Run-off, in Cubic Feet Per Second.	
	Areas below elevation 4 000 ft.	Areas above elevation 4 000 ft.		Areas below elevation 4 000 ft.	Areas above elevation 4 000 ft.
1	544	408	40	8 650	6 490
2	915	686	50	10 200	7 670
3	1 240	930	60	11 700	8 800
4	1 540	1 150	70	13 200	9 860
5	1 820	1 360	80	14 500	10 900
6	2 090	1 560	90	15 900	11 900
7	2 340	1 760	100	17 200	12 900
8	2 590	1 940	200	28 900	
9	2 830	2 120	300	39 200	
10	3 060	2 290	400	48 700	
20	5 140	3 860	500	57 500	
30	6 970	5 230	1 000	96 800	

Table 29 is applicable to drainage areas of the character of the Bear and Yuba Rivers, and of such portions of other drainage areas as do not contain broad flat valleys. Of course, it would not apply to portions of the Feather River drainage area, containing the Sierra, American or Indian Valleys, or Big Meadows.

Considering for a moment the effect of mining débris on floods, it should be borne in mind that only on the Yuba River is it likely that the flood plane will continue to rise materially owing to this cause. The beds of the American, Bear and Feather Rivers (Feather above the mouth of the Yuba) have reached as near a state of equilibrium as is common for streams flowing through alluvial formations. The curtailment and regulation of hydraulic mining has largely stopped

Mr. Wadsworth. the accumulation of tailings in the torrential tributaries of the Yuba, and many of the canyons of these tributaries, which a few years ago were filled with tailings to a depth of from 20 to 60 ft., have been scoured out to bed-rock. Below the gauging station of the Geological Survey, at The Narrows on the Yuba River, there is a vast deposit of mining tailings standing on slopes which succeeding floods will continue to readjust; but any further extensive rise of the flood plane at Marysville does not seem probable.

In connection with the subject of partial control of the flood flow of the Sacramento River by reservoirs, it is interesting to note that the total capacity of five flood basins of the Sacramento Valley, as computed by the authors, amounts to 3 731 000 acre-ft., and that the effect of these basins was to delay the arrival of the flood crest at Rio Vista about 4 days. Had there been levees sufficient to confine the river to the channels, how much higher stage would have been reached at this point?

Of the storage reservoirs located and surveyed by the United States Reclamation Service, the total estimated capacity is 4 817 000 acre-ft., or about 1 000 000 acre-ft. in excess of that of all the Sacramento Valley flood basins. The latter are much more likely to be empty, or at least to have considerable capacity for the storage of flood waters when a flood occurs, than are the former, since, owing to the uncertainty of further large run-off before the dry season, it would jeopardize the agricultural interests dependent upon irrigation to leave these reservoirs nearly empty until after the middle of March. The worst flood of the season is likely to occur after that time, as was the case with the flood of 1907. On the other hand, no flood may occur, as has been the case during the season of 1908.

In whatever way the control of ordinary floods may be effected, it will very likely be found that:

"The task of rectification and enlargement of channel necessary to pass such floods as that of March, 1907, is so great as to make it economically impossible."

Mr. Dillman. George L. Dillman, M. Am. Soc. C. E. (by letter).—The conclusion expressed in this paper is that mountain storage will be the ultimate solution of the flood problem of the Sacramento Valley. With this, the writer would take issue. He here states that the storage outlined is impracticable; also, that, if accomplished, it would prove inefficient, insufficient, precarious, and temporary. These hard names do not all apply to each case, but some of them apply to each case, and all to a few of them.

The reservoirs mentioned, except Big Meadows and Clear Lake (which will be built by private corporations for power purposes), should never be built for flood regulation. The reasons are various. Some of these reservoirs are located where their water capacity would diminish rapidly by filling with débris, ultimately reaching zero. This applies specially to the Stony and Putah Creek Reservoirs, and also to Iron Cañon. Mr. Dillman.

Some of these proposed reservoirs would flood lands which are too valuable to buy for such purposes. This is specially true of Indian Valley, where the lands which would be flooded are the finest kind of dairy lands, and include three small towns, with improvements which would make the cost great. From an irrigation standpoint, this storage is not needed, and would destroy more agricultural land than it would reclaim, less the cost of reclamation.

The Big Valley Reservoir—two-thirds of all the storage proposed—would be fed by an arid country. No doubt, its capacity compared with its cost is quite favorable, but some water should be allowed to pass to users between Bieber and Fall River. The evaporation would be large, and it would probably take several years to fill it. The authors' figures show that the 4 days' abnormal flood would fill 6% of it, if all was stored. During part of every year, the inflow would not equal the evaporation.

There is no reason, from an irrigation standpoint, for making the Feather River storages. Feather River is not much used for irrigation, by reason of expensive diversion. At low stages there is water without storage for any who will divert it; and there are no adverse claimants for it.

These storages are precarious. The high-water mark for years on the East Branch of the Feather was made by the failure of a small dam above Indian Valley. The flood of 1907 caused the failure of a dam on the Yuba, when the waters flooded large areas, some going south and breaking across Bear River, finally reaching American Basin. The possible damage by the failure of a reservoir dam on top of a flood can hardly be estimated.

The paper assumes the removal of water from the crest of a flood. This would hardly be the fact. This flood's crest came after weeks of flood, and it is fully supposable that the reservoirs would have been filled and held full long before the time had arrived for such beneficial effects. The location of these reservoirs averages more than 100 miles from the seat of damage. What man is wise enough, during a period of such storm and flood, to say when the psychological moment arrives for closing gates and making storage?

Taking from Table 27 the volume available for storage, or the capacity of the reservoirs, the total storage for this flood would have been 931 300 acre-ft. The side basin capacity is given as 3 775 000 acre-ft., or more than four times the mountain storage. This basin storage, like the mountain storage, would be largely made prior to the crest of the flood, but there is another factor. At about the time when the crest of a flood occurs, side levees break. In 1904, the Edwards

Mr. Dillman. break, on the east bank of the Sacramento, and in 1907, the Kripp break, on the west bank, gave great relief, as far as flood heights on the lower river go. The first poured into the Sacramento Basin and discharged into the San Joaquin through the Mokelumne. The second poured into the Yolo Basin and discharged into the Sacramento through Cache Slough. These side basins are not only storage reservoirs, but are great by-passes, through which flows a volume sometimes greater than the main stream. At flood times Yolo Basin is a stream 10 miles wide, flowing rapidly. The basin relief is from flow more than storage. It is at hand, and acts automatically at the right time.

Years ago, an avalanche of débris from hydraulic mining started toward the valley. Mining has stopped, but the débris is still coming. Diversion of waters, whether natural or assisted, lessens the current, increases deposits of débris, and raises the beds of streams. This silt problem cannot be divorced from the flood problem. As a rule, the stream beds are rising, so that the same volume of water reaches a height increasing with time.

From all the foregoing it would seem that mountain storage is not advisable for flood control. The valley problem is a complicated one, including the protection of lands, the navigation of streams, the drainage of flood water, and the care of the débris. Different solutions have been proposed.

The State Board of Works, about 1890, proposed an elaborate system of by-passes through the valley. This never met general approval. Such an installation would increase silt deposit in the streams, raise the flood and ground-water planes, and increase the area affected by floods. The ultimate effects would be detrimental to agriculture, navigation, and flood conditions.

In 1905, a Board of Engineers recommended a plan based on concentration, instead of diversion, of waters. The plan met general approval as to method. The cost, however, was great, though the agricultural land reclaimed would have been worth several times the outlay. The interests are so many and so divided, each wanting to benefit out of proportion to the outlay, that no action has resulted.

It seemed once that the débris problem might be divorced from the others by a series of débris barrier dams. The failure of the first one, on the Yuba, settled it otherwise.

The mountain storage solution has been brought up from time to time, but has never stood close analysis. To obviate the floods by preventing them, sounds all right until the illusion is dispelled by the cost, and the amount of storage is compared with the flood volume. The valley must take care of the floods. They will come. Mountain storage increases the risk, with very small compensation in possible results.

A rational solution lies in adequate waterways, made and maintained by a combination of dredging and levees. If plans to this end Mr. Dillman. could be adopted, and all future work be obliged to conform to them, the end would be reached in time, without enormous initial outlay. As the plan was executed, cross and other auxiliary levees could be gradually abandoned as they became useless. River levees could be strengthened annually by dredging, as reclamation decreased the basin effect on floods. The levees would protect agriculture and confine the waters. Dredging would lessen the necessary height of levees, furnish material for them, and assist in the débris problem. The necessary height of levees, and the depths and widths of channels are the main questions. Rectification of alignment and foundations are important auxiliaries. As far as the Sacramento is concerned, débris has largely solved the foundation problem. The unstable peat bogs have been silted up, and now sustain enormous levees on Grand Island and elsewhere. The San Joaquin foundations are more serious problems, because the silt is deposited long before the waters reach the delta.

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PAPERS AND DISCUSSIONS

ERECTION OF THE BELLOWS FALLS ARCH BRIDGE.

Discussion.*

By F. W. Skinner, M. Am. Soc. C. E.

Mr. Skinner. F. W. Skinner, M. Am. Soc. C. E.—Everybody admires a great engineering work which is designed and built conservatively, thoroughly, slowly, and deliberately, sometimes even ponderously. Everything is done with great care and forethought, and with perfect apparatus. All the appliances are complete, and the entire construction is worked out in the most minute, most solid, and most monumental manner.

There is no doubt that such works are great engineering triumphs, but, in achieving them, the engineer often deviates very little from established precedent, even though the construction is on a larger scale than usual, and the speaker thinks that such construction does not lead to professional progress, or at least to as great an extent as desirable.

All admire the great Forth Bridge, with its unprecedented span, but its weight and cost were great, and it took a long time to build. This is true of many other engineering works. The speaker has in mind an illustration, doubtless quite familiar to many, showing that extremely careful and costly engineering constructions are sometimes inadvisable, and by the use of more rapid, more daring and original methods, work can be executed which might otherwise fail.

In New York City, not very long ago, there was an important piece of engineering work, on which, before completion, radical repairs or

* Continued from April, 1908, *Proceedings*.

Fig. 1.—Erection of the Eads Bridge.

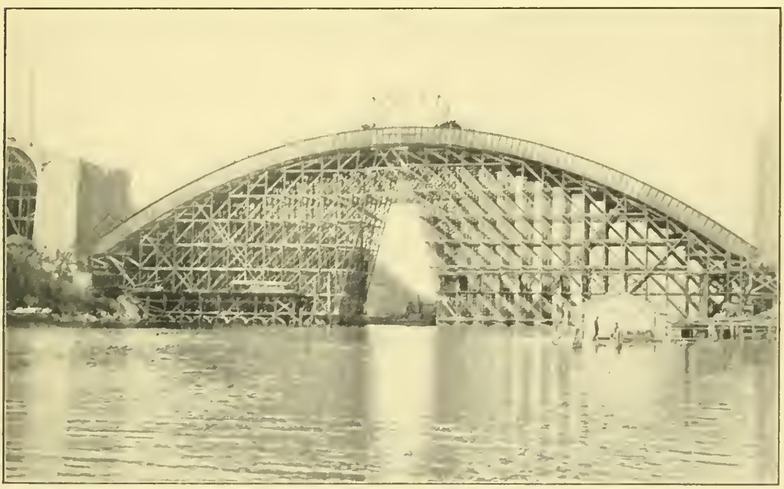


Fig. 2.—Erection of the Washington Bridge.

changes were found to be necessary. An estimate was obtained, from an engineer eminent in that particular class of work, which involved the expenditure of somewhat more than $1 000 000. Eventually, the changes were effected for $100 000, by an entirely different method, devised by a member of this Society. This method was considered daring and impracticable until the contrary was demonstrated by its unqualified success. Therefore, considering the greatest good to humanity, the greatest advances seem to be made by, and the greatest praise to be due to, the engineer who accomplishes the best results, with the least money, with the greatest safety, and in the quickest time. Mr. Skinner.

Judged by that standard, and by the results obtained, the designers and constructors of the Bellows Falls Arch Bridge rank very high in bridge building.

This span of 540 ft., with its substructure, cost only $46 000, or less than $3.50 per square foot of floor. Such a result is unprecedented for such a long span. The erection time is also unprecedented, for it required only 28 working days. In Europe a year or two would generally be taken in building a structure of that kind.

There are several features in the design of this arch which commend themselves to every bridge engineer. Among them, the scheme of making the crown connection with plates proved a very happy device, not usual in ordinary practice. The construction of the crown panel, too, is advantageous.

It is to be regretted that the author did not give the details of the members and connections, and it is hoped that he may yet present a paper which will deal with these features.

In the erection of the Bellows Falls Arch, a happy mean was established between a self-sustaining structure and a mass of falsework by giving it an economical amount of temporary support, and the result abundantly justified the means.

Another feature in the erection of this arch is its exemplification of the fast growing tendency to utilize the great advantages of steel derrick booms over other apparatus for handling heavy steel members. These booms were 60 ft. long, but Mr. Rights, if he were repeating the work to-day, might use 100-ft. booms, as he has in other recent erections.

The Bellows Falls work was admirably designed, and served its purpose thoroughly well. The highest compliment that can be paid to the bridge is to compare it with some of the notable bridges of similar type; because the methods used in their erection will show the excellence of this one much more effectively than any assertions.

The following description covers fairly well the erection features of all the large arch spans which have been built:

The famous Eads Bridge—"the Father of Arches"—had three spans, two of 537 ft. and one of 552 ft., which were noted as being

Mr. Skinner. the longest railroad spans, and for a very long time remained the longest arches. They were made with four ribs or trusses with steel-stave chords 18 in. in diameter, and were erected by the balanced, guyed, cantilever system. The trusses of adjacent spans were built out simultaneously from the piers, and were supported by an elaborate system of guys or back-stays reaching from the successively erected panel points of the trusses back to the tops of pairs of falsework towers on the piers.

Each tower was a pyramidal skeleton, 50 ft. high, with a 24 by 24-in. oak mast, 12 by 12-in. batter legs, and 18 by 24-in. oak sills set on special hydraulic jacks. The skewbacks were tied together by horizontal anchors through the piers, which made them self-sustaining for about one-quarter length, beyond which the two middle trusses were supported by eye-bar guys with sleeve-nut adjustments provided with very elaborate falsework supports, set on top of the trusses to diminish the deflection. The center panel connections were made by using the guy adjustments and by the operation of the jacks under the towers, which moved the latter vertically 6½ in. Both these means together, however, were not adequate to provide for extreme temperature variations, and there was great difficulty in making some of the connections. The chords were packed in ice, and special sections were cut to fit the last panels.

At the shore spans special anchorages had to be provided to secure the ends of the back-stays. One of them was made with castings set in a shaft excavated 30 ft. in solid rock, and the other with a horizontal oak girder, 4 ft. square, engaging a quadruple row of 12 by 12-in. sheet-piles driven in sand in the bottom of a deep excavation. All four ribs of each span were built simultaneously for the first three-elevenths of their length, after which, work on the outer ones was suspended until the center ones were completed. These then served as platforms from which the remaining outer ones were erected. The materials were put in place by a hand-power traveler, advanced by rack and pinion, and equipped with four derrick booms. The maximum clear height above high water is 73 ft. 9 in., and the cost, including that of the difficult substructure and approaches, was about $10 000 000.

Fig. 1, Plate L, is to be valued for its historical associations. It shows the Niagara gorge with the three great types of long-span bridges, all built without falsework. In the foreground may be seen portions of the original railroad suspension bridge of 800 ft. span, with steel stiffening trusses which replaced the original trusses of wood and iron some 20 or 30 years after the construction of the bridge. In the background is the second cantilever built in America, the famous work of C. C. Schneider, Past-President, Am. Soc. C. E. Partly completed, and in the same plane as the suspension bridge, is

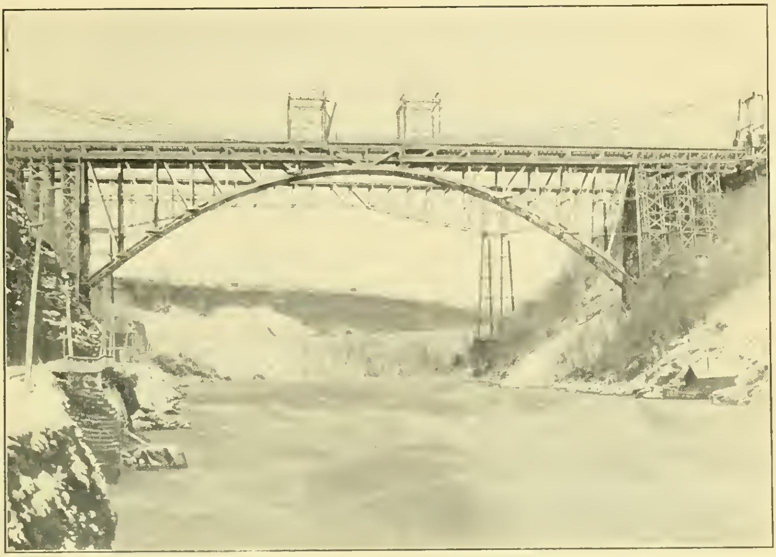


Fig. 1.—The Railroad Suspension, Railroad Arch, and Cantilever Bridges, at Niagara.

Fig. 2.—The Highway Suspension Bridge at Niagara Being Replaced by a Steel Arch.

the double-deck, 550-ft., spandrel-braced, arch span, designed by L. L. Buck, M. Am. Soc. C. E. This arch was also built by the guyed cantilever system. The trusses were assembled entirely by small overhead steel gantry travelers, which ran on the new top chords, outside of the old suspension bridge, and allowed the traffic to be maintained on both tracks of the latter while the new bridge was being built. Mr. Skinner.

The adjustments for the connection of the center panel were made by slightly revolving the semi-arch trusses about their skewback pins by using a toggle inserted in a chain connecting each top chord with a temporary anchorage of I-beams bedded in concrete in chambers excavated for the purpose in the solid rock. The anchor chains were made with eye-bars proportioned for stresses of 1 000 000 lb. per truss, and were connected to parallelograms of eye-bars with vertical screw diagonals operated by sixteen men to a capstan-head, thus raising or lowering the span, as required.

The construction of the railroad arch was soon followed by that of another arch, for highway traffic, just below the Falls. It has a span of 840 ft., is about 200 ft. above the water, and was built in very much the same manner, but, as there was no horizontal top chord to form part of the anchorage, the guys for the cantilevered semi-trusses were lines of eye-bars attached to alternate panel points on the top chords as fast as they were built out. These were adjusted by the same toggle which had been used on the railroad arch.

This bridge was also built in the plane of an existing suspension bridge, but in this case the old structure was used for the support of the very light travelers which handled the members for the arch trusses, and as none of them weighed more than 5 tons, the load imposed on the old bridge was very small. The toggles were only required to lower the semi-arches, and this they accomplished with a force of twenty men on each. The bridge weighs 3 651 000 lb., and was erected by 100 men in about 3 months.

The Washington Highway Bridge across the Harlem River, New York City, has two 510-ft. main spans, each with six two-hinged plate-girder arch ribs of 90 ft. rise and 133 ft. clear height above high water. The ribs have a uniform depth of 13 ft., and the flanges are curved to parabolic arcs and support transverse bents, 15 ft. apart, carrying the floor platform. The spans were erected on framed-trestle falsework on piles, and materials were delivered from a service track at the skewback level, parallel to the bridge axis, to the erection travelers on the top flanges of the arch ribs. The travelers consisted of pairs of adjustable stiff-leg derricks which erected the six ribs simultaneously from both skewbacks to the crown and then moved back to the ends of the span on the permanent floor which they erected in advance. The channel span falsework had inclined bents providing an 80-ft. center opening for navigation. The ribs were swung by jack-screws on each falsework bent. Each span weighs about 1 670 tons.

Mr. Skinner. The Victoria Bridge, across the Zambesi River, in Africa, is a double-track railroad structure with two-hinged spandrel-braced arch trusses of 500 ft. span, 400 ft. above the water. The connections are riveted, but, for erection, were provided with 2-in. auxiliary pins. The trusses are in battered planes, are 105 ft. deep at the skewbacks, and 15 ft. deep at the crown. Half of the 1 650 tons of steel in the bridge was carried across the gorge by a cableway of 10 tons capacity, with electric mechanism and an adjustable shear-leg tower. A line carried across the gorge by a rocket was the first step in the erection of the cableway.

The arch trusses were erected, in a manner similar to that used in the Niagara Railroad Arch, as anchored cantilevers, fulcrumed on their skewback bearings, and the reactions were provided for by multiple anchor cables carried through U-shaped inclined tunnels in the solid rock, and adjusted by nuts and screws.

The erection travelers were simple platforms, moving on the horizontal top chords, and provided with 30-ft. derrick booms. To receive the last chord section, the center panel opening in the top chord was adjusted by hydraulic jacks, and the last bearing was made with planed shim plates. A safety net was at first swung under the travelers to protect the workmen, but was found to make them nervous. Contrary to American practice, the bridge was completely assembled in sections, at the shops in England where it was fabricated, thus involving considerable extra expense. The trusses were erected in about $5\frac{1}{2}$ months.

The double-track, Müngsten, or Kaiser Wilhelm Bridge, across the Wupper River, in Prussia, is 350 ft. high, and has a clear span of 525 ft. The riveted trusses are battered 1 : 7, and were built as guyed cantilevers after the completion of the skewback towers erected on elaborate and heavy falsework, and the construction of the approach spans, erected on falsework trusses. These were assembled on the surface of the ground and hoisted bodily to position on top of the permanent viaduct towers, a proceeding which might apparently have been as well applied to the permanent spans, even as the towers might have served as their own falsework.

Both arch trusses and high-level roadway trusses, supported by spandrel posts on the arches, were erected simultaneously as cantilevers. Materials were delivered on a "low-level" service track on a falsework bridge, about at the skewback level, high above the surface of the water. The cable guys were adjusted by hydraulic rams, and hydraulic rams were inserted in the crown panel to release the lower chord skewback wedges which had been inserted to compensate for deflection. The crown joint was riveted, and then the final stresses were adjusted by hydraulic jacks at the skewbacks. The bridge weighs 5 622 tons, cost $1 230 000, and was erected in 22 months.

The single-track Garabit Viaduct, in France, has one two-hinged,

FIG 1.—THE MUNGSTEN, OR KAISER WILHELM BRIDGE.

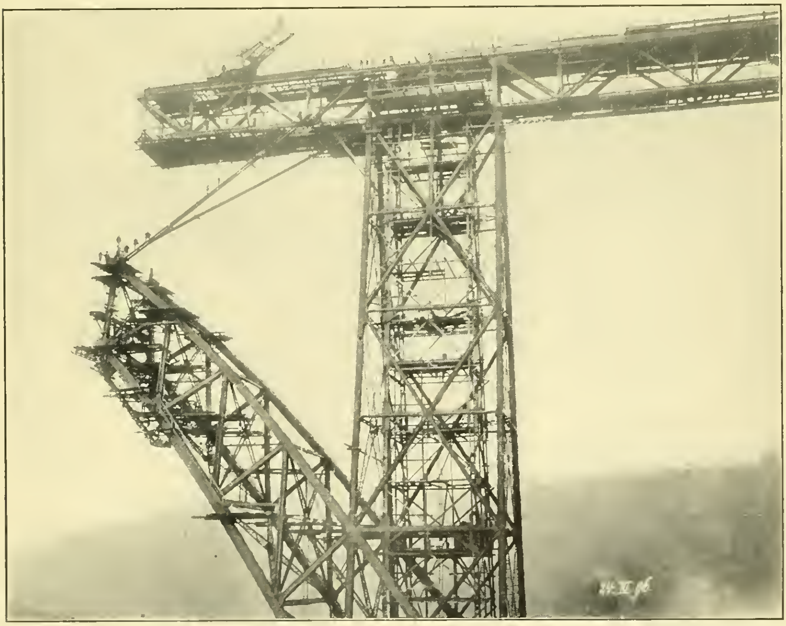
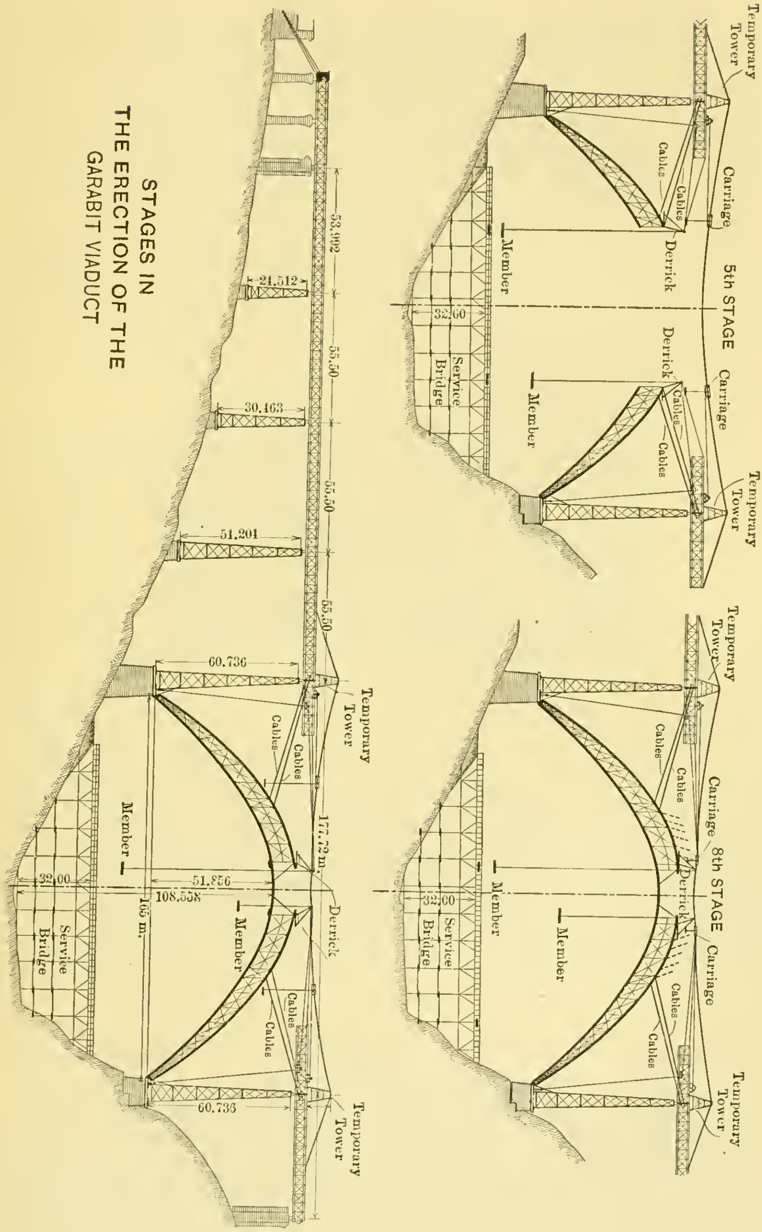


FIG. 2.—ERECTION OF THE MUNGSTEN, OR KAISER WILHELM BRIDGE.

Mr. Skinner.



STAGES IN
THE ERECTION OF THE
GARABIT VIADUCT

Fig. 3.

Mr. Skinner. 541-ft. arch span, 406 ft. high, with riveted trusses having a great rise and carrying spandrel towers which support the roadway trusses of about 125 ft. span. The skewback and viaduct towers were first erected, and then the approach spans were erected on shore, at both ends of the bridge, and launched forward by protrusion over the tops of the towers until they projected some distance beyond the arch abutments. Locomotive derricks were installed on them, and a cableway was set up with its towers on top of the skewback towers.

With these tools, the arch members, delivered on a low-level service bridge 100 ft. above the water, were erected, the two end panels on each side being assembled on falsework, while the remainder was built out simultaneously from each abutment as cantilevers, guyed to the tops of the towers by 28 steel cables. The other roadway spans were erected simultaneously as cantilevers. The erection lasted about 4 years.

The four 200-ft. spans of the electric car bridge across the Schuylkill River, in Fairmount Park, Philadelphia, each have three spandrel-braced riveted trusses, and were erected with the lower chords supported on pile falsework. Materials were delivered on a track laid on the bridge floor, and were handled by a wooden gantry traveler, 72 ft. high, with a 23-ft. overhang.

The Rochester Driving Park highway bridge across the Genesee River, has two three-hinged, spandrel-braced arch trusses of 428 ft. span which were erected on unusually heavy framed falsework more than 212 ft. high, with a wide trussed opening over the river. The truss members, having a maximum weight of 10 tons, were assembled by a 16 by 30-ft. wooden tower traveler, 28 ft. high, with a derrick boom on each corner. The falsework was notable for its great strength and rigidity, equal to many permanent wooden trestle viaducts for railroad service.

The longest arch span in Europe is that of the Bonn Bridge, 614 ft., which was erected by 8-ton electric gantries on 7-story falsework, 112 ft. high, with two trussed openings of 102 ft. for navigation. Upper falsework, long since obsolete in America, was built above the curved bottom chords to provide a horizontal track at the top chord level for the two 8-ton electric gantries by which materials were hoisted from boats and erected. The 594½-ft. arch span of the Dusseldorf Bridge was erected in a similar manner, its falsework being provided with a 164-ft. trussed opening for navigation.

The Lake Street highway bridge, across the Mississippi River at Minneapolis, has two 458-ft. spandrel-braced arch spans. These were erected on falsework 120 ft. high, the piles having been driven through the ice. The falsework terminated at the curved lower chord, and the superstructure was erected by an overhead timber tower traveler with hoisting tackles suspended from an overhang, which traveled on the finished deck of the bridge. This method necessitated the unusual

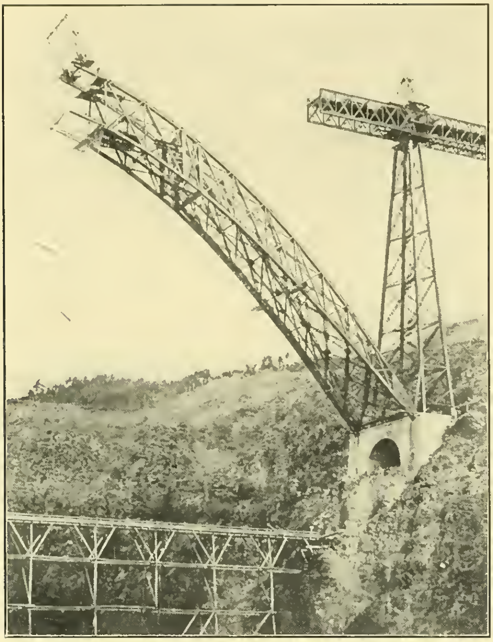


FIG. 1.—ERECTION OF THE GARABIT VIADUCT.

FIG. 2.—ERECTION OF THE BONN BRIDGE.

FIG. 3.—ERECTION OF THE DÜSSELDORF BRIDGE.

Mr. Skinner.

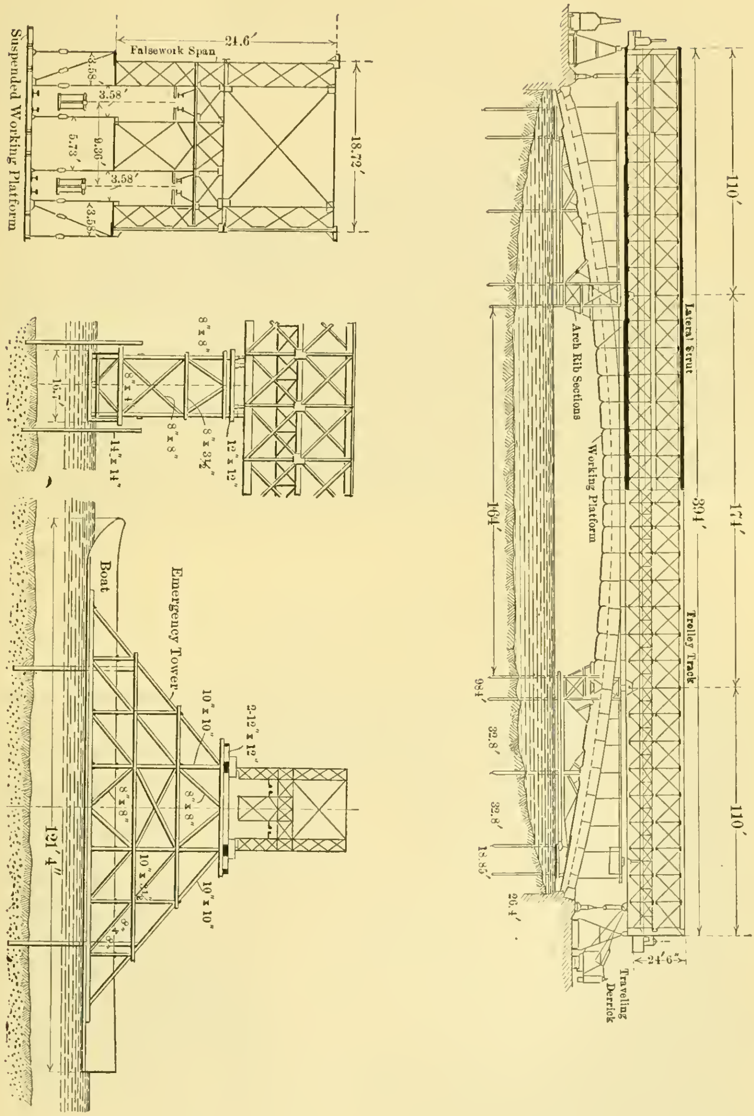


FIG. 4.

Mr. Skinner. procedure of erecting the arch trusses from one abutment across the entire span to the other abutment and making the final connection at or near the skewback pin. This was accomplished successfully, and without difficulty in adjusting the last panel members. During erection, the unbalanced longitudinal thrust, due to unsymmetrical loading on the falsework, was provided for by very heavy inclined timbers bracing the falsework bents diagonally from top to bottom.

The 377-ft. span of the Kornhaus Bridge, in Switzerland, was erected on high and very expensive falsework supporting the arched lower chords on a solid convex plank floor platform, like the lagging for a masonry arch, above which upper falsework was built for the erection of the horizontal roadway trusses supported on spandrel towers, and for light gantry traveler and material tracks outside the arch trusses.

The Panther Hollow Bridge, in Schenley Park, Pittsburg, has one span with four 360-ft., three-hinged spandrel-braced arch trusses. One peculiarity of this erection was that the trusses were erected from one abutment to the crown before the falsework for the other half of the span was built. Field connections were made with small pins at panel points, and, after the arch was swung and these connections had adjusted themselves to the dead-load stresses, the joints were all field-riveted, on the assumption that the rivets would carry the live-load stresses of the bridge in service.

One of the most elaborate of arch span erections was that of the Alexander III Bridge across the Seine, Paris, which has fifteen cast-steel segmental ribs of very flat curvature. During erection these were suspended from a movable overhead falsework span, mounted on towers traveling transversely to the bridge axis. The falsework span was assembled on shore and erected by protrusion across the river, with an auxiliary emergency scow under the forward end. The span was traversed by a pair of trolleys which took the arch segments from shore and sustained them until they were assembled to the preceding ones and were supported by temporary suspension from the trusses. After a pair of ribs was thus simultaneously erected, the pair was swung by slacking off the suspension, and the traveler moved two panels forward and erected the next pair, and so on.

These examples illustrate the principal types of long-span arch erection, and describe most of the principal structures thus far built, giving a general idea of their structural characteristics and of the time and cost of erection. Only two or three of them have spans exceeding that of the Bellows Falls Bridge, and certainly none of them was erected with anything like its economy, or with as small a force as 36 men, or in so short a time as 28 days. These figures and the total cost, contrasted with those of the other bridges, pay a higher tribute to the skill and courage of the designer and erector than any mere compliment or admiring criticism.

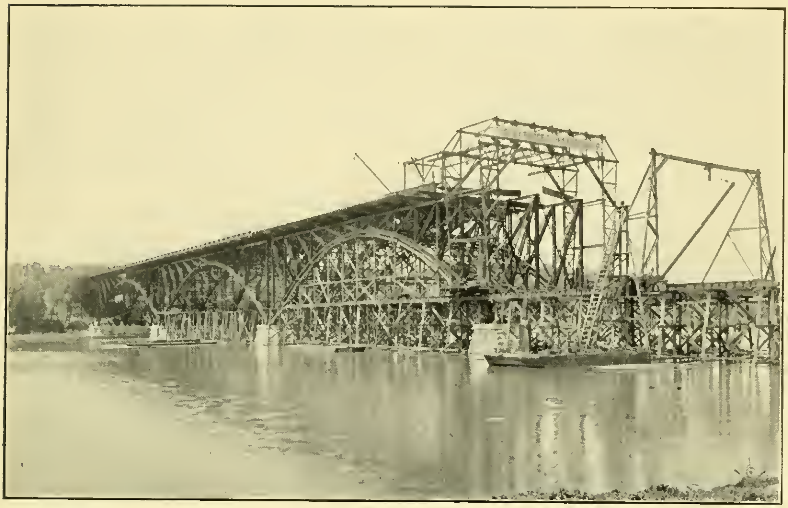


FIG. 1.—THE ERECTION OF THE FAIRMOUNT PARK BRIDGE, OVER THE SCHUYKILL RIVER, IN PHILADELPHIA.

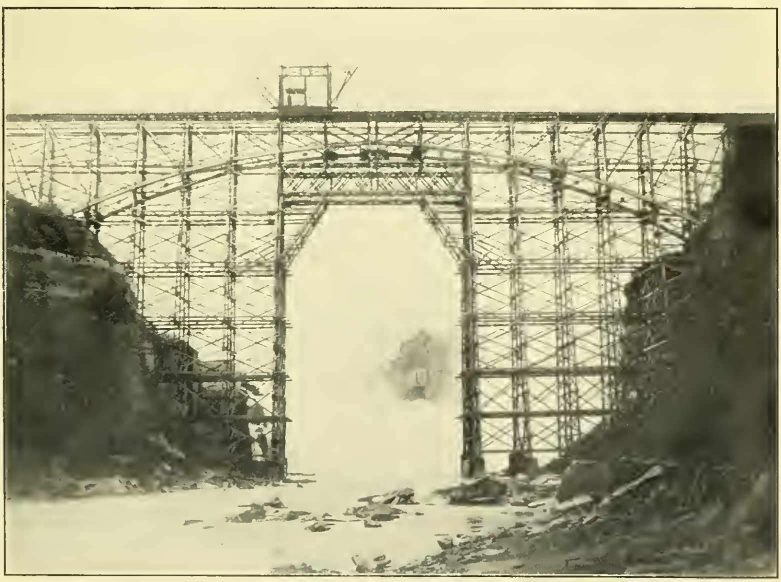


FIG. 2.—THE ERECTION OF THE ROCHESTER DRIVING PARK BRIDGE OVER THE GENESEE RIVER.

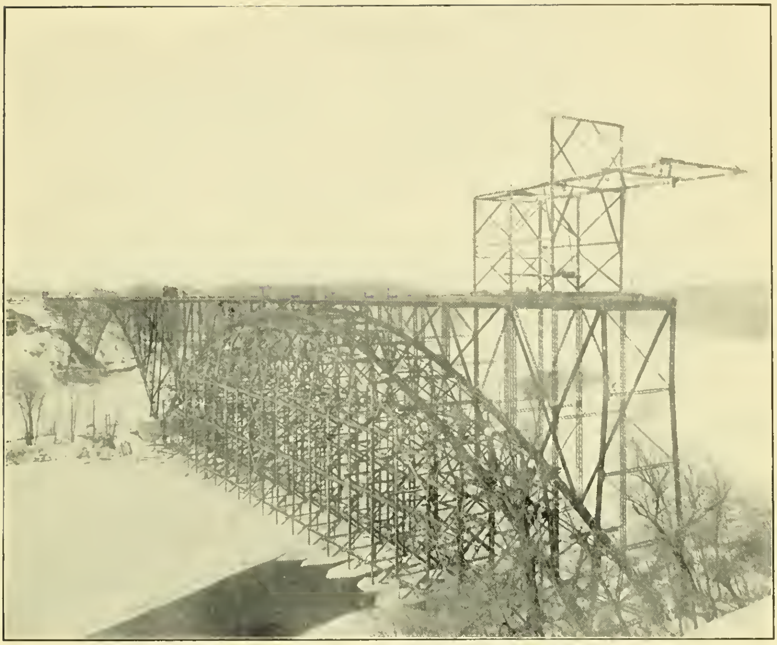


FIG. 1.—THE ERECTION OF THE LAKE STREET BRIDGE, OVER THE MISSISSIPPI RIVER, AT MINNEAPOLIS.

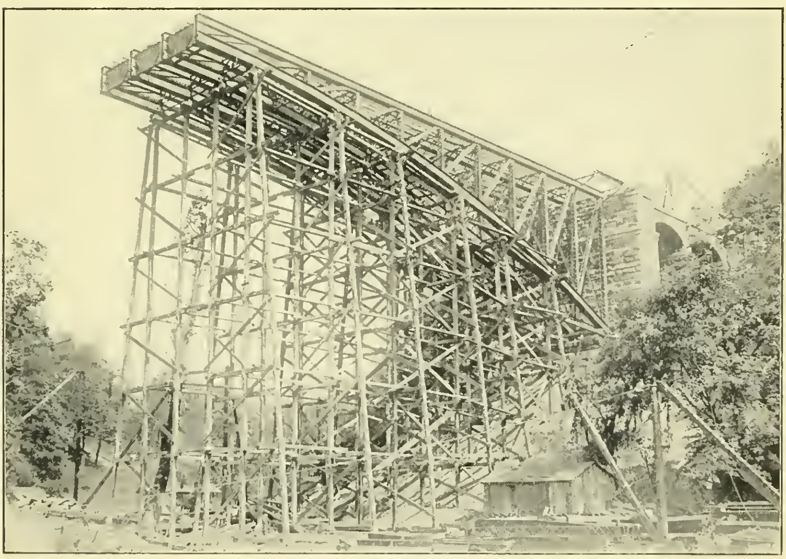


FIG. 2.—THE ERECTION OF A HALF SPAN OF THE PANTHER HOLLOW BRIDGE.

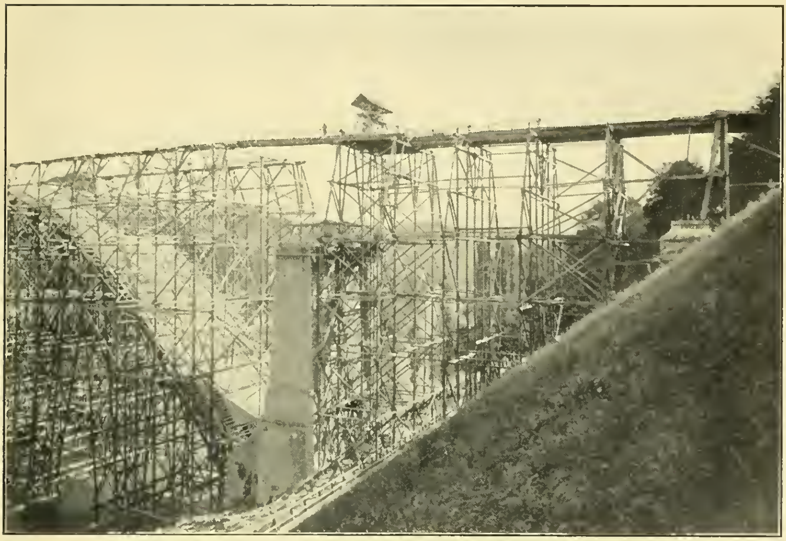


FIG. 1.—ERECTION OF THE KORNHAUS BRIDGE.

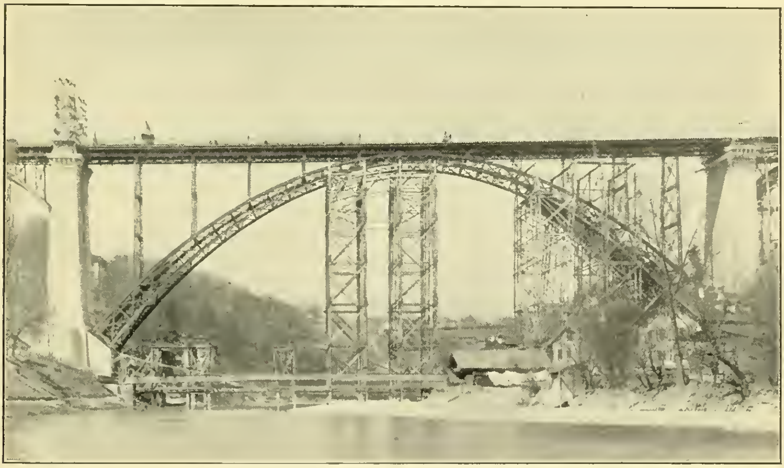


FIG. 2.—ERECTION OF THE KORNHAUS BRIDGE.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

SAFE STRESSES IN STEEL COLUMNS.

Discussion.*

By William Cain, M. Am. Soc. C. E.

William Cain, M. Am. Soc. C. E. (by letter).—In considering the merits of any new column formula, it is well to "take stock" of what we have that is sound and useful, and particularly to compare, side by side, correct theory with experimental data. Mr. Cain.

The "ideal column" is a prismatic, homogeneous column, without initial stress, having the resultant load applied at one end, in the direction of the straight axis, passing through the centers of gravity of the cross-sections. Although there are no ideal columns in practice, the theory pertaining to them is absolutely essential in order to understand fully the behavior of actual columns, or those which are not straight, not homogeneous as to material, strength, modulus, limit of elasticity, and perhaps with initial stress, besides eccentric application of the load.

Two diagrams, Figs. 23 and 24, are submitted; these are reproduced from the discussion by A. Marston, M. Am. Soc. C. E., on the writer's paper,† "Theory of the Ideal Column." In these diagrams, showing the results of Tetmajer's tests on columns, both of wrought iron and steel, the ends being pivoted, three curves are drawn. The upper curve, partly dotted, is drawn from Euler's formula, the full line is from Mr. Marston's formula, given on the figure, and the remaining dotted curve is from the parabolic formula of the late J. B. Johnson, M. Am. Soc. C. E.

* Continued from March, 1908, *Proceedings*.

† *Transactions*, Am. Soc. C. E., Vol. XXXIX, pp. 109 and 111.

Mr. Cain.

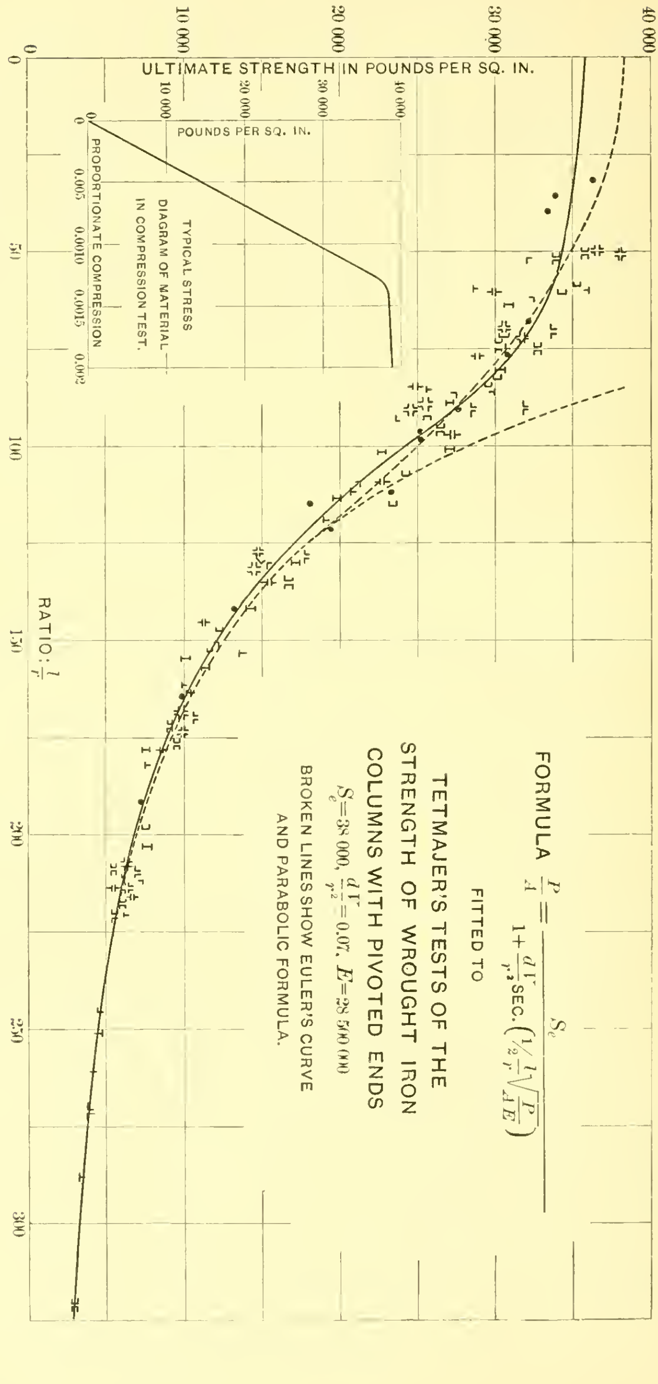

Fig. 23.

Mr. Cain.

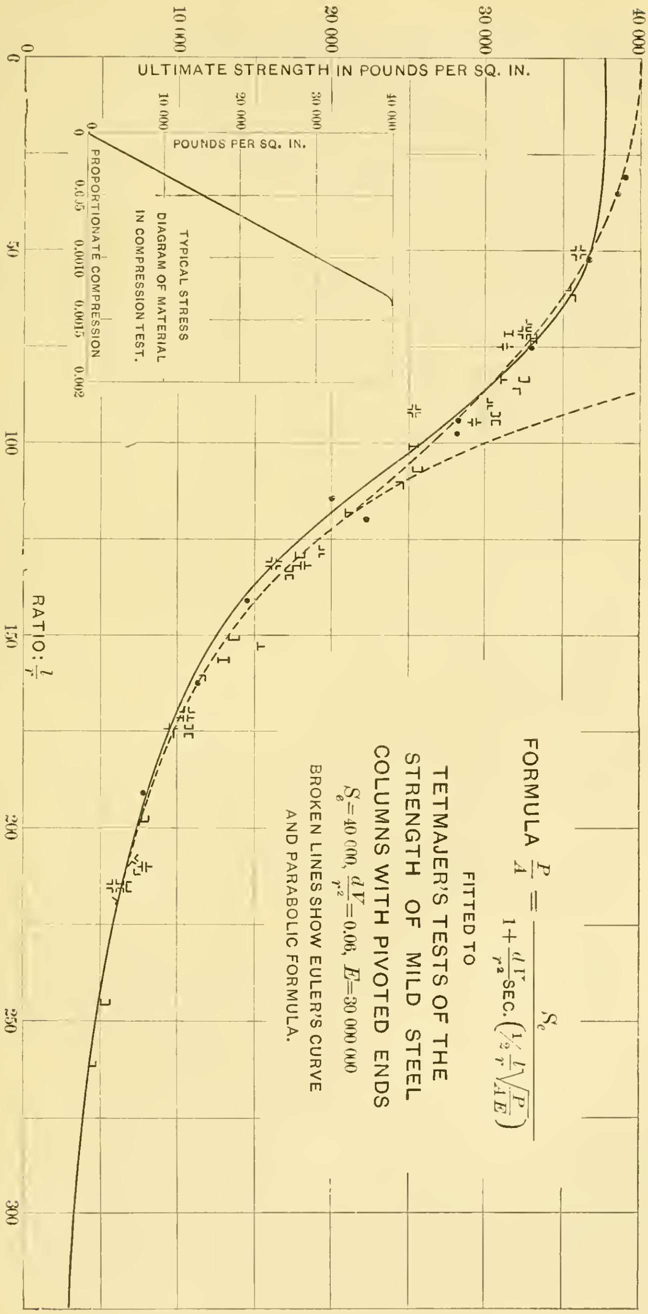

FIG. 24.

Mr. Cain. In Euler's formula it is supposed that the column is "ideal" and that the load is applied without eccentricity.

First, with regard to Euler's formula: it has been shown by the writer that when $\frac{P}{A}$, or the average unit stress as given by Euler's formula, is greater than S_c, the elastic limit, the formula is inapplicable, even theoretically. When $\frac{P}{A} = S_c$, or is $< S_c$, corresponding to greater values of $\frac{l}{r}$ than when $\frac{P}{A} = S_c$, the formula gives the load at which bending just begins. Further, for a very small proportionate increase in the load, the column will fail from the stresses due to the considerable bending and the uniform compression. As a numerical illustration, a column, pivoted at the ends, 325 in. long, was assumed as built up of two 5-in. channels. The inch being the unit, $A = 3.9$, $I = 14.8$, and $E = 29\,000\,000$ lb. per sq. in. Euler's formula gives the load that causes incipient bending as 40 105 lb. It was computed, by an exact formula, that an increase of the load of only 5 lb., caused a deflection of 3.44 in. at the center, with a resultant stress on the most compressed fiber, greater than the elastic limit. Finally, an increase of load of 2 or 3 lb. more would entail rupture, or a breaking in two of the column.

It is sometimes said that a load, as given by Euler's formula, only causes bending, and that, in the derivation of the formula, the uniform compression is neglected. In the writer's analysis, the uniform compression was considered from the start; also, the example shows that Euler's formula is practically a formula for rupture, since a few pounds added to the load (40 105 lb.) computed from it, leads to rupture.

It is admitted that Euler's formula has a limited application, but it is by no means useless to the practical man who has to erect poles in constructions, build derricks, etc.

A very important theorem follows from the foregoing: that for ideal columns, too short for Euler's formula to apply, no bending will occur, and the stress will be the same, and uniformly distributed, on every cross-section, the elastic limit not being exceeded. A similar statement may be made in reference to long columns, to which Euler's formula is applicable, when the load is anything less than the formula gives. In these cases, the ideal column remains straight; there is no bending stress and no shear.

Next, consider the load, P, on the ideal column, to be placed at a distance, d, from the axis of the column. The resulting formula is given on Figs. 23 and 24. This formula is theoretically exact. In it, A = the area of the cross-section, r = the radius of gyration of the cross-section, about an axis through its center of gravity, perpendicular to the plane of bending, and $V =$ the distance from this axis to Mr. Cain. the most compressed fiber.

To apply this formula to the actual column, Mr. Marston assumed, for wrought iron, $\frac{d\ V}{r^2} = 0.07$, and for steel, 0.06.

Now, it is true, as pointed out in the writer's paper (quoted previously), that when $\frac{d\ V}{r}$ is taken as constant, the ratio of d, the eccentricity, to the width of column, varies considerably for different shapes; but when it is considered that the eccentricity assumed for the actual column has to allow in a rough way for crookedness, lack of homogeneity of every kind, bad workmanship, initial stress, as well as the actual eccentricity experienced, the objection loses much of its weight. In practice, too, the line of force may be inclined to, or actually cross, the axis; so that the abnormalities to which the actual column is subjected are so manifold and various that it seems hopeless to deal with them all under the one head of eccentricity. The proof, however, is in the results. Mr. Marston's curves take a middle course through the whole set of plotted values for the wrought-iron columns, and nearly so for the steel columns. Here is seen a practical method of dealing with a truly rational formula, to give practical results for actual columns. No such definite conclusions as have been noted thus far can be reached by a simple observation of thousands of tests.

It may be remarked, that the full curve in Figs. 23 and 24 practically coalesces with that corresponding to Euler's formula for $\frac{l}{r} = 225$, about, as should be the case.

Now, although the theoretical curves fit the experiments so well, the parabolic curves are just as good for practical results, as far as they extend; then Euler's formula or curve can be used for greater values of $\frac{l}{r}$.

A brief reference will now be made to another rational formula, first given by the writer, in July, 1887,* and derived again on p. 120 of "Theory of the Ideal Column:"

$$\frac{P}{A} = \frac{S}{\left(1 + \frac{d\ V}{r^2}\right) + \frac{1}{8\ E}\left(S - \frac{P}{A}\right)\frac{l^2}{r^2}}.$$

In this formula, d, V, and r have the meaning previously given; S is the total fiber unit stress on the concave side of the column at mid-length. The column, of length l, is pivoted at the ends.

The only approximation used in deriving this formula was in assuming the neutral axis to be parabolic. The late Professor J. B.

* *Journal*, Franklin Institute.

Mr. Cain. Johnson derived a similar formula in "Modern Framed Structures." When $d = 0$, an exact theoretical form is reached by replacing 8 by π^2.

Exactly as in the preceding case, to apply this formula to the actual column, $\frac{d\,V}{r^2}$ will have to be taken as constant. Unfortunately, to find the value of $\frac{P}{A}$ for a given column, a quadratic must be solved; hence the formula was not easily adaptable to computation and, consequently, was laid aside.

As Rankine's formula, which is of the form,

$$\frac{P}{A} = \frac{S}{1 + c\,\frac{l^2}{r^2}},$$

is sometimes spoken of as a rational formula for the actual column, it may be well to consider it briefly.*

As this formula does not suppose the load to be applied eccentrically, but does suppose bending, the latter must come from crookedness or lack of homogeneity; for, as has been seen, for values of $\frac{l}{r}$ less than pertain to Euler's formula, there can be no bending for the ideal column, and it is for just such lengths that Rankine's formula has been mainly used. Comparing it with the preceding formula when $d = 0$, it is seen that c cannot be a constant, since it is proportional to $\left(S - \frac{P}{A}\right)$, or the maximum unit stress at mid-length, on the concave side, due to flexure only. In fact, in the derivation of Rankine's formula, the assumption is made, that the deflection varies as $\frac{l^2}{r}\,S$, whereas the very theory of beams to which reference is made, shows that S must here be replaced by the maximum unit stress due to flexure only, or by $\left(S - \frac{P}{A}\right)$. This leads again to the preceding form of formula.

The Rankine formula is thus irrational, and it is surprising that it should still be used in such problems as, for a given column and an assumed load, to compute S, assumed to be the total maximum fiber stress. Mr. Marston's formula, replacing S_e by S, should be used in such problems. As Rankine's formula can be made to fit the tests very well, it has been used extensively; but it must be relegated to the class of empirical formulas, like the parabolic, that fit the tests equally well and are more convenient to use.

The writer was very much impressed with the straight-line for-

* This part of the subject was discussed so thoroughly by Henry S. Prichard, M. Am. Soc. C. E., in *Engineering News* for May 6th, 1897, that an apology seems to be due for discussing it again.

mulas of Thomas H. Johnson, M. Am. Soc. C. E., when they were Mr. Cain. first published, but is leaning now to the use of the parabolic formulas of the late Professor J. B. Johnson, the curves corresponding being made tangent to Euler's curve. In this way results are obtained which can be used in testing the strength of existing structures, as well as in designing. If, in designing, it is desired to exclude columns having the ratio, $\frac{l}{r}$, greater than 100 or 120, say, a simple clause to that effect in the specification should suffice.

It will be seen from this, that the writer thinks that the author's intention can be carried out in a different way than in using the circular curve. If all the innumerable curves that have been proposed could be swept away and a curve be drawn by hand, steering a middle course between the plotted points (noting carefully, also, the lowest points), it would answer the purposes of the designer as well as a formula. For competitive designs, however, a formula is almost imperative. It should not give as large values, for very short columns—especially with riveted or butt ends—as the straight-line formulas. The imperfect "fixing" of the ends of some columns leads to such indefiniteness that it is customary, perhaps, to use the formula for hinged ends for all cases, though some allowance is often made for riveted ends or butt joints.

There is some indefiniteness, too, in the case of pin-end columns, on account of the friction of the pin; so that, for such columns, an eccentricity different from that used for pivoted ends, would have to be assumed, perhaps, in applying the exact formulas of Mr. Marston. In all cases, the details of a built-up column must be designed carefully, for the details, rather than the length, are frequently the main factor in determining the strength of a column.

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PAPERS AND DISCUSSIONS

THE ELECTRIFICATION OF THE SUBURBAN ZONE OF THE NEW YORK CENTRAL AND HUDSON RIVER RAILROAD IN THE VICINITY OF NEW YORK CITY.

Discussion.*

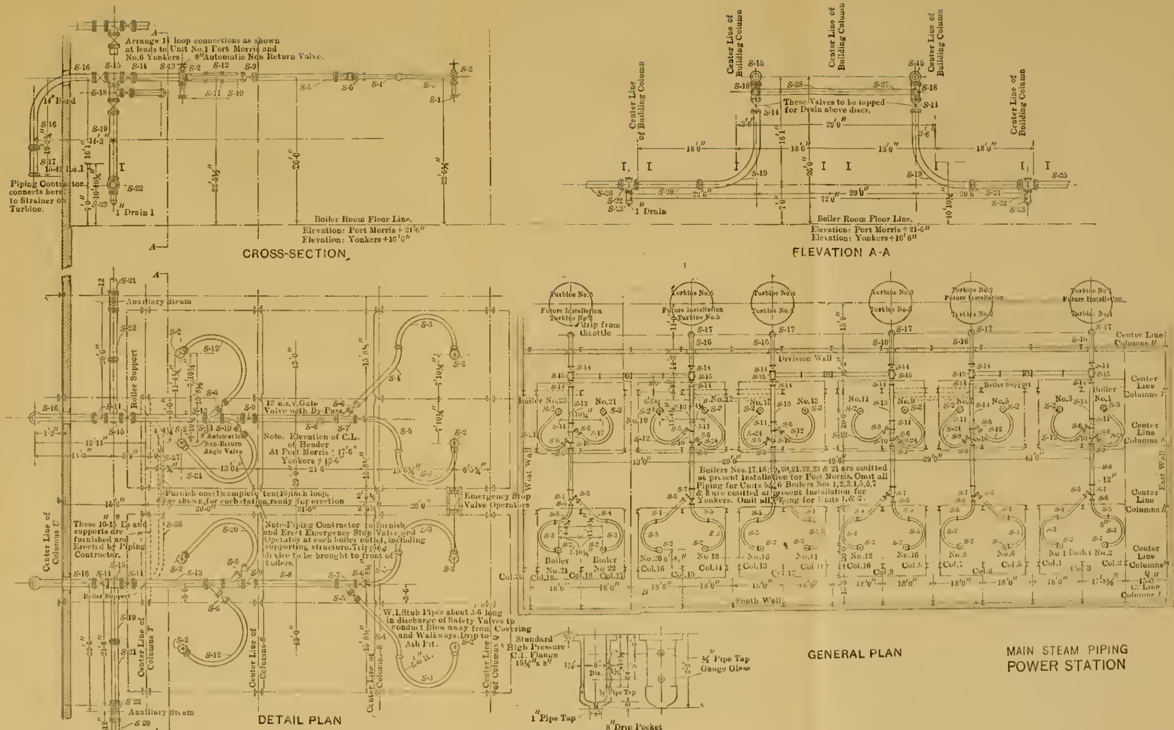
By Messrs. Edwin B. Katte, W. S. Murray, George A. Harwood, W. B. Potter, Frank J. Sprague, Henry G. Stott, and W. J. Wilgus.

Mr Katte. Edwin B. Katte,† Esq.—Mr. Wilgus has covered the subject so completely that it does not seem possible to add much of general interest; however, some of the details of the work already described will perhaps be of value to those interested in a further consideration of this electrical installation.

Storage Batteries.—Mr. Wilgus has explained that storage batteries were installed as an insurance against interruption to the train service; this value was strikingly illustrated a few months ago, when, during the most severe wind storm in this locality for many years, several telegraph poles, which were on a high bank above the aerial transmission lines, were blown down, and one pole, with its numerous telegraph wires, hung suspended on the 11 000-volt aerial transmission lines. The effect was to open instantly the circuit breakers in the power-station, and the safety devices in the sub-stations automatically

* Continued from April, 1908, *Proceedings*.

† Electrical Engineer, New York Central and Hudson River Railroad.



MAIN STEAM PIPING
POWER STATION

disconnected from the load every rotary converter on the system. The batteries were "floating" on the bus-bars, and immediately took up the train load, and there was no interruption to service. The load despatcher, knowing that the batteries would carry the load for a sufficient time, was able, in an orderly manner, to locate the cause of the trouble and then direct the various operators in charge of the several sub-stations how to start up and throw in their rotaries and pick up the load. Mr. Katte.

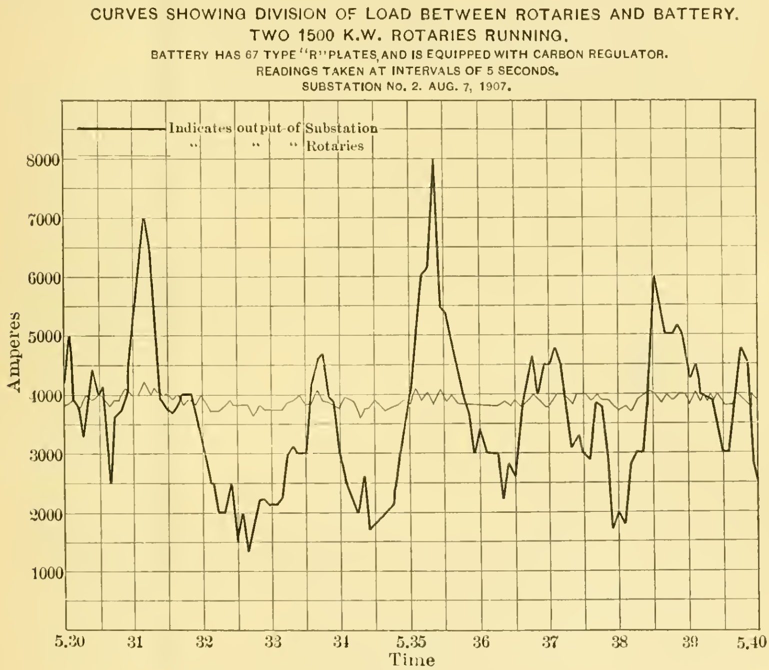


FIG. 8.

Another reason mentioned by Mr. Wilgus for installing storage batteries was to relieve the rotary converters and generators from the sudden fluctuation of load due to the starting, stopping and passing of heavy trains. That this has been effectually accomplished will be apparent from the diagram, Fig. 8, taken from actual readings at one of the sub-stations, the heavy line representing the output of the sub-station and the fine line indicating the load on the rotaries, the heavy fluctuations having been taken up by the battery. The readings were taken at 5-sec. intervals for a period of 10 min.

Mr. Katte. *Aerial Transmission Lines.*—Mr. Wilgus has referred to the reliability of well-built aerial lines. Aside from the one instance just referred to, when telegraph poles fell on the line, there has never been an interruption in the high-tension aerial circuits; and it is of interest to note that the interruption mentioned only interfered with one circuit, and the amount of damage to that circuit was the mechanical breaking, by the swaying by the telegraph pole, of three out of seven strands of one conductor. It is the practice of this company to place the aerial transmission lines on the opposite side of the right of way from the telegraph poles, but, at the location above cited, it was necessary, for a space of three pole lengths, to place the telegraph and transmission line poles on the same side.

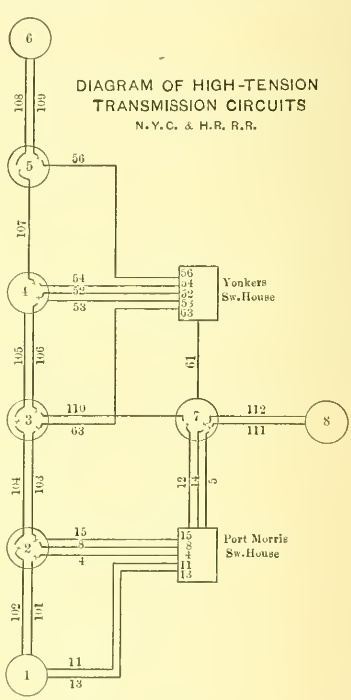
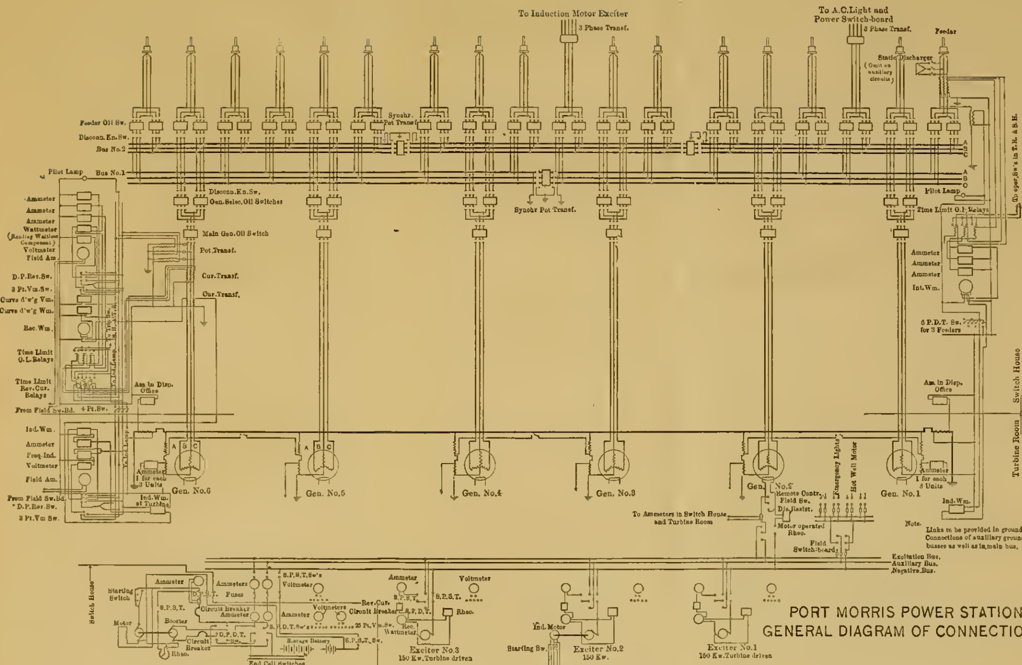


Fig. 9.

High-Tension Distribution System.—The 11 000-volt circuits have been laid out to afford the maximum flexibility with the minimum quantity of copper from the two main generating stations to the eight sub-stations. From the diagram, Fig. 9, it will be noted that each sub-station, with the exception of the outlying stations, Nos. 6 and 8, has a direct circuit from the adjoining power-station, and in the case of Sub-stations Nos. 6 and 8, the supply is from direct feeders through Sub-stations Nos. 5 and 7. Each sub-station is fed by two or more independent circuits, and in such a manner that either power-station can feed any sub-station. The more important sub-stations, namely, Nos. 1, 2, and 7, which supply current to the congested Harlem Division, are each fed by at least two circuits direct from the power-station. The duplicate circuits are entirely independent, being located on the two sides of the right of way, so that an accident to one circuit could not possibly affect the other.

Power-Stations.—In addition to what Mr. Wilgus has said regarding the power-stations, a feature has been made of the sectionalizing of each unit. Each turbo-generator, with its condenser, auxiliary apparatus, boilers, feed pumps, etc., is a complete unit, and can be isolated from other units in the station in case of trouble. This is true of the piping arrangement as well as of the electrical connections. These

PLATE LVII.
PAPERS, AM. SOC. C. E.
MAY, 1908.
KATTE ON
ELECTRIFICATION OF SUBURBAN ZONE
OF N. Y. C. & H. R. R. R.



PORT MORRIS POWER STATION
GENERAL DIAGRAM OF CONNECTIONS.

Mr. Katte.

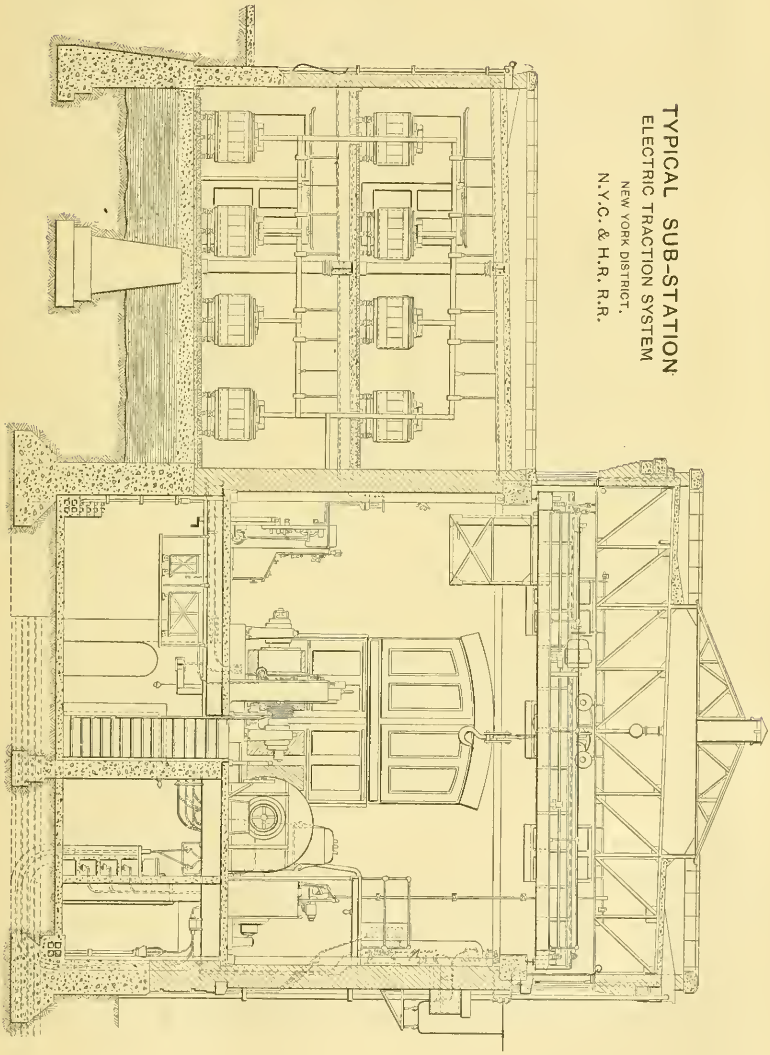


Fig. 10.

Mr. Katte. features are illustrated in the general piping plans, Plate LVI, and the wiring diagram, Plate LVII.

Switch-Houses.—The separate switch-houses are a distinctive feature of the Port Morris and Yonkers power-stations, and contain several novel features, although the idea of separate houses is not new, having been used at the Fisk Street Station, in Chicago. The two switch-houses, in generic principles, are the same, although the layout is different in each. Plates LVIII and LIX illustrate the salient features of the Port Morris switch-house. From the plans it will be noted that the low-tension or control wiring is kept separate from the high-tension circuits, thus eliminating all danger in making repairs to these circuits. By the use of barriers, every precaution has been taken to protect attendants from short-circuit flashes in the high-tension compartments. The elevation shows clearly the use to which the building has been put, the sub-basement being devoted entirely to conduits and manholes. The basement, which is on the same level as the main floor of the generating station, contains the bus-bar compartments, and the floor above is given up to the oil-switches, switch-boards and bench-boards. The second floor, which is on the level of the switch-board gallery, and is connected thereto by a bridge, contains the load despatcher's office, the exciter storage battery, the heating and ventilating system, locker-rooms, and storerooms.

Sub-Stations.—In the present electrification scheme there are eight sub-stations, located at the most economical load centers. Five of these stations are now complete, and four are in daily service. The general principles governing the design of all the sub-stations are the same, although the detailed arrangements vary somewhat in each particular case. The general features are shown in the typical drawing, Fig. 10. As in the design of the power-stations, the sectionalized-unit system has been carried out, as far as possible, and each rotary converter, with its transformers, switches, etc., is as independent as conditions will permit. The typical sub-station wiring diagram is shown on Plate LX.

Third-Rail.—The under-running third-rail has now successfully passed through the snow and sleet storms of three winters, the first year at the experimental track, west of Schenectady, and two years in actual service in New York where about 90 miles are in daily use. There has not been a single instance of a moment's delay due to sleet or ice on the third-rail, or snow interfering with collecting the current or the passing of trains. The adequacy of the third-rail protection is illustrated by the fact that not a single patron of the road has been injured on account of contact with the third-rail, and but few employees, while, in every instance of injury, it has been due to gross carelessness or negligence on the part of the employee injured.

Low-Tension Feeder System.—The direct current is in all cases fed to the third-rail through circuit breakers which are controlled from

PLATE LVIII.
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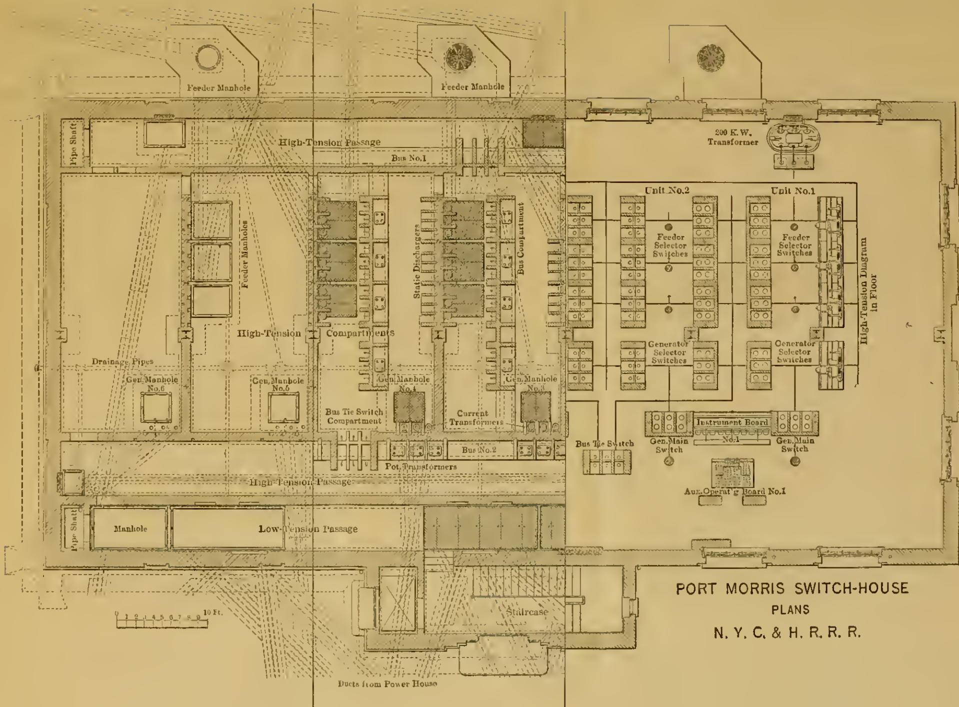

PORT MORRIS SWITCH-HOUSE
PLANS
N. Y. C. & H. R. R. R.

PLAN OVER BASEMENT

PLAN OVER FIRST FLOOR

the nearest sub-station, and the entire third-rail system is paralleled by Mr. Katte. auxiliary direct-current cables; this permits cutting out all third-rails in a given section, and feeding around through the auxiliary cables to adjoining sections for the operation of trains on either side of the dead section. The diagram of positive feeders, Plate LXI, shows the various connections to the third-rails and the auxiliary cables. Opposite each sub-station, except the end ones, there is an isolated section of third-rail sufficiently long to take the longest multiple-unit trains. The purpose of this isolated section is to prevent a train bridging from a live rail to a dead third-rail section, on which section there may have been an accident, or on which men may be working, and further, to prevent bridging, two sections of third-rail in which, because of their being fed by different sub-stations, there might be a difference of potential and thus cause the blowing of the motor fuses in the train.

Precautionary Devices.—Among the devices to ensure safety to the system may be mentioned the indicating wire which has been woven into the protecting braiding of the direct-current cables along the Park Avenue Viaduct and through the Park Avenue Tunnel. The function of this wire is to trip the circuit breakers and notify the sub-station attendants by the ringing of a gong should a short circuit occur of sufficient severity to cause burning or injury to the cable, but not involving a quantity of current which would open the circuit breakers on an over-load. The need of this device was demonstrated last year on the Park Avenue Viaduct when a defective joint in a direct-current cable failed, and, because of the disobedience of an attendant to follow an order, all the switches to clear the short circuit were not promptly opened. The burning continued for some time because the quantity of current flowing was not of sufficient volume to open the over-load circuit breakers, which, necessarily, have to be set to carry large quantities of current for the regular handling of heavy trains.

A safety device installed in the Park Avenue Tunnel consists of a cord, running parallel to all tracks, and connecting with signal alarm boxes. If for any reason it becomes necessary to cut off current quickly from any third-rails, a pull on the safety cord will open the circuit breakers feeding that section of third-rail, and at the same time will notify the attendant at the sub-station that the third-rail has been cut out.

W. S. Murray, Esq.*—The thanks of the railway engineering Mr. Murray. fraternity are due to Mr. Wilgus for this paper.

The author's theme is broad, and in the accomplishment of the results severally cited, the speaker joins in the general assent regarding the marked successes which rightfully have been obtained in this stupendous work.

Only about half a page of this paper is devoted to conclusions with

* Electrical Engineer, New York, New Haven, and Hartford Railroad.

Mr. Murray. which the speaker is not in perfect agreement, and as there is such a small difference of opinion—in space at least—he will waive further comments.

In order not to be misunderstood in this matter, the speaker must state first that, given the New York Central zone of electrification, free for a decision as to the form to be adopted, he believes that a majority of the best informed electrical engineers of America would to-day cast their ballots in favor of single-phase electrification. If this statement causes surprise, it should be at once explained that it is not because the speaker has not been of this opinion ever since the New Haven road made its decision as to the form of electric traction it would adopt, but because he could find no reason to criticise what had already been accomplished. Mr. Wilgus makes the following statement:

"The wisdom of adhering to the type of equipment already chosen has been proven by recent comparative tests of locomotives of the two types under exactly the same conditions, which demonstrate that the one designed only for direct current consumes from 15 to 25% less current than the one intended for use on both systems. This will effect a saving to the company of at least $140 000 per annum."

This statement deserves careful analysis. Before discussing this item of economy, which, as stated later, becomes $300 000 per annum, a previous paragraph on the same page will be mentioned, in which are discussed the three principal reasons for adopting the direct-current system.

(1).—The insufficient practical development of the alternating-current system for a trunk-line problem requiring absolute reliability of service;

(2).—Restricted clearances, which forbade the use of overhead conductors;

(3).—The legal obstacles to the use of overhead trolley wires carrying high voltages within the limits of the City of New York.

(1).—Under "Reasons for Electrification of New York Central," Mr. Wilgus has stated that legislative action required the complete abandonment of the steam locomotive in Park Avenue, south of the Harlem River, within a period of five years, terminating July 1st, 1908. He also states that the change of motive power for schedule trains was completed on July 1st, 1907; thus, three or four years after the date of the decision to electrify, commercial trains were placed on schedule one year before the date required by the State of New York. A year previous to the decision of the New York Central to electrify, a paper was read, before the American Institute of Electrical Engineers, by Mr. Benjamin G. Lamme, Chief Engineer of the Westinghouse Electric and Manufacturing Company, descriptive of the Washington, Baltimore, and Annapolis single-phase railway, in

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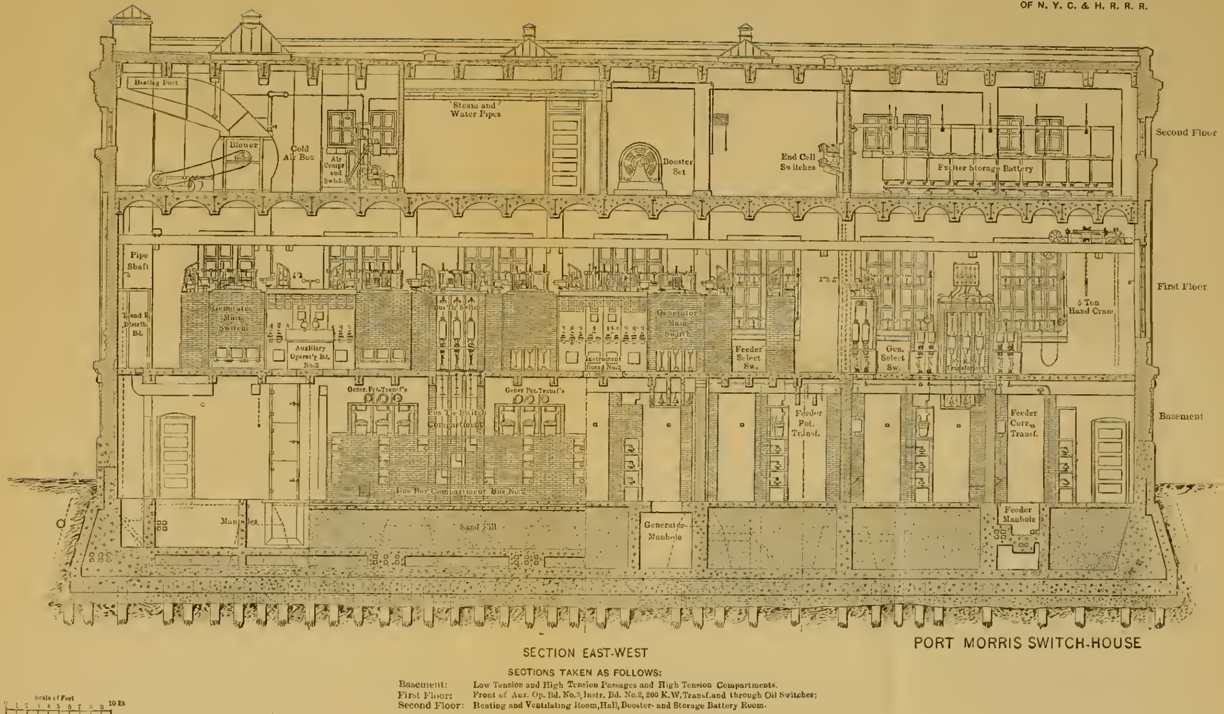

PORT MORRIS SWITCH-HOUSE

SECTION EAST-WEST

SECTIONS TAKEN AS FOLLOWS:

Basement: Low Tension and High Tension Passages and High Tension Compartments.
First Floor: Front of Aux. Op. Bd. No.3, Instr. Bd. No.2, 200 K.W. Transf. and through Oil Switches;
Second Floor: Heating and Ventilating Room, Hall, Booster- and Storage Battery Room.

which diagrammatic power connections were shown fundamentally in principle, as adopted to-day, together with practical data, in the form of traction, speed, power-factor, and efficiency curves. It seems to the speaker that the State of New York would probably have been satisfied if the full time allowed had been taken, and, instead of being one year ahead in operation, thought had been devoted to the single-phase proposition. If, at the expiration of this year, the author did not believe the art had been sufficiently advanced for its adoption, the speaker believes that his first reason would not have existed. Mr. Murray.

(2).—Restricted clearances, which forbade the use of the overhead conductors: There is a zone in which this matter has been handled. Where the author can point to one restricted clearance on his line, the speaker can point to five on the New Haven road, and does not doubt but that the overhead obstacle clearance on the New Haven will be found to be closer to the rails. The answer to this can be anticipated:—the overpowering argument—how would the Park Avenue Tunnel be electrified? How have the Simplon Tunnel, the Sarnia Tunnel and other tunnels been electrified? All these have restricted clearances, and yet there is nothing mystifying or difficult about the installation of overhead conductors under clearances of this character. In short, unless the clearances in parts of the New York Central electrification zone, other than those over which the New York, New Haven and Hartford trains operate, are of a character strangely different, the speaker would undertake to install the overhead type of construction.

(3).—The legal obstacles to the use of overhead trolley wires carrying high voltages within the limits of the City of New York: Even before that organization known as the Gas, Water, and Electric Light Commission was dissolved by the bill introduced and adopted under the present administration, Mr. Wilgus elected to erect his overhead conductors within the limits of the City of New York, and chose for their location, not points over the part of the railroad company's right of way, where their traffic is most dense, but at its edges, and there his transmission lines are carrying 11 000 volts. Why, therefore, if these voltages can be assimilated by the City of New York on the edges of the right of way, can they not be tolerated toward the center of that strip of land, which will place them farther from the public? The speaker ventures the assertion that now that the State of New York controls its public service corporations by a Public Utilities Commission, it will confirm Mr. Wilgus' action in placing these high-tension wires on the New York Central Company's right of way, even if he has elected to place them as near as possible to the public. In connection, also, with the question of high voltage, this discussion would not be complete without a reference to such towns as Windsor, Ont., Hanover, Pa., Colfax, Wash., Palouse, Wash., Connersville, Ind., Mr. Murray. Rushville, Ind., Greensburg, Ind., Shelbyville, Ind., Napa, Cal., Vallejo, Cal., Exeter, Cal., and Rochester, N. Y., where single-phase systems are installed to-day. In all but Rochester, N. Y., the voltage in the trolley wires varies from 3 300 to 6 600 volts, not in restricted territory devoted exclusively to the terminals of the respective roads, or private right of way, but in the streets and highways. If high voltage brings with it high efficiency, smaller currents to be collected, smaller fixed charges, and in consequence of its physical attributes, lower operating costs, there can be but one argument against it. It is granted that the argument of safety is one deserving the greatest consideration. But is it not true that the greatest source in the agitation of this question of safety is prejudice? Every electrical engineer will agree that 100 000 volts can be placed on a trolley with perfect safety if beyond peradventure of a doubt two conditions are satisfied: (1).—That it is beyond the reach of the tallest man; and (2).—That the trolley wire will remain in its place.

It can be asserted that the first condition on all roads, as far as the public is concerned, is satisfied. As concerns the second requisite, on the 1 000 miles of high-voltage trolley that have been installed to date, the speaker has yet to learn of a citizen, not employed by the railroad, whose life has been forfeited on account of this form of electrification. These statistics will dictate the use of the high-voltage system, as they controvert the only argument that stands in its way to-day. This is the day of civic reform. Political control of appointments of public officers who are to regulate public service corporations will soon be a thing of the past. The States will see the advantage of the appointment of high-salaried engineers, whose integrity will be a guaranty that the practices of the past will be abandoned, and in consequence the railroads of this country will be urged to present arguments based upon facts rather than prejudices; and, while the City of New York and other cities may not yet adopt high-voltage trolleys, such men, when convinced that railroad companies can secure greater economies in their several operating departments by the use of the high-voltage trolley, will allow its use within the restricted zones included in terminal and right-of-way property.

Returning to the matter of the saving of $300 000 per annum, on account of the adoption of an electric engine designed for exclusive use on direct current instead of one which is operative on either direct or alternating current, and before referring to data of actual record concerning the economies of these two classes of engines, the speaker wishes to ask Mr. Wilgus why he compares the direct-current locomotive with the locomotive which is operative with either class of current? Why not compare the direct-current locomotive with the alternating-current locomotive? The author makes a comparison between two locomotives, one of which, by necessity, performs two func-

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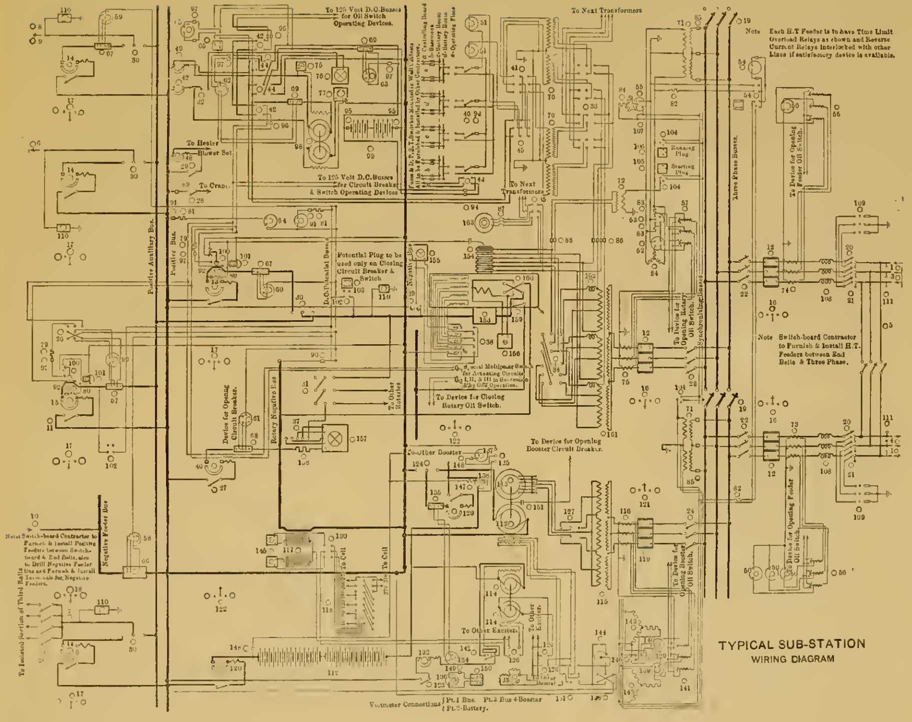

TYPICAL SUB-STATION
WIRING DIAGRAM

tions to its competitor's one. In spite of this, however, the inherent law which requires that a machine, in order to be a good alternating-current motor must necessarily be a good direct-current motor, has been demonstrated in the records of the electric meters installed on the New Haven locomotives for measuring the power while handling the trains in the New York Central zone. Mr. Murray.

While tests could be conducted to show that either locomotive would carry a given trailing load from the Grand Central Station to Woodlawn at the expenditure of a less number of kilowatt-hours than the other, such tests, unless conducted over a long interval, are valueless, but Table 5 is the record for the month of February, showing the energy consumption upon which the New Haven road is billed by the New York Central Company, and is of value.

TABLE 5.—TOTAL KILOWATT CONSUMPTION FOR ELECTRIC-TRAIN SERVICE IN NEW YORK CENTRAL ZONE.

Date. February.	Direct current, hours.	Direct current, miles.	Total tonnage.	Passengers carried.	Number of commercial trains.
1	7 110	564	11 384	7 961	40
2	4 650	420	7 148	5 196	21
3	6 880	552	11 116	9 190	39
4	7 010	540	11 388	7 708	38
5	8 110	600	11 666	8 401	44
6	7 490	564	12 205	8 639	42
7	7 750	600	11 687	9 611	41
8	8 860	660	14 114	11 270	46
9	4 150	396	7 295	5 131	21
10	5 690	456	9 231	7 515	35
11	5 360	420	8 314	6 578	32
12	6 420	588	12 018	7 415	43
13	7 900	648	13 261	10 536	46
14	8 290	624	13 757	10 973	46
15	9 030	736	15 932	12 055	50
16	4 730	432	7 778	5 628	21
17	8 760	660	9 008	11 799	46
18	8 530	624	13 463	11 112	45
19	8 730	696	14 285	10 432	48
20	7 890	624	13 851	10 957	46
21	8 650	672	14 607	12 098	48
22	8 910	784	15 494	8 094	49
23	4 230	360	6 607	4 575	18
24	8 610	672	14 561	11 833	47
25	7 840	660	14 412	11 448	48
26	7 610	636	14 412	10 860	48
27	9 310	660	14 547	11 732	48
28	8 780	684	14 629	11 844	48
29	8 580	756	16 332	12 887	50
	215 860	17 388	354 399	273 469	1 197

Average watt-hours per ton-mile = 41.9.

The speaker does not wish to enter here upon a theoretical discussion of the rate of energy required to discharge a given schedule, but wishes to point out the fact, well known by all electrical traction en-

Mr. Murray. gineers, that slow-downs or stops not included in the regular schedule between any two points always increase the amount of energy required. Neither the committee of which the speaker was a member, nor the several others that followed in rapid succession, could arrive at a mutually agreeable conclusion as to an equitable rate the New Haven road should pay the New York Central for power, and it was seen, if power was to be purchased at the electric locomotive shoes, that the rates would have to be settled by an impartial committee. A firm of high standing in engineering circles was engaged by the two companies, and the question of delays and stops included between the Grand Central Station and Woodlawn, played so important a part in the consideration of a mutual basis of agreement between the two companies, that the New Haven road was requested to accept six slow-ups and five stops between South Mt. Vernon and the Grand Central Station, and six slow-ups between Grand Central Station and South Mt. Vernon; and the estimated rate of consumption, which that road was requested to accept as a determining factor for the amount of capacity that should be reserved for it in the Port Morris Station was at the rate of 68 watt-hours per ton-mile. As shown in Table 5, the average was 41.9 watt-hours per ton-mile. The estimated amount is thus 62% greater than the actual amount measured, and it has occurred to the speaker that Mr. Wilgus may have also been mistaken about the relative amount of current taken by the two types of locomotives. Mr. Wilgus has given 33.9 watt-hours per ton-mile in his table of comparative tests of steam and electrical locomotives in switching and hauling service. These figures are in a manner a confirmation of his statement, and yet these represent figures in a test covering only two trains for two weeks. The New Haven figures cover an equipment necessary to the haulage of as many as fifty trains per day during a period of one month, and, in addition, are the basis of the New York Central charge for current. It should be added, also, that in actual test the speaker has seen the New Haven locomotives perform the same schedule as the one mentioned by Mr. Wilgus, at an energy rate of consumption not greater than the figures mentioned by him. It can be done.

In Mr. Wilgus's statement of a $300 000 saving per annum (which the speaker has reason to believe is zero instead of this large amount), there is no mention of the attendant fixed charge of the system producing the saving. In order to prorate properly the operating and fixed charges of the electrical distributing system jointly used by the two companies between Woodlawn and Grand Central Station, a distance made up of 12 miles of four-track railroad, it was necessary for the New Haven Company to audit the construction charges of the New York Central Company. This duty devolved upon the speaker. Exclusive of power-house and motive power (and, incidentally, the New

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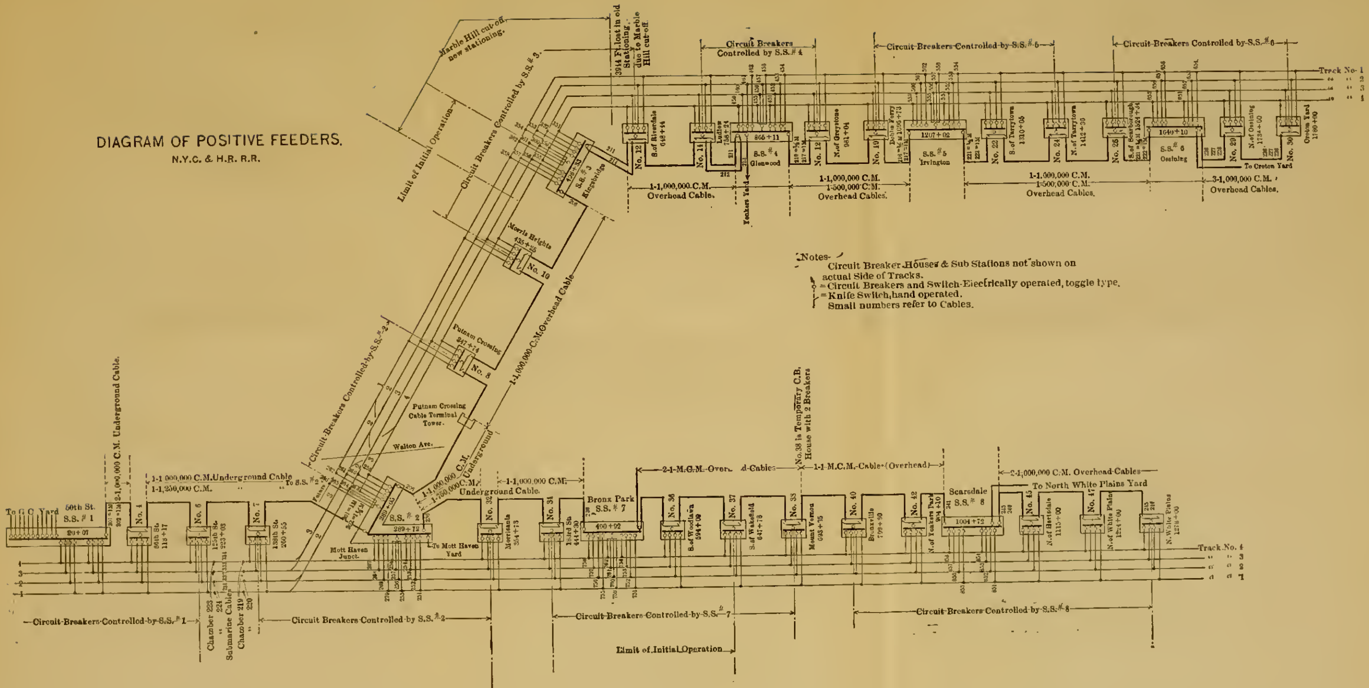
DIAGRAM OF POSITIVE FEEDERS.

N.Y.C. & H.R. R.R.



Haven alternating-current-direct-current locomotive cost less than the New York Central locomotive, notwithstanding the continuous capacities of the two machines are very nearly the same), the cost per mile of electrification is in the ratio of 5 to 1. It is true that much of this cost is made up of land, which was necessary for sub-stations, and thousands of pounds of copper, but it should be remembered, in comparing it with the single-phase system, that in the latter these requirements are practically dispensed with. Sub-stations are reduced to zero, and copper to a minimum, and so the speaker thinks that in mentioning this increase of power required to operate New Haven locomotives, this attendant factor of five times the fixed charge should be, at least, mentioned. Mr. Murray.

As much in importance, after considering the fixed charge of an installation, is the cost of operation. Again, in virtue of the necessity of the close co-operation of the two companies, in so far as the auditing of accounts is concerned, figures appertaining to this interesting subject necessarily passed through the hands of the speaker. It was found that for 12 miles of four-track road there is a charge, for maintenance and operation, of five times the amount the New Haven road pays to maintain and operate 21 miles. Although he has the exact figures covering the disbursements necessary to the installation of each form of distributing system, he has not felt justified in presenting these figures at this time. However, the ratios mentioned serve the same purpose.

In conclusion it should be stated that the principal object of the New Haven engineers has been simplicity. True, the use of the direct current on the rails over which their locomotive had to operate was a sort of kink in the wire, but they were not responsible for this, nor has it carried any especial terror to their hearts, and had it not been superimposed upon them, like the other parts of the system, the control in their locomotives would have been simplicity itself.

Mr. Wilgus has spoken of the violent fluctuations of the load on the power-station and sub-stations, which are corrected by the use of storage batteries. On the twenty-one miles of the New Haven road there are no storage batteries, but, due to the high efficiency of transmission and the prompt regulation of the generators at the power-station for fluctuating loads, even at the western terminus, the most distant from the power-station, the voltage seems to be practically as stiff as at Cos Cob.

In yards, a light but strong cross-catenary form of construction is readily applicable to this branch of electrification. The question of dodging the third-rail no longer confronts the yard hand. There is 8 ft. of good air between the trolley wire and the tops of the freight cars, thus providing clearance for the tallest man.

The speaker's argument has been based upon official data for the

Mr. Murray. purpose of controverting the statements made against the alternating-current system, and he is willing to go on record that the near future will see the high-voltage distribution system, with its attendant alternating-current locomotive, propelling trains from and between the terminals of cities where the density of traffic is sufficient to warrant the increased fixed charges for electric traction.

Mr. Harwood. GEORGE A. HARWOOD, M. AM. SOC. C. E. (by letter).—This paper, for the period which it covers, is so complete that little can be added. Some features of the improvement, however, which have been taken up since the paper was prepared, may be of interest. The most important of these is the removal of the train-shed of the Grand Central Station.

It was originally contemplated that traffic would be transferred to the new east side station before it became necessary to remove the old train-shed. This was prevented by the increase in traffic and the progress of the excavation. Last fall, therefore, it was decided to remove the shed while the through passenger trains of the New York Central and the through and local trains of the New Haven continued to use the old station.

It is proposed to remove all of the shed north of the passenger waiting-room. The length of this portion is about 600 ft., and consists of wrought-iron arches, the material for which was imported from England and erected in 1870. The arches are built in the form of a truss, the section being about 4 ft. from back to back of chord tees. They have a span of 200 ft. 1 in., from center to center of pins, with a clear distance of 85 ft. from the top of the platform to the under side of the arch. The bottoms of the arches are tied together under the tracks with 3-in. rods. The distance from center to center of arches, longitudinally, is 20 ft. $3\frac{1}{2}$ in. The details of construction are shown on Plate LXII. There are about 1 350 tons of wrought iron, 350 tons of cast iron, 90 000 sq. ft. of corrugated-iron roofing, 60 000 sq. ft. of glass, and 530 000 brick to be taken down, loaded into cars and removed from the terminal without interference with the regular business.

To accomplish this, and reduce to a minimum the possibility of accident by falling material, it was decided to erect a traveler, the outlines of which would conform to the general contour of the train-shed, spanning all platforms, with heavy floors extending the entire width of the shed. The supports for the traveler, details of which are indicated on Plate LXIII, rest on the five intermediate platforms, and are carried on heavy cast wheels which roll on standard 100-lb. rails. The load on each platform, including the weight of two of the train-shed trusses, is 200 tons, this being distributed over the entire width of the platform by ties supporting the rail, and covered with temporary planking so as not to interfere with the use of the platform for regular business. The traveler contains 370 000 ft., b. m., of lumber, 65 tons of bolts and washers, and 33 tons of plates and castings. It has a length

PLATE LXII.
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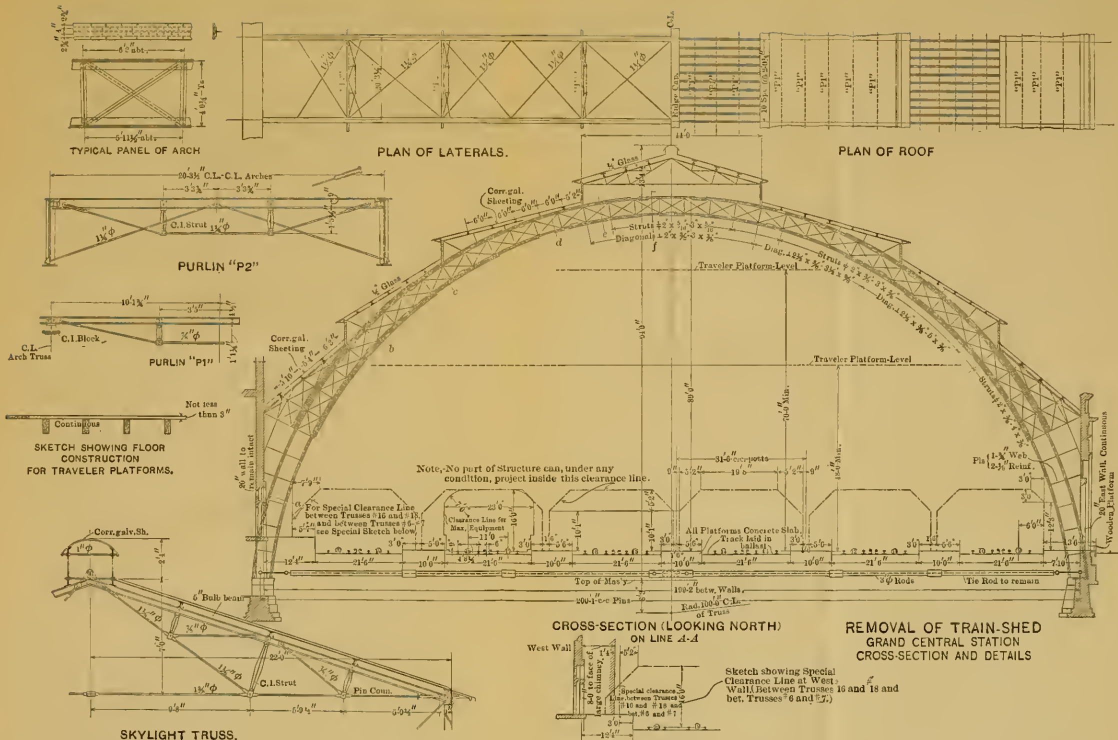
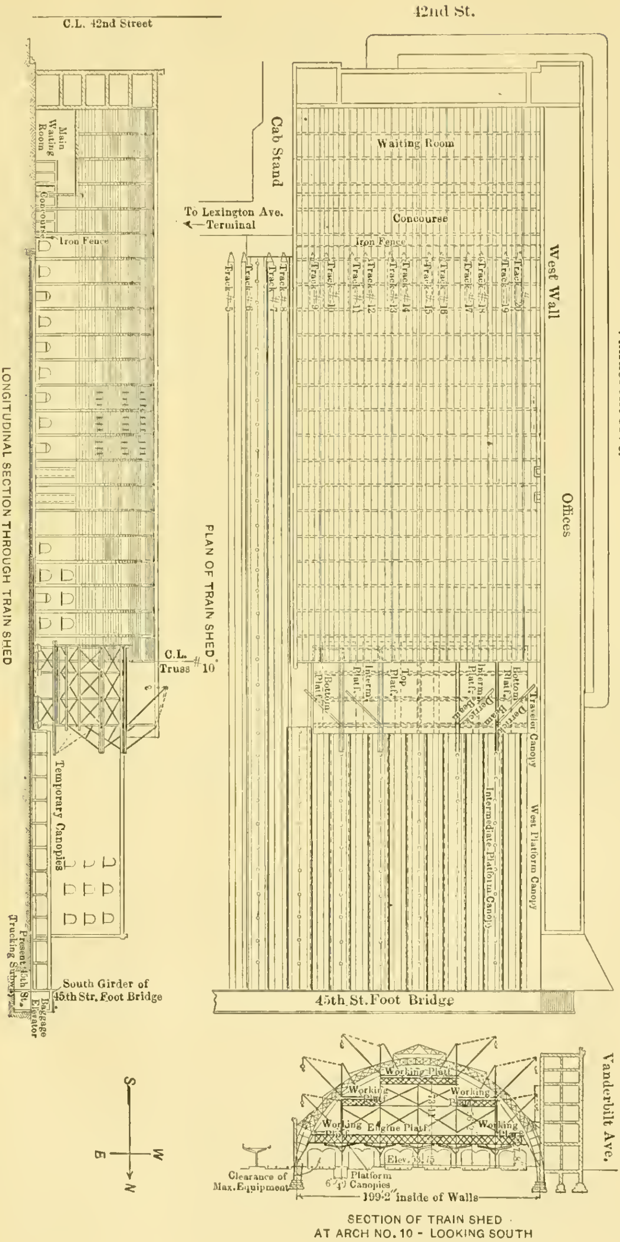


Mr. Harwood.



GENERAL METHOD OF PROCEDURE IN REMVOING TRAIN-SHED

Fig. 11.

Mr. Harwood. of 65 ft., which will permit of blocking up two of the shed trusses on it at the same time. It is equipped with six stiff-leg derricks which are operated by two engines. Flaps are hung on the sides, from the first platform level, to protect passengers on the outside station platforms. The south face is boarded over, forming a false end for the train-shed as the traveler moves south. The material was all framed at the company's yard at Harmon. Portions ready for erection were then brought into the station and set, up to the elevation of the first floor, at night. After this had been placed, the remainder of the traveler was erected during the day, the members being brought in and lifted to the first-floor level at night.

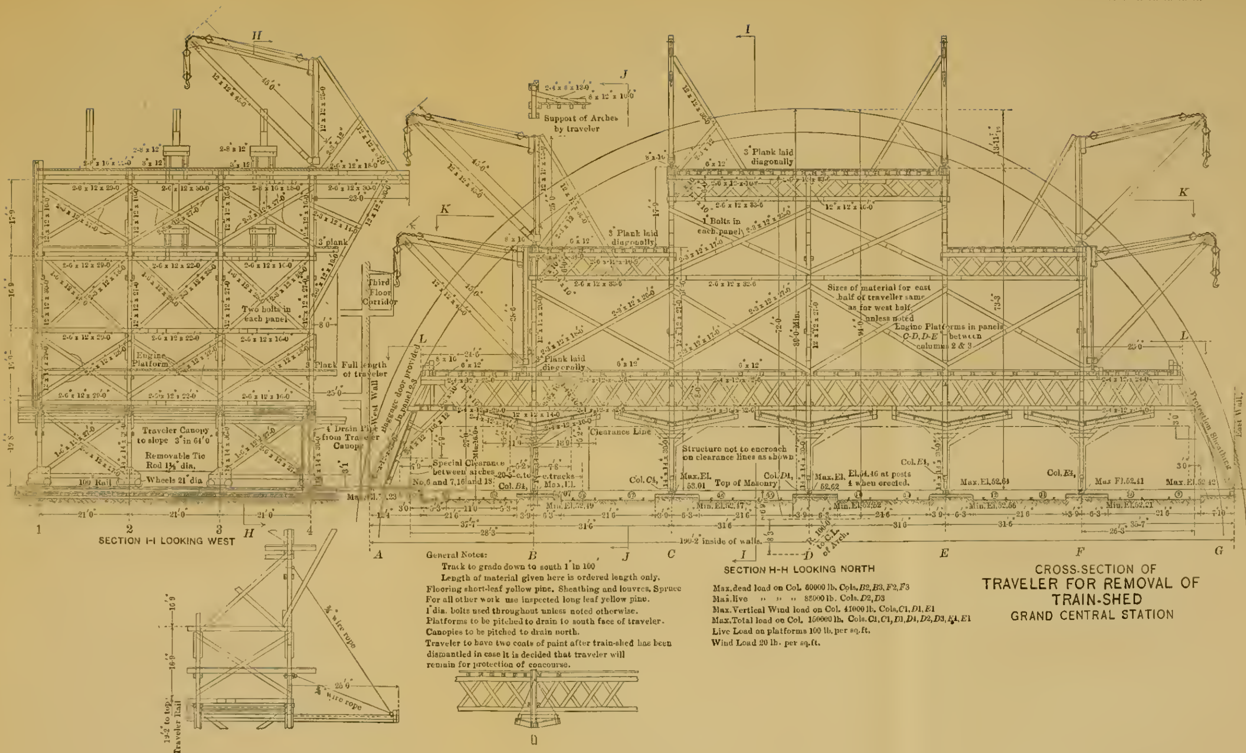
Fig. 11 indicates the general method of procedure. As fast as the traveler is moved south it is followed up with temporary wooden canopies, thus subjecting the platforms to a minimum amount of exposure. As soon as the traveler has been moved to a new position and blocked up, two train-shed trusses are blocked on top of it, and the corrugated-iron roofing, glass, skylights, and purlins are removed. The northerly truss is then cut into eight sections by using hack-saws and knocking off the rivet heads at the joints. The derricks then place these sections on the traveler platforms, all this work being done during the day. The night gang loads the material from the platforms in cars placed on the passenger tracks under the traveler.

The traveler is moved by jacks, two 15-ton jacks being placed on each platform, and each being operated by two men at signal so as to maintain a uniform movement. Fig. 1, Plate LXIV, shows the manner of applying the jacks. The work is done in units of 40 ft., and it requires 5 hours to move the traveler this distance. Now that the north portal and the first few northerly bays are removed, it is expected that the average progress will be about one truss for each 4 days.

The taking down of the north portal was the most delicate part of the operation, on account of the necessity for cutting away all connections between the portal and the train-shed before the work of demolishing could begin. The north end of the traveler was constructed with beams projecting 5 ft. beyond the face. These were pushed through the window openings, or through openings cut in the metal sheathing, and the entire portal was lashed to the traveler. Wooden troughs were constructed at the various platform levels so as to prevent loose metal from falling on the tracks below. Fig. 2, Plate LXIV, is a progress photograph of the removal of the north portal.

Fig. 3, Plate LXIV, a progress photograph giving a general idea of the work, was taken after the first two trusses south of the north portal had been removed, and shows the first canopy posts supporting the ends of the old canopy which existed north of the train-shed.

Fig. 4, Plate LXIV, shows the southerly face of the traveler, and,



General Notes:
Track to grade down to south 1" in 100'
Length of material given here is ordered length only.
Flooring short-leaf yellow pine. Sheathing and louvres, Spruce
For all other work use inspected long leaf yellow pine.
1" dia. bolts used throughout unless noted otherwise.
Platforms to be pitched to drain to south face of traveler.
Canopies to be pitched to drain north.
Traveler to have two coats of paint after train-shed has been dismantled in case it is decided that traveler will remain for protection of concourse.

Max. dead load on Col. 60000 lb. Cols. B2, B3, F2, F3
Max. live " " " 85000 lb. Cols. D2, D3
Max. Vertical Wind load on Col. 41000 lb. Cols. C1, D1, E1
Max. Total load on Col. 150000 lb. Cols. C1, C1, D1, D4, D2, D3, E1, E1
Live Load on platforms 100 lb. per sq. ft.
Wind Load 20 lb. per sq. ft.

CROSS-SECTION OF
TRAVELER FOR REMOVAL OF
TRAIN-SHED
GRAND CENTRAL STATION

through the openings, the canopy construction following the movement of the traveler may be seen. Mr. Harwood.

Fig. 12 shows the details of the temporary canopies. These are of the butterfly type, with a large overhang so as to provide maximum protection.

The preparation of the detailed plans and the execution of the work are under the charge of Mr. J. L. Holst, Engineer of Structures.

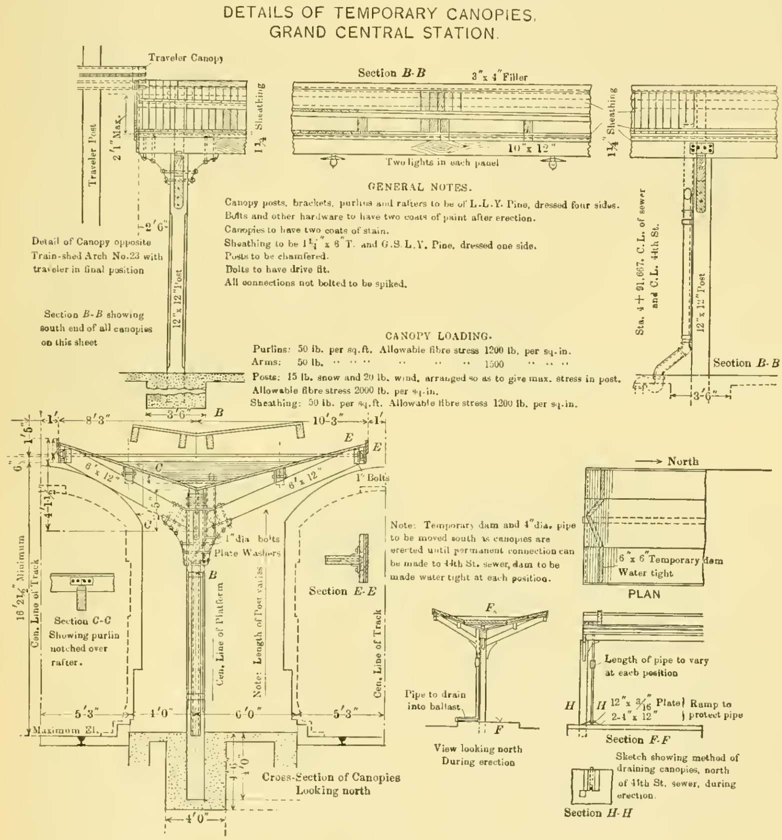


FIG. 12.

W. B. POTTER, M. AM. SOC. C. E.—This paper presents a comparison of steam and electric operation in a very interesting and comprehensive form, and the speaker wishes to express his appreciation of the valuable information given therein. Mr. Potter.

Reliability of service under electric operation is very properly regarded as a matter of first consideration. The installation of duplicate power-houses and storage batteries, considered as an insurance

Mr. Potter. may have been well justified by the importance of the traffic. The speaker believes, however, that the individual generating units will be found to be a sufficient reserve to ensure reliable service, and that the second power-house will ultimately be regarded as simply a plant for additional load.

The type of electrical equipment for the rolling stock was also selected with reference to reliability. Neither duplicate power-stations nor batteries would be of avail to clear a block caused by trouble with the rolling stock. The New York Central locomotive was designed to handle a 550-ton train, including the weight of the locomotive, and the heavier trains were to be double-headed. Double-heading, however, has not been found necessary, a single locomotive in ordinary service having frequently drawn an 800-ton train, that is, twelve or fourteen Pullman cars.

Mr. Murray has remarked that if one were constantly starting one would never get anywhere. This is equally true if one does not start at all. A successful locomotive must be capable of starting the train which it has the horse-power capacity to haul at the required maximum speed. The New York Central locomotives have about 137 000 lb. on the drivers, and, with sanded rails, can exert a draw-bar pull of 45 000 lb., and, while this is in excess of the usual requirement, it does ensure starting under abnormal conditions. The diagram, Fig. 2, given by Mr. Wilgus, shows the normal acceleration and speed of the locomotives with different weights of trains.

For the through trains on the New York Central, the locomotive was unquestionably the proper motive power, and for the local service either a locomotive or multiple-unit cars might have been used, but the selection of the latter was undoubtedly wise. As each multiple-unit car can be moved independently, yard movements are facilitated and also the making up of trains which are leaving on close headway, and, as either end of the train may be the head, economies are effected in the way of extra locomotives, track space, and switching that would be required in the case of locomotive operation.

Multiple-unit equipment, further, provides for an amount of power proportional to the requirement. The speaker doubts whether it is appreciated that an Interborough express train has the same motor-power equipment as a New York Central locomotive, and, with over 50% more weight on the drivers, the Interborough trains accelerate at nearly the slipping point. The equipment of each car with motors suitable for the service makes it possible to maintain the schedule with heavy trains with the same certainty as in the case of a single car.

The speaker has been particularly interested in the author's comparison of the cost of steam and electricity, as applied to locomotive service. His figures for work of this class, as the estimates fre-

PLATE LXIV.
PAPERS, AM. SOC. C. E.
MAY, 1908.
HARWOOD ON
ELECTRIFICATION OF SUBURBAN ZONE
OF N. Y. C. & H. R. R. R.

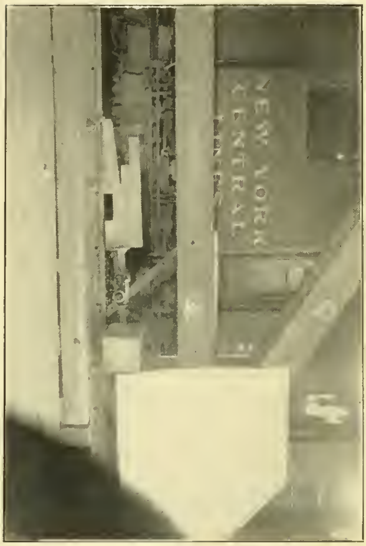


FIG. 1.—MANNER OF APPLYING JACKS, IN MOVING TRAVELER.

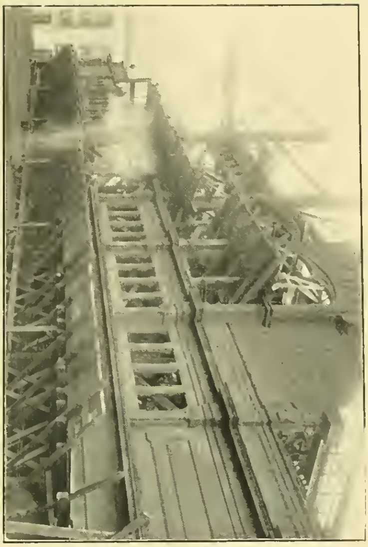


FIG. 2.—REMOVAL OF NORTHERN PORTAL OF TRAIN-SHED.

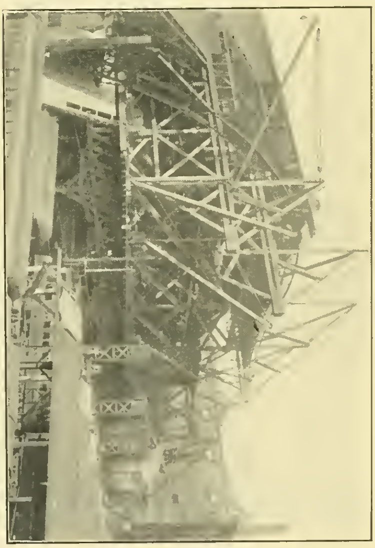


FIG. 3.—REMOVAL OF TRAIN-SHED. PROGRESS VIEW.

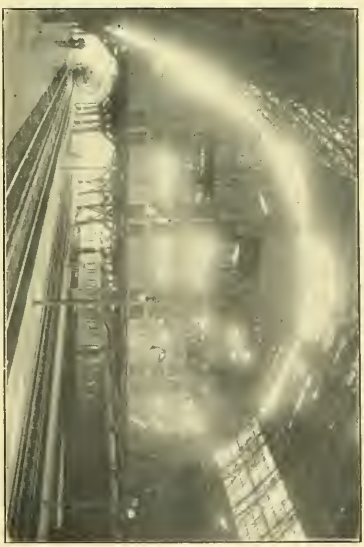


FIG. 4.—SOUTHERLY FACE OF TRAVELER.

quently indicate, show that there is a saving in favor of electricity which will be found in reduced incidental expenses rather than in the cost of direct operation. Mr. Potter.

There are some points in Mr. Murray's discussion with which the speaker does not agree. The reliability, initial cost, and cost of operation, for the service between New York and Stamford, are all in favor of direct current. This will be more especially true if multiple-unit trains are operated in the local service. The complexity incident to alternating-current and direct-current operation, and the relative cost, as influenced by the number of trains under the particular conditions in this instance, introduce features unfavorable to the alternating-current system. The speaker, however, does not wish to be misunderstood, or to be regarded as not favoring the alternating-current system under conditions more favorable to its application. He thoroughly endorses what Mr. Henderson has said in regard to the importance of studying the local conditions as affecting the general scheme of electrification, but believes that greater density of traffic usually insures a greater uniformity of load, so that the common expression, "density of traffic is favorable to electrification," is usually true.

Mr. Wilgus and his associates are to be congratulated on their success, and for the satisfactory service which they have rendered the public in the work they have done.

Frank J. Sprague, M. Am. Soc. C. E.—The Society is to be congratulated, and Mr. Wilgus complimented, upon the presentation of one of the clearest expositions of the results obtained in an electrical equipment of great magnitude, under particularly trying steam-railroad conditions. It is a concise and definite statement of facts, where real comparisons are possible, made in plain terms, and within the reach of such railroad men as are not trained electrical technicians. Mr. Sprague.

With the view taken by Mr. Henderson, the speaker wishes to express cordial agreement, as it is one which he has spent a number of years in voicing, namely, that the wisdom of the application of electricity to the equipment and operation of a steam railroad—and, it may be added, the method of such application if adopted—is a problem individual to the needs and conditions existing on that particular road, and cannot be determined by any general statements of real or fancied gains in fuel economy. Nor is it sufficient that economies, of whatever nature, promise a fair interest on the new investment required, for, if this were all, there would be no excuse for the adoption of electricity, because the existing investment would have derived no advantage, and every railroad man knows that there are ways of investing money in railroad improvements which will result in much more than a return sufficient to pay interest on the new capital—a fact not without vital influence at a time when money seeks the seclusion of the tin box and the stocking.

Mr. Sprague. At the present moment the field of electric application is more apparent where trunk-line terminals and suburban service are concerned, for here there are certain favorable conditions conducing to success, such as density of traffic, fair load factor, and reasonably uniform traffic conditions and train movements, while public sentiment favors the abolition of smoke and steam, and suburban populations demand more rapid and frequent train service.

In addition, there are mountain roads where onerous conditions are imposed by local difficulties or connecting divisions, and these, in spite of an irregular and widely varying traffic, may advantageously consider electrical operation. In both classes, however, the key-note is capacity, with an eventual gain in unit and system economy.

Up to the present, the most conspicuous example of terminal and suburban equipment is that so well described by the author, and with regard to this Mr. Murray states his belief that a mistake has been made, and that if the decision as to the most suitable equipment were to be made now "a majority of the best informed electrical engineers of America would to-day cast their ballot in favor of single-phase electrification."

Just here, and without expressing any opinion as to the merits of that system—and there are many valuable arguments in its favor for some classes of roads—the speaker must record his judgment that this statement is entirely wide of probability and fact, and, basing his conclusion upon the comparative results shown by the competitive systems now in operation on the New York Central and New Haven roads, must record his conviction that if confronted to-day with like actual conditions, not only would the majority—and he trusts all—of the members of the New York Central Electrical Commission, but any other competent board representing sound electrical engineering and conservative railroad practice, make the same general decision as originally made, modified possibly in degree of potential adopted and some details of equipment. Furthermore, it is quite possible that the somewhat extraordinary degree of insurance of operation would not now be decided upon, but it must be remembered that, to a certain extent, much of what was adopted was based upon the idea of providing insurance at first and utilizing capacity later.

In the development of the equipment for the New York Central, there were some matters of greater or less novelty concerning which some risk had to be taken, and among these there may be mentioned the vertical turbines, the particular type of locomotive, and the under-contact protected third-rail. There were ample criticisms and many predictions of failures as to all three features, and from many quarters; but it is with some degree of complacency that the speaker recalls his own connection with this development, and the practical results attending the adoption of these particular features.

There have been no "frightful accidents" with the turbines, and the locomotive has shown a capacity and reliability simply amazing, for it has been called upon for a hauling capacity of nearly 100% in excess of its guaranty. On some recent trials it has run more than 650 miles on consecutive days, stopping and starting within 6-mile limits, and has made nearly 700 miles within 24 hours under these conditions. The machines are not perfect, and, undoubtedly, some minor changes will ultimately be adopted, but it is instructive to note that, in the year and a half of operation, the makers have spent probably not more than $4 000 in changes on the entire equipment, and not one dollar of this has had to do with the motors or their design. In that period the electric service has increased up to a present maximum of about 300 movements a day by electric locomotive and by multiple-unit train, and in all that time there have been recorded but two electric locomotive failures on the track, where the locomotive could not pull itself. Its single-phase competitor, in one-third of that time, on 13 miles of road, and with a maximum of 60 train movements, has a record of at least 40 failures. If it had not been for the New York Central's electric locomotives, or multiple-unit trains, the effect of these failures would have been more pronounced. Mr Sprague.

Undoubtedly, the causes of some of these failures will be removed, and a better record will be shown by the single-phase machines, but it will be some time before there can be any final comparison between these two systems. Two facts, however, even now stare one in the face: The first is that the speaker's predictions (made some time ago) as to the relative capacity, have been borne out in practice, for while single direct-current locomotives have thus far pulled any load which has been put behind them, it is customary to use two single-phase locomotives whenever the trailing load exceeds six suburban cars or six cars of moderate weight. The second is that, in spite of the adoption of locomotive operation only, operative demands will eventually make necessary the use of multiple-unit trains for all suburban service out of great terminals; and, when this latter is attempted under the conditions which exist, the net result of the operation of such trains on combined direct and single-phase divisions will be absolutely disappointing when compared with the operation of multiple-unit trains on the direct-current system alone.

A third feature, which was a matter of some concern, was the under-contact third-rail, in the development of which the speaker had the pleasure of co-operating with Mr. Wilgus. It was possibly somewhat more costly to install this third-rail, under existing conditions, and it has been more or less costly in its up-keep. The speaker has made an effort to secure some comparative costs of maintenance of this rail, both on the New York Central and in other localities. The

Mr. Sprague. local conditions on the New York Central are unusual in character, and the accounts are complicated more or less by the costs of new construction, so that they cannot at the present time be given with any degree of reliability. One special difficulty has developed, to which attention should be called, which, however, is the fault of a collateral feature of the general equipment. A third-rail, whether of the top- or bottom-contact type, and especially where there is much special work, will operate most satisfactorily when the contact shoe is maintained within reasonable limits of operation. Unfortunately, the contact shoes on the New York Central locomotives are not carried, as they should be, upon the frames moving with the pilot trucks, but upon the superstructure above the equalizing springs. The result is a horizontal and vertical movement probably fully three times as great as would exist with another possible method of mounting. This has been the cause of some trouble at side inclines and elsewhere.

The speaker has reports of very different character from some other roads, where this particular difficulty does not exist, and where the operation is strictly normal, and comparable with other systems of working conductors. On the West Shore Railroad, operating for the 8 months from July 1st, 1907, to February 29th, 1908, with a monthly average of 77 204 miles, the official reports give the following averages per month during this period:

Material	$163.26
Labor for repairs	18.32
Labor for inspection	297.71
Total	$479.29

As this equipment covers 105 miles of track, the total cost chargeable to third-rail is $4.56 per month per mile.

The Philadelphia Rapid Transit Company has 12 miles of this rail, and the chief engineer reports an actual car mileage of 1 500 000 miles for 1907, and states that:

"The total cost of up-keep during 1907 has been practically nothing, as but two insulators required changing, and these were probably cracked when installed. The covering will probably require painting during 1908."

Of course, in time there will be opportunity to make direct comparison between the cost of up-keep of this type of third-rail and various types of overhead construction, and, concerning the latter, the speaker is inclined to think there will be some developments, but it is interesting to note one comparison, that of the maintenance of a top-contact unprotected third-rail and an overhead trolley of the usual class on one of the most important electrical railroads in America, operating nearly 128 miles of third-rail and nearly 20 miles of trolley wire. In this particular instance, recent reports show a ratio of 3.8 in favor of the third-rail. Mr. Sprague.

On the subject of the adoption of systems, and as illustrating how unsafe it is to predict what will be done in any particular instance, it is but proper to call attention to the fact that the electrical engineer of the Government railways in Belgium—they are all owned by the State—came to America a few months ago, from close proximity to alternating-current developments, and with some prejudice in favor of them. He had ample facilities for inspecting equipments in America, and on his return he reported to his government squarely in favor of using direct current and third-rail for the first equipment put in by the Belgian Government, and orders for a part of this equipment were recently supplied from America.

The speaker has also recently received from one of the foremost electrical engineers in England, Mr. Parshall, who has been identified with a large amount of electrical work there, is very familiar with conditions abroad, and follows developments there very closely, a letter which reads as follows:

"You have heard a great deal of alternating current development on this side of the water; I beg to assure you that they are all interesting from a laboratory standpoint, not one would meet the conditions of American practice; they are instructive as telling what not to do rather than what to do."

The speaker does not wish to prejudice the cause of the single-phase system. He seeks but the truth with regard to any equipment; there are cases in which he believes that that particular system may be used with advantage, but he holds unalterably to the view that the very best interests of electric railway development require every possible advance in either system, and that a hide-bound adherence to any one cannot but result in adverse developments.

Henry G. Stott, Esq.*—Mr. Wilgus gives certain reasons for duplicating power-stations and transmission lines, but a stage in the art has been reached where there is no longer any very strong argument for duplicating power-stations. The speaker's experience, covering quite a number of years with power-plants, is that the greatest danger now is not from the apparatus, but from the men who operate it. Take, for example, the two largest power-stations in New York City, each of which is giving out about 75 000 h-p., morning and night. In the past six years of operation the speaker can recall only two shut-downs which were due to the power-plant, and these were of very short duration, from 10 to a maximum of 20 min. It is obvious that the duplicate power-plant would not help such a situation, because, unless Mr. Stott.

* President, American Institute of Electrical Engineers.

Mr. Stott. all the boilers were fired up and the units were turning over continuously, at least 40 min. would be required to get them into service, by which time the power-station would be in operation.

In regard to the transmission lines, the power-stations necessarily must all be connected to a common system, otherwise, the copper installed would be wasted, and the rotaries in the sub-stations would have to be started over again, unless they were connected.

A few days ago there was a shut-down on the Manhattan system. The trouble was in the transmission lines, and was caused by one of the steam heating mains in the city destroying the insulation. If there had been a dozen power-houses, instead of two, there would have been the same trouble; and it can safely be said that it is a great deal better now to "put all our eggs in one basket and watch that basket, than to put them into separate baskets," in view of the fact that the human element is now the most dangerous one.

The operating and fixed charges are certainly less for one plant than for two. In working out power costs it is just as necessary to take into consideration the fixed charges as the operating costs. The fixed charges in many cases are greater than the actual operating and maintenance cost. It is a very poor plant to-day which cannot operate at less than 0.7 cent per kw-hr., that is, for operating and maintenance charges only. Fixed charges on a load factor of 50%, which is about the best load factor that railroads can expect, would be at least equal to that. Now, if the load factor is less, naturally the operating charges become of less importance, and the fixed charges of greater importance. In the plants described in the paper there is apparently going to be 100% spare apparatus. The safe over-load capacity of modern generators and turbines is usually given at 50% above rating, so that, even if the whole apparatus is being used, there is always a reserve capacity of 50%, making in this case a reserve of 200%; but, in considering peak loads, such as all railroads get during the morning and evening, due to the movement of suburban trains, a very much smaller load factor must be considered. In this case it is most important to keep down the fixed charges, which, under these conditions, may become three or four times as great as the operating charges.

Mr. Wilgus. William J. Wilgus, M. Am. Soc. C. E. (by letter).—As anticipated, this discussion has brought out many points which will go far toward solving the modern problem of electrifying steam railways.

Reference is made by Mr. Henderson, Mr. Gibbs, and Mr. Francis to the large collateral expenditures that have a bearing on the comparative cost of operation by steam and electricity; but one should not lose sight of the fact that such contingent expenses, in the installation under discussion, are a necessity entirely apart from the question of electrification. For instance, the enlargement of the Grand Central Terminal, four-tracking, elimination of grade crossings, and similar items, are required for handling a growing traffic properly. Indeed, it is true that most of these improvements are only possible with a change of motive power; but that is an added argument in favor of electricity. In other words, if there had not been the desire and necessity of radically enlarging the capacity of the railroad, it would have been possible to have made the change from steam to electricity, with a resultant material but insufficient increase of capacity, without incurring any expense for other improvements. Mr. Wilgus.

On the other hand, there are many places where the installation of electricity, with its superior operating advantages, will not only save in annual cost of operation, but, in addition, obviate the undertaking of expensive improvements that with steam would be necessary for increasing the capacity of the railroad.

Therefore, while it is proper to concede that the use of electricity on the New York Central has invited the undertaking of other large expenditures for greatly increasing the earning power of the railroad, it is not just, in comparing the cost of steam and electric operation, to charge against the latter fixed charges other than those made necessary by the change of motive power alone.

The proof of the wisdom of making these collateral improvements must rest on later developments of traffic, when the entire scheme has been completed.

Mr. Henderson's analysis of the relation of average to "peak" traffic is very interesting, and is borne out by an application of his reasoning to the New Haven Company's installation, had direct current been used between Woodlawn and Stamford. As shown below, the annual propulsion current requirements at the Cos Cob power-station bus-bars is, say, 18 500 000 kw-hr. The power-station installation for propulsion current is assumed to be 13 500 kw-hr. for direct-current operation. This would make the ratio of power installation to average train requirements 6.4, the cost of electric operation being slightly lower than by steam. Probably a ratio of 6 would represent an equality of cost of operation by steam and electricity under New York Central and New Haven conditions, which would agree closely with Mr. Henderson's conclusions, if he had omitted the portion of the power-station installation not intended for propulsion purposes. Of course, on both roads, increase of passenger traffic and the later handling of freight traffic, labor-saving machines, and yard switching by electricity, will raise the average load and produce still better results.

As intimated by Mr. Gibbs, it is perhaps too early to forecast with positiveness the saving of electric operation over steam in the various installations made to date, but the many misstatements to the contrary that have appeared from time to time in technical discussions certainly point to the wisdom of throwing some light on the subject before the means disappear of making true comparisons under like condi-

Mr. Wilgus. tions. Regarding locomotive repairs, there is no doubt that changing of types and details will ensue, and the cost will be chargeable to repairs; but such changes are also constantly occurring in steam practice. The obsolete machine is simply relegated to a less exacting service. It is true that duplicate facilities must be maintained for handling the motive power at interchange terminals; but there will soon be no occasion for expensive facilities for handling suburban steam locomotives, as their electric substitutes will require small inspection sheds only, and the relatively small steam locomotive plants for through trains will be removed from costly New York City real estate to outlying cheap lands. As Mr. Gibbs states, there is difficulty, on electrified steam roads, in obtaining the desired full mileage from electric rolling stock. However, where the traffic is dense, and reasonable attention is paid to this feature by the operating department, it is believed that reliance may be placed on the results outlined in the paper.

Mr. Waitt's reference to European practice, based on his personal investigations abroad, is both interesting and instructive. There is the frequent tendency among American engineers to urge the adoption of foreign practice unsuited to American conditions.

The remarks of Mr. Lewis and Mr. Francis point to the desirability of a future paper dealing with the New York Central improvements from the "static" standpoint, but this, of course, cannot be done until the work has more nearly approached completion. Mr. Katte and Mr. Harwood briefly touch on some of the details.

Mr. Brinckerhoff's experience with elevated railway service gives much weight to his comparison of the operating costs and efficiency of steam and electric motive power, especially the item of maintenance, about which some question has been raised.

As remarked by Mr. Sprague, if the question of electrification is to be approached from the standpoint of a fair return on the new investment required, something beyond the ordinary rate of interest must be offered to the investor. While the electrification of the New York Central was undertaken for reasons apart from possible economies of operation, as will be shown below, the prospective savings are such as to promise a return of not only the ordinary rate of interest on the capital invested, but also an additional amount for the stockholders, the aggregate of these two items being estimated at about 9% on the additional capital required for electrification. The cost of maintaining the third-rail in the initial zone is comparatively high, not for the reasons given by Mr. Sprague, but because of the many minor adjustments, alterations of tracks, and close inspection, all of which are incident to the newness of the installation.

Both Mr. Stott and Mr. Potter express the belief that reliable service may be amply insured without the precaution of a second generating station. Referring to the reasons given for duplicate power-stations,* it will be noted that two power-stations were decided upon:

* *Proceedings*, Am. Soc. C. E., for February, 1908, p. 72.

"each with sufficient capacity, utilizing its spare unit, and working Mr. Wilgus. 'overload,' to carry the entire demand of the service at the rush hours, should the other fail."

Had one power-station been decided upon, an additional unit would have been required for spare purposes, making 25 000 kw-hr. instead of 20 000 kw-hr., from which it will be noted that instead of 200% spare apparatus in the two power-stations, there is but 60%, including the 50% over-load capacity of the generators. Therefore the question of the wisdom of installing duplicate power-stations hinges upon the necessity of having this 60% excess capacity at the initial stages of the service, and the additional expense attendant upon the operation of two power-stations instead of one. The Electric Traction Commission considered these features thoroughly, and concluded that the company was justified in this additional expense for the reason that the geographical location of the two divisions, with their possible future extensions to the north, was such as to make unwise the adoption of but one station for both, located at a remote point and subject to such injury to itself or the connecting transmission lines as to make possible a long-continued interruption of train service. For instance, should a single power-station or its connecting transmission line suffer serious injury from rioters, strikers, accidents on adjoining property, or any other contingency sufficiently serious to place the power-station out of commission for a long-continued period, as was experienced on one of the English railroads which was electrified some years ago, a return to the use of steam locomotives would be imperative. The contemplated future electrification of freight service within the electric zone, including the terminals on the west side of Manhattan Island, the electrification of all or a portion of the Putnam Division, and the utilization of company current for lighting, yard switching, labor-saving devices, etc., promise a reasonably early use for the excess capacity of the power-stations sufficient to justify its expense for these insurance purposes during the early stages of electric operation. The location of the two stations, in better relation to the load centers than would be possible with one station, offers a saving of transmission losses that tends to compensate for the extra cost of operating two power-stations over one. That the company anticipates early need for the excess capacity is shown by its recent decision not to accept a proposition for its purchase or lease by outside commercial interests.

It will be interesting to note here that the cost of the power-stations was very low—less than $90 per kilowatt of capacity.

In the paper there was no intention of raising any issue with the representatives of the New Haven Company because that company saw fit to select a form of electrification different from that adopted by the New York Central. The paragraph first quoted by Mr. Murray had reference to the proven wisdom of adherence to the chosen type of

Mr. Wilgus. direct-current equipment, despite the urging by a manufacturing company upon the New York Central of the alternating-current-direct-current apparatus; and in no manner reflects on the use of the alternating-current system, *per se*.

However, as Mr. Murray has broached not only the question of the wisdom of the policy pursued by the New York Central, but also the wisdom of his own company's adoption of the alternating-current system north of Woodlawn, there is no recourse but to set forth the whole matter in sufficient detail for the drawing of correct conclusions. This is perhaps fortunate, for the present uncertainty in the minds of steam railroad officers is injurious to the advancement of the art of transportation, and peculiarly hurtful to the legitimate growth of electrification, whether by direct or alternating current.

There is really no quarrel between the alternating-current and direct-current systems. Both have legitimate fields. It is no more proper to compare them broadly than to contrast, say, a "Pacific" type passenger locomotive with a Mallet compound freight locomotive. They must be viewed in relation to a known service, and the care devolving on the engineer is to see that they are not misplaced. Legal restrictions, nature of traffic, mixture of steam and electric motive power, population, clearances, and other special conditions, all have bearings on the selection of the system of electrification best suited to any particular locality. It is a cause for congratulation that there is a choice of three systems, direct-current, alternating-current single-phase, and alternating-current three-phase, rather than but one system which would be unsuited to many localities seeking release from the limitations of steam.

The question, then, is whether or not either company has misapplied the system that it has adopted. The elements to be considered are:

(1) Physical and legal restrictions,
(2) Operating requirements,
(3) Safety,
(4) Reliability,
(5) Cost.

(1) *Physical and Legal Restrictions.*—The law taking effect July 1st, 1903, requiring the abandonment of steam in Park Avenue south of the Harlem River, within five years, gave an insufficient margin of time for the making of radical experiments, which, if unsuccessful, would cause delays alike distasteful to the public and the railroads. The temper of the public, atmospheric conditions in the Park Avenue Tunnel, and the congested nature of the Grand Central Terminal yard operations, were such as to dictate the utmost speed in effecting the change, in the interests of safety and public comfort.

Of the two systems, direct current and alternating current, the Mr. Wilgus. former offered apparatus of proven reliability and efficiency, whereas the latter, at the time of the decision in the fall of 1903, was declared by its warmest advocates to be unsuitable for meeting the onerous conditions of the case. The state of the art was not sufficiently advanced to warrant the use of a system still untried in heavy trunk-line service; but, apart from this reason, there were others of even a more convincing nature.

The four-track Park Avenue Tunnel, 2 miles in length, with a head-room affording but 1 in. of clearance above the top of the rolling stock, is confined between the city street pavements immediately over the roof and the city sewers beneath. The overhead conductors and contact devices on the equipment of the alternating-current system would require at least 2 ft. 6 in. more head-room, obtainable only by radical changes in city sewers, to obtain consent for which would be very problematical; and the lowering of 8 miles of tracks in solid rock, in a smoke-laden tunnel through which flows at nearly all hours of the day a congested traffic of from four to five times the volume of the New Haven Company's traffic north of Woodlawn. Even if feasible and safe, the cost of doing this would be prohibitive. That such a change of the tunnel is impracticable from the legal standpoint is well shown by the fact that the public authorities stopped the company from drilling and blasting for electric ducts in the side-walls of the tunnel, during the few hours of the night when traffic conditions permitted the prosecution of work of even that simple character; and pipe ducts were substituted, hung from the tunnel walls. Then, too, the required method of rebuilding the Grand Central yard in sections during a period of many years, in conjunction with the construction of lofty buildings over tracks carrying traffic, prohibited the use of exposed overhead trolley wires alive with a current as dangerous as 11 000 volts.

That Mr. Murray can place these trunk-line conditions, in next to the largest city in the world, in the same class with the totally different and vastly less complicated problems on his own line, and at the Sarnia and Simplon Tunnels, is strange. The repeated promises and as frequent failures of Mr. Murray's company during the past year to complete its change to electricity, with the resultant serious delay to the Grand Central Terminal reconstruction, and annoyance to the public in the Park Avenue Tunnel, are the most speaking commentaries on the offer of Mr. Murray to undertake so blithely this task for others.

Apart from these physical objections to the overhead alternating-current system, there were serious legal obstacles.

The four-track Park Avenue Viaduct north of the tunnel, 1½ miles long, was built under legislative enactment that prescribed the exact design. To modify this materially by the erection of trolley wires and supports would surely invite injunctions by abutting property owners, Mr. Wilgus. and resultant indefinite delays and enormous damages. The previous experience of the company with its neighbors in this thoroughfare, costing millions of dollars, taught a lesson that could not be disregarded.

The crowning legal obstacle was the objection of the city authorities to any form of overhead wires carrying high voltages along and over streets within the city limits. While the company has contended that transmission lines in the outer and sparsely-settled sections of the city, placed on the exterior edges of the right of way and passing far above the surface of intersecting streets, were permissible and even desirable, at least until the growth of population required a change to ducts, it did not feel that it could be denied that trolley wires carrying 11 000 volts immediately over and close to rolling stock and immediately beneath public travel on intersecting street bridges, would be sufficiently objectionable to invite ultimate adverse action by the public authorities that would entail a complete abandonment of a system costing the stockholders many million dollars.

With these absolute barriers to the use of high-voltage trolley wires, the New York Central, apart from other reasons, could not do otherwise than adopt the direct-current system, which in New York City is feasible and legal. The New Haven line, lying in the open country, did not have these obstacles, and adopted the alternating-current system.

(2) *Operating Requirements.*—The constantly increasing traffic in the congested Grand Central Terminal demanded a type of self-propelling electrical equipment that would minimize the number of switching movements across the throat of the yard. Experience elsewhere, also, had demonstrated the need of an elastic system of train operation, which, apart from the question of economies, would permit quicker acceleration, a more frequent service, and the regulation of the number of cars per train to the volume of traffic at different hours of the day. All these objects could be obtained by the use of multiple-unit cars, which, in the existing state of the art, seem best adapted to direct current.

The New Haven Company, in adopting alternating-current operation, rejected the use of the multiple-unit system, whereas the New York Central seized the opportunity to use it, with resultant immediate benefits to its operating department. That Mr. Murray's company now realizes its mistake is shown by its recent design of an alternating-current-direct-current multiple-unit train consisting of a motor car on each end of a six- or eight-car train. The success of such an arrangement is very questionable, from operating as well as electrical standpoints, and at least one large manufacturing company has declined to build it.

(3) *Safety.*—This item, referred to by Mr. Murray, should be considered in its twofold relation, to the employee and to the public, and

not to one alone. Both, naturally, deserve the very best judgment and care in deciding upon the kind of distributing system to be used. Mr. Wilgus.

Experience has shown that the low-voltage third-rail, suitably protected, is not dangerous. During the period of a year and a half that the working conductors have been energized in the congested initial electric zone of the New York Central, not a fatality has occurred either to employees or the public, primarily due to the third-rail or transmission lines. Three instances have been due to trespassing on the transmission line, another to a porter reaching beneath the third-rail for a pack of cards, and one to a prior contributing cause.

On the other hand, up to the present time, the New Haven trolley-wire system and transmission lines have apparently caused thirteen fatalities, largely due to wires not being "beyond the reach of the tallest man."

Carelessness, no doubt, is responsible for the majority of these unfortunate occurrences, but is it not a duty to select the system which local conditions dictate as least dangerous to the negligent employee?

As to the public, the third-rail is entirely removed from neighboring thoroughfares, and the transmission lines in sparsely settled districts, spanning far above intersecting streets, seem to have the advantage of safety, superior to that of 11 000-volt trolley wires passing directly beneath street bridges of low clearance above the tracks and within a few inches of the passers-by.

On the direct-current system, in case of accident, the passenger is as well or better guarded from the third-rail by means of protecting sheathing and circuit-breakers, as he is from knocked down trolley wires which may affect not one but all four tracks.

May it not be concluded that, as measured by both practice and theory, the direct-current system, in the territory under discussion, is preferable to the alternating-current system, from the standpoint of safety, having in mind the local conditions?

(4) *Reliability.*—This question is of first importance to a trunk-line railroad. What has experience shown in the two systems under discussion?

During two representative months the delays per 1 000 locomotive-miles between Woodlawn and the Grand Central Station, due to locomotive failures, were as follows:

New York Central direct-current locomotives..	1.2 min.
Steam locomotives..........................	2. "
New Haven alternating-current locomotives...	12.4 "

It will be noted that the alternating-current locomotives in this service caused eleven times as many train delays as the direct-current machines, and six times as many as due to steam power.

Mr. Wilgus. Since July, 1907, the New York Central has not had a single interruption of electrical service, whereas the New Haven Company, on its own territory north of Woodlawn, has had nine interruptions, of which four were very serious—in one instance lasting for 38 hours and necessitating a complete return to steam operation.

These facts demonstrate that, for reliability, the New York Central installation is far superior to that of the New Haven Company.

(5) *Cost*.—Comparisons of cost are absolutely valueless unless they are based on the same premises and conditions, and are in sufficient detail to permit analysis. To make a bare statement, unsupported by details, comparing the cost of the battery-less New Haven installation in the open country, with the one on the New York Central carrying four times the traffic through a section requiring expensive ducts instead of aerial lines, and into a terminal in the midst of a great city, is like showing side by side the cost per mile of the Union Pacific and Pennsylvania Railroads, without making allowance for the differences of topography, grades, number of tracks, terminals, and character of construction. Mr. Murray's ratio of 5 to 1 is misleading, as will be shown below.

Mr. Murray questions the accuracy of the saving of \$300 000 per annum by the New York Central from avoiding the use of the alternating-current-direct-current locomotive, but he attempts to analyze the smaller part (\$140 000) only, curiously enough not questioning the larger portion of the saving (\$160 000 per annum) due to the requirement for a less number of locomotives. His silence on this point is most impressive if one considers the millions of dollars that have been spent by American railroads in grade reductions in order to reduce the number of locomotives for handling a given traffic—not to increase them.

Mr. Murray attempts to cast doubt on the statement of lower energy consumption for the direct-current locomotives by comparing the alleged actual results of the New Haven alternating-current locomotive on direct-current territory, with the theoretical assumptions of an arbitrator, whose decision, by the way, his company rejected.

Upon carefully checking the correct mileage, weights, and bills for current, he may also find that his watt-hours per ton-mile should be 50.7 instead of 41.9. However, in the following figures, the writer has used the figure which is most favorable to his contention.

The energy consumption upon which the item of \$140 000 saving was based, was obtained in the following manner:

Several trial runs were made between the Grand Central Station and Woodlawn with both alternating-current and direct-current locomotives hauling identically the same weight of trains, at the same speeds, and with the same limited number of stops. The average results were:

Mr. Wilgus.

Alternating-current locomotive (New Haven)	36.7	watt-hr. per ton-mile.
Direct-current locomotive (New York Central)	28.9	" " "
Saving in favor of New York Central locomotive, equal to 27%	7.8	" " "

Another comparison is available, as a check on the foregoing results:

New Haven: Average consumption south of Woodlawn, including more stops than were made in the foregoing trial runs	41.9	watt-hr. per ton-mile.
New York Central: Observations for direct-current locomotive, including more stops than were made in the foregoing trial runs	33.8	" " "
Saving in favor of New York Central locomotive, equal to 24%	8.1	" " "

To be well on the safe side, in showing a money saving, a difference of but 15% in favor of the New York Central locomotive is used in the following comparisons, this agreeing with a careful study of the characteristics of the several electrical parts of both locomotives:

New York Central Electric Zone saving, due to use of direct-current instead of alternating-current-direct-current locomotives:	
Direct-current locomotive annual requirements at the contact shoes, 36 000 000 kw-hr. at $2\frac{6}{10}$ cents	$936 000
Alternating-current-direct-current locomotive annual requirements at the contact shoes, 36 000 000 kw-hr. plus 15% excess, 5 400 000 kw-hr. = 41 400 000 kw-hr. at $2\frac{6}{10}$ cents	1 076 400
Annual saving	$140 400

New Haven loss, due to use of alternating-current-direct-current locomotives instead of direct-current locomotives south of Woodlawn:	
Direct-current locomotive annual requirements at the contact shoes, 10 500 000 kw-hr. at $2\frac{6}{10}$ cents	$273 000
Alternating-current-direct-current locomotive annual requirements at the contact shoes = 10 500 000 kw-hr. plus 15% excess, 1 575 000 kw-hr. = 12 075 000 kw-hr. at $2\frac{6}{10}$ cents	313 950
Annual loss	$40 950

Mr. Wilgus. It thus appears that by adopting a locomotive suited to the system over which it is to operate, the New York Central will effect a saving, for current only, over what would have to be expended if a locomotive had been adopted for operating on two systems, of $140 000 per annum; whereas the New Haven Company, by adopting the reverse policy, will suffer a loss between Woodlawn and the Grand Central Station of $40 950 per annum.

Mr. Murray asks why not compare the direct-current and alternating-current locomotives, apart from the complications attending the necessity of performing two functions by the latter. To do this requires a study of the first costs and annual costs of operation, by both systems, in the same territory, under precisely the same conditions, and embracing all variable elements. Therefore the writer has selected the New Haven Company's line, between Woodlawn and Stamford, having a total single-track mileage of more than 100 miles, in which is included a number of small yards.

The cost of the generating station for direct-current operation is found to be at least 20% cheaper than the one intended for alternating-current operation, for the reason that the generators for the latter, as built by the New Haven Company, are designed for three-phase output, but they are utilized for single-phase purposes, which largely cuts down their capacity. Then, too, the magnetizing of the motor fields of the alternating-current locomotives requires a large amount of wattless current not needed with the direct-current system. These conditions result in a much larger generator installation than would be needed for the direct-current system, and a corresponding higher cost.

To do the same work required on the 41 locomotives ordered by the New Haven Company, only 28 would be required for direct-current operation. This is due to the limit of five to six passenger cars to a single New Haven locomotive in order that the schedule speeds may be maintained, as compared with the ability of the New York Central type to make the same schedules with two or three times that number of cars. In other words, heavy trains require to be double- and triple-headed with the alternating-current locomotives, whereas but one direct-current locomotive of substantially the same weight and cost would suffice. The evil feature of this system is not only the heavier annual cost for repairs and for current needed by the additional alternating-current locomotive, but there is the serious operating handicap of holding spare units in readiness to attach to trains which, at the last moment before leaving the termini, are found to be heavier than was at first anticipated. The cost per locomotive of each type is taken at $30 000, but reliable information points to a considerably higher cost for the alternating-current locomotive, having approximately one-half to one-third the capacity of the direct-current locomotive.

The effect of this excess number of locomotives is to counteract the saving due to the superior efficiency of the distributing system of the

New Haven Company. For the 28 direct-current locomotives, it is estimated that 15 000 000 kw-hr. annually will be required at the contact shoes, based on which the requirements of the generating station bus-bars, with an efficiency of 81% (New York Central results) for the distributing system, would be about 18 500 000 kw-hr. The larger number of alternating-current locomotives for doing the same work increases the current demand at the contact shoes for the locomotive ton-mileage, so that, as compared with the 15 000 000 kw-hr. for direct-current operation, there is needed a 15% increase for the alternating-current system, or 17 250 000 kw-hr. Mr. Wilgus.

On the basis of 95% distributing-system efficiency, this makes the requirements at the bus-bars of the generating station 18 200 000 kw-hr. Thus it is seen that, for the two systems applied to the territory in question, the demands of current at the generating station bus-bars are nearly alike.

The depreciation of the alternating-current system under New Haven conditions is greater than that of the direct-current system, because of the comparatively short life of the trolley wires and catenary construction, which are subjected to abrasion and corrosion. This item is aggravated in this instance by the combined operation of electricity and steam. In both systems, due to deterioration and obsolescence, a life of twenty years may be used, except for the trolley wires and catenary system, for which an extreme life of five years is assumed. Personal observation prompts the belief that the last named period is much greater than will be actually experienced.

Maintenance of generating plants and distributing systems is taken at 2% per annum for both systems. Observations of the annual costs of maintenance of third-rail and overhead work show that they are about equal where the conditions are similar, provided that separate provision is made for the more rapid depreciation of the latter.

The maintenance of the New Haven locomotives is assumed to be 6 cents per locomotive-mile, as compared with 3 cents per locomotive-mile for the New York Central locomotive. The excessive number of locomotive failures, the complication of parts, and the known large size of repair gangs at Stamford and at the Grand Central Terminal indicate that the cost of maintenance of the New Haven alternating-current locomotives is much in excess of the assumed figure. Thirty thousand miles per annum per locomotive is assumed for both systems.

The actual experience of the New Haven Company with injury to employees due to contact with 11 000-volt trolley wires warrants the assumption that payments for damages will be at least five times those that are chargeable to the use of the third-rail of the direct-current system. Low clearances at street bridges for trainmen and other employees; possibilities of falling wires caused by abrasion and corrosion and from high winds similar to those that recently felled neighboring wires, trees, and poles, causing a temporary suspension of New Haven

Mr. Wilgus. service; and proximity of signals to the trolley wires—all these aggravate the New Haven situation.

With these explanations, the following statements have been prepared.

Comparative Estimated Costs of Electrification of the N. Y., N. H. & H. R. R. from Woodlawn to Stamford.

Items.	Direct-current.		Alternating-current.	
Generating stations	....	$1 300 000	....	$1 500 000
Distributing systems	Sub-stations $600 000		Trolley system. $500 000	
	Working conductors, etc.................. 800 000		Overhead bridges, etc.. 750 000	1 250 000
	Transmission lines, etc.................. 550 000	1 950 000		
Rolling stock	28 locomotives................	850 000	41 locomotives...........	1 250 000
Totals	....	$4 100 000*	....	$4 000 000†

* Based on a liberal estimate of probable cost.

† Based on a liberal estimate of probable cost. The New Haven Company's annual reports indicate a higher cost.

Comparative Estimated Annual Costs of Operation by the Direct-Current and Alternating-Current Systems: Woodlawn to Stamford.

Items.	Direct-current.		Alternating-current.	
Fixed Charges { Int. 4%, Taxes 1½%, Ins. and Risks 1½% }	7% on $4 100 000.....	$287 000	7% on $4 000 000......	$280 000
Depreciation. (Annual sums required to accumulate a fund sufficient to extinguish cost at expiration of assumed life)................	Assumed life of 20 years = 33.60 per $1 000.............	137 760	Assumed life of 20 years, except for the trolley system = 33.60 per $1 000......$117 600	
			Assumed life of trolley system, 5 years = 184.60 per $1 000...... 92 300	209 900
Maintenance.				
Generating stations	2% of cost....$26 000		2% of cost... $30 000	
Distributing system.............	" " " 39 000		" " " ... 25 000	
Rolling stock.....................	840 000 miles @ 3c....... 25 200	90 200	1 230 000 miles @ 6c....... 73 800	128 800
Operation (exclusive of maintenance).				
Generating stations	18 500 000 kw-hr. @ $\frac{5}{10}$ cent.......$92 500		18 200 000 kw-hr. @ $\frac{5}{10}$ cent........ $91 000	
Sub-stations......................	3 @ $6 000... 18 000			
Distributing system.............	Personal injuries...... 5 000	115 500	Personal injuries...... 25 000	116 000
Grand totals....................	....	$630 460	....	$734 700

Mr. Wilgus.

Excess cost of alternating-current over direct-current north of Woodlawn	$104 240	per annum	= 16 per cent.
Add excess cost of operating alternating-current locomotives on direct-current territory....	40 950	" "	
Total excess cost to New Haven Company by adoption of alternating-current system	$145 190	" "	= 23 per cent.

It will thus be seen that, under the conditions of traffic on the New Haven Road, the alternating-current system costs 16% more to operate than the direct-current system, and if the excess cost of operating alternating-current locomotives on direct-current territory is considered, the loss is 23 per cent. If, in the estimate, consideration had been given to the use of the multiple-unit cars for suburban service, instead of locomotives, the direct-current system would show even a greater saving.

Summarizing all the cost figures for steam, direct-current and alternating-current service, on both the New York Central and the New Haven lines, and adding locomotive wages, so as to agree with the conclusions given in the paper, the comparative annual results are:

New York Central (Ultimate Electric Zone).

Steam.................. 1 000 000 thousand car ton-miles at $2.77	$2 770 000
Electric: direct-current.. 1 000 000 thousand car ton-miles at $2.02	2 020 000
Difference: Saving............................	$750 000

New Haven Company (Woodlawn to Stamford).

	Direct-current. (Not adopted.)		Alternating-current. (Adopted.)		Loss by alternating-current system.
Steam	262 500 thousand car ton-miles @ $2.77	$727 125	262 500 thousand car ton-miles @ $2.77.........	$727 125	
Electric......	262 500 thousand car ton-miles @ $2.71 ±.......	711 310	262 500 thousand car ton-miles @ $3.11 ±.......	815 550	
Difference...	Saving	$15 815	Loss....................	$88 425	$104 240
			Add for loss south of Woodlawn...........	40 950	40 950
			Total loss...............	$129 375	$145 190

Mr. Wilgus. These figures are not claimed to be absolutely accurate, as the New Haven Company's detailed costs have been withheld, but they are based on careful estimates made on the ground, and are believed to be sufficiently accurate to enable one to form the reasonably correct conclusion that, under the conditions existing on the territory in question, the New York Central, by reason of its adoption of the direct-current system, will ultimately show a very large saving over steam, say, approximately, $750 000 annually; and that the New Haven Company, instead of showing the saving over steam of, say, $15 000 annually, which might have reasonably been expected from the use of the direct-current system, will show a loss of, say, approximately, $129 375 annually, a total net loss of, say, $145 190 annually.

Conclusions.—From the foregoing it seems proper to draw the conclusions that the early decision of the New York Central to adopt direct-current to fit its local conditions, has resulted in the following advantages that would have been denied had the system been selected that Mr. Murray's company later adopted:

(a) Timely relief of the public from the products of combustion and noise of the New York Central's steam locomotives in the Park Avenue Tunnel and along Park Avenue;

(b) A reliable and efficient substitute for steam service;

(c) Compliance with the law, and hence absence of danger of drastic fines for use of steam locomotives in Park Avenue after July 1st, 1908;

(d) Promise of substantial economies of operation which, upon the completion of the change to Harmon and North White Plains, are believed will approximate $750 000 annually.

As previously stated, Mr. Murray misinterprets the intention of the paragraph he first quoted; and, in "controverting the statements made against the alternating-current system," he has undertaken an unnecessary task. The writer has genuine pleasure in cordially agreeing with him "that the near future will see the high-voltage distribution system, with its attendant alternating-current locomotive, propelling trains from and between terminals * * * "; but, for the sake of both stockholders and public, let it not be forgotten that there are other items, in addition to "density of traffic," which, if ignored, lead to a misapplication of a worthy system.

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PAPERS AND DISCUSSIONS

RECENT DEVELOPMENTS IN PNEUMATIC FOUNDATIONS FOR BUILDINGS.

Discussion.*

By F. W. Skinner, T. Kennard Thomson, and Louis L. Brown.

F. W. Skinner, M. Am. Soc. C. E.—Mr. Usina has presented in a very interesting manner the recent developments of pneumatic-caisson work. The speaker agrees with the author in his description of some of the latest and most important developments; but as the first application of pneumatic-caisson work to foundations for buildings is scarcely more than fifteen years old, and as its development has included many changes from the original caissons, which were operated without essential deviation from the methods used in constructing submerged foundations for bridge piers, it may be excusable to call attention to a few other matters which seem to be important links in the chain of development, from the Manhattan Building to the United States Express Building, the City Investing Building, the Singer Building, and the Hudson Building, recently completed, all in New York City. There are only two or three instances where pneumatic caissons for buildings have been used outside of New York City. Mr. Skinner.

The first pneumatic-caisson foundations for buildings—those sunk in 1893 for the Manhattan Building—were simply riveted-steel caissons, assembled partially at the site and sunk by the usual process. They were costly, and open to some objections from structural considerations.

* This discussion (of the paper by D. A. Usina, Assoc. Am. Soc. C. E., printed in *Proceedings* for March, 1908), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Skinner. They were made excessively heavy, in order to carry the entire weight of the massive pier and its load, sustained, in the first place, by very heavy and deep transverse girders reinforced by knee-braces to the cutting edge, making a costly and complicated construction. Since that time this type has been entirely eliminated, an advantage largely due to the improvements described by the author.

One of the first important changes was the substitution of wooden walls and a wooden deck for steel in rectangular caissons, thus effecting a reduction in the cost and a greater reduction in the time. The rectangular caissons were built of solid courses of timber, sheeted and caulked inside and outside, and with crossed courses of timber, sheeted and caulked inside, for the deck.

The next considerable stride was made by the substitution of cylindrical caissons with wooden staves for rectangular ones. These were more easily made, handled, and sunk, and were more economical. John F. O'Rourke, M. Am. Soc. C. E., who has built a large number of difficult and important pneumatic-caisson foundations, is to be credited with that improvement. His courage, resourcefulness, and indomitable energy have been important elements in this field. To him is also due the credit for a simple method of controlling the escape of air from caissons. In sinking a caisson, it is often difficult to lower it after the cutting edge has been undermined a considerable distance. The side friction is sometimes so great that the weight in and on the coffer-dam is inadequate to lower it, and the air pressure has to be diminished by letting it "blow out," as it is technically called. A blow-out is likely to be quite a critical operation, especially if the adjoining buildings are on quicksand. Thus the foundations might be jeopardized if material entered under the cutting edge in large quantities. Mr. O'Rourke simply bevelled the cutting edge, thus providing a high point which located the blow-out, and that was arranged to be on the safe side of the caisson.

Still more important, however, and perhaps of greater value than any other single feature of the development of caissons, has been the improvement made by D. E. Moran, M. Am. Soc. C. E., in the air-lock for the passage of materials. The man-lock has been practically unchanged, but, originally, the air-lock, through which materials were removed from and introduced into the caisson, was like the man-lock, and it was necessary, in removing a bucket of spoil or introducing a bucket of concrete, to detach it from the hoisting tackle and handle it by separate apparatus inside the lock. That is wholly unnecessary now, but it was formerly impossible to keep the bucket in uninterrupted connection with the derrick.

After considerable experience with caissons, the upper door of the air-lock was made in two leaves, fitted on the joint line, with a detachable stuffing-box engaging the hoist line. The upper door can

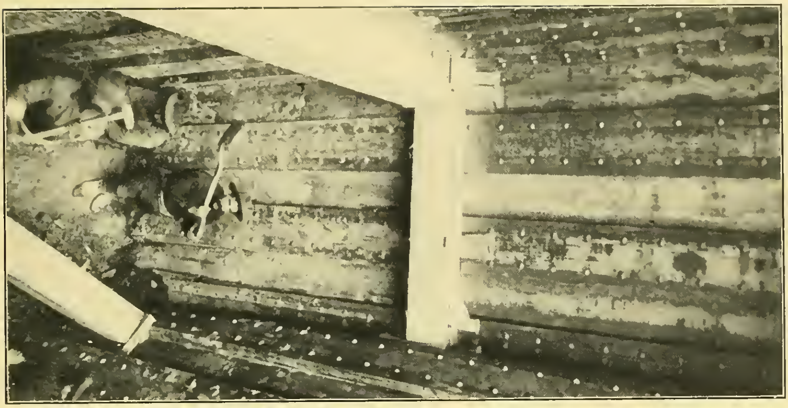


FIG. 1.—OLD-STYLE HEAVY STEEL SHEETING FOR FOUNDATION PITS.

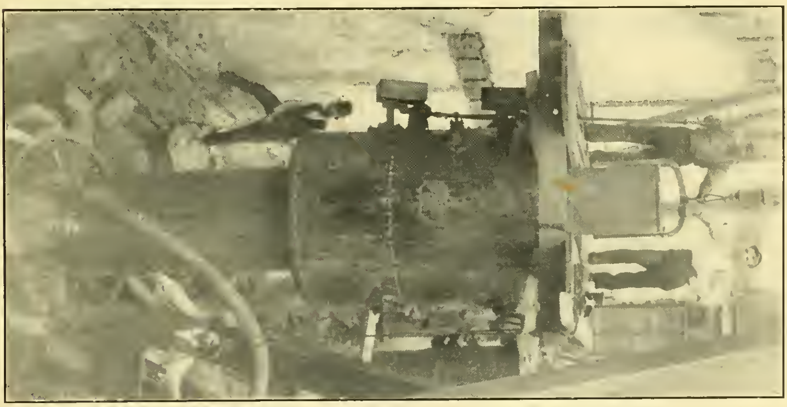


FIG. 2.—DETACHABLE STUFFING-BOX ON HOIST ROPE.

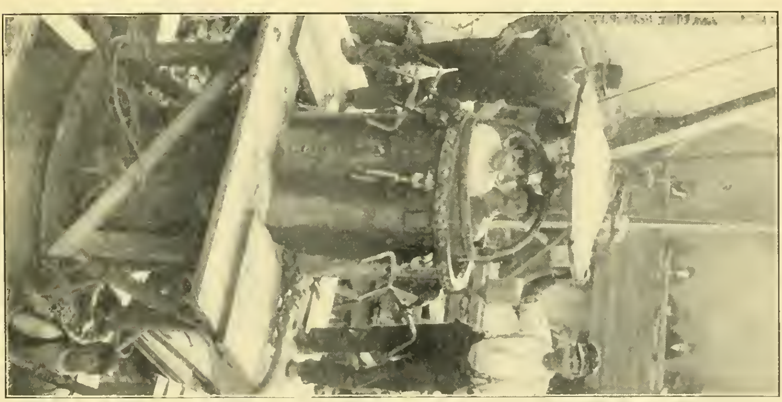


FIG. 3.—DETACHABLE AIR-LOCK DOOR.

now be opened or closed regardless of the position of the line, which passes freely through it and need never be detached from the bucket. The stuffing-box permits the bucket to be handled almost as rapidly and as easily as in an open caisson, and with very small loss of air, and probably increases the rapidity of removing and entering material more than 100 per cent. Buckets can now make a round trip in and out of the caisson in less than 1 min., and previous to this improvement 2 or 3 min. were often required. Mr. Skinner.

Another method of maintaining the attachment of the bucket to the hoisting line while passing through the air-lock was devised by Mr. O'Rourke, in the work for the Commercial Building. He made the top door of the air-lock detachable, and connected the stuffing-box permanently to it.

Another important development is the special arrangement by which the exterior caissons supporting the wall columns of several of the largest and most important buildings in New York City have been made to form a continuous water-tight wall, or, as it was termed in one of the first applications, a sort of dam enclosing on some or all sides the whole site of the building, and thus, theoretically and sometimes actually, avoiding the necessity of using pneumatic caissons for the interior piers. Having enclosed the building, a flow of quicksand from beneath adjoining buildings is prevented, thus allowing interior piers to be built in open excavations or coffer-dams.

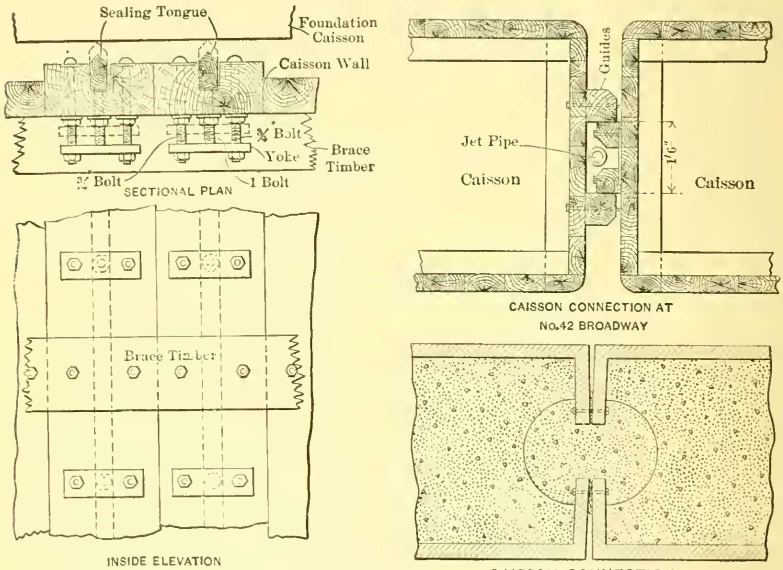
Considerable difficulty is found in making this construction, because there is a limit to the size of caissons which can well be sunk, and a length of about 30 ft. is a maximum for wall caissons supporting two columns or, possibly, three. In a building 100 ft. or more in length it takes several such caissons for the walls. Some clearance must be left between their ends, and that clearance may be from 4 in. to 12 or even 18 in., and its closure, and the permanent exclusion of groundwater and quicksand, in cases where the sand is very lively and the pressure is heavy, is a difficult problem, and has been met in several ways.

In the Commercial Cable Building, rectangular steel wall caissons were sunk a few inches apart. Pipes were jetted down between the adjoining ends, and after they reached the hardpan, 50 or 60 ft. below the surface of the street, they were filled with clay cartridges rammed by a piston or plunger operated by a pile-driver. The pipes were gradually raised, distributing the clay vertically, and such great pressure was developed by the ramming that it sheared the steel plates of the caisson itself, and very effectively excluded the water until the brick lining was built in the excavation.

This method was not in all respects satisfactory, and, not long after, Mr. O'Rourke devised a method of making a continuous concrete bond between adjoining caissons of the Stock Exchange and other

Mr. Skinner. buildings. As the caissons were sunk, semicircular recesses or wells were formed in the adjoining ends of both caissons and coffer-dams.

After the caissons were sunk and concreted, men entered and bolted the adjoining outer wooden walls together, removed the center part, between the bolts, and caulked the cut edges. The two wells were thus combined in a single one with a 4 by 5-ft. cross-section and an outline like that of a button-head rivet, which was filled with concrete, thus making a key and a bond between the caissons.



CAISSON CONNECTIONS, BANK OF STATE OF NEW YORK.

CAISSON CONNECTION, STOCK EXCHANGE.

FIG. 22.

Later, various ingenious methods were used in bonding wall caissons together. In the Bank of the State of New York the wooden caissons were sunk about 8 in. apart, and one caisson was provided with a pair of 2-in. vertical timbers, 12 in. apart, recessed 3 in. into its thickened wall. These vertical ribs, which projected at first only very little beyond the face of the caisson, were equipped with 1-in. stud-bolts or set-screws projecting through the wall of the caisson and bearing against heavy steel yoke pieces secured by bolts to the wall of the caisson. After the caisson was sunk, the nuts on all the screws were operated to force the ribs out through the quicksand until their sharp cutting edges penetrated the wooden face of the adjoining caisson, and made a fairly tight and satisfactory joint, excluding the quicksand from the interior of the excavation.

Another method of connecting adjoining caissons was made somewhat more simply for the building at 42 Broadway, where the first caisson was sunk with a couple of vertical 6 by 8-in. guide-timbers bolted to the end wall, the second caisson being located very close to Mr. Skinner.

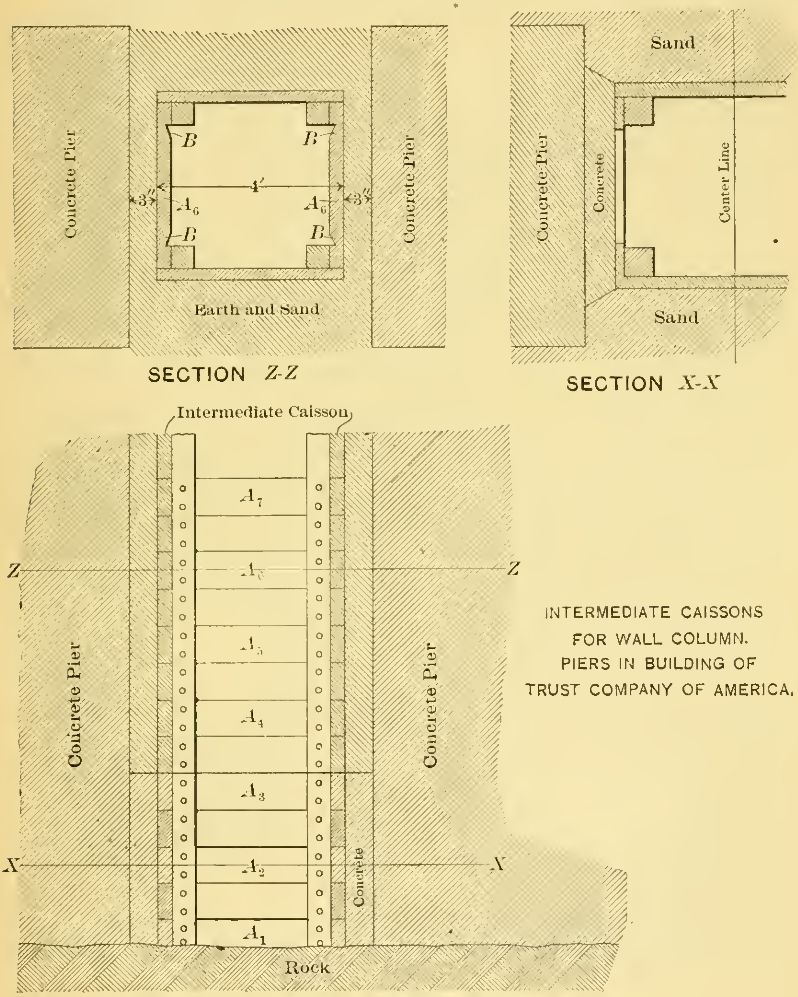


FIG. 23.

these guide-timbers and provided with interlocking guide-timbers. Together, they formed a sort of tongued-and-grooved joint, effectually closing the space, and leaving a small core through which a pipe was jetted down, thus serving to scour out the sand down to rock and leave a space for the introduction of grout.

Mr. Skinner. Afterward, still another method was devised by the same contractors. A special sheet-pile was driven in the quicksand across the gap on both sides of the coffer-dams, protecting the narrow space between their ends, so that men could enter and excavate down to the

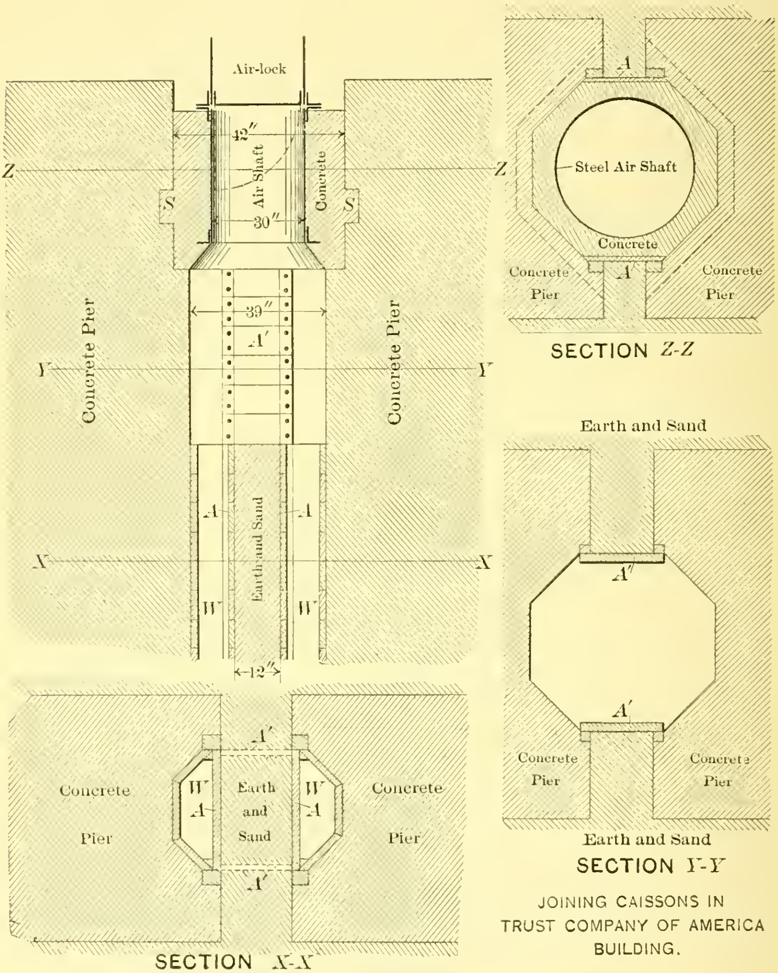


Fig. 24.

deck of the caisson, below which the narrower space was cleared by post-hole diggers down to the cutting edge and the spaces concreted.

In the building for the Trust Company of America different methods were adopted for connecting adjoining wall-column piers. These piers were sunk about 4½ ft. apart by the pneumatic-caisson process before it was decided to make them continuous. Small intermediate wooden rectangular caissons, of the full depth of the piers, were sunk between them, and after they were landed on rock, alternate horizontal planks, A_1, A_2, A_3, A_4, A_5, A_6, etc., in the sides next the large caissons were successively removed, the sand scooped out, from the bottom up, and the spaces filled solid with concrete, after which the caisson itself was concreted, thus sealing the space between the large caissons or piers. The removal of the side pieces A_1, A_2, etc., was facilitated by the slots, *B, B, B*, etc., cut half through them when the caisson was built. Every other side piece was made without the slot, and was left in position to tie the caisson together. Mr. Skinner.

Other pneumatic-caisson wall-column piers in the same building were sunk about 12 in. apart, with the intention of joining them afterward, and, to provide for this, semi-octagonal recesses 42 in. in diameter were cored in the concrete in the adjoining ends of the piers when the latter were made, and were closed on the outer faces by horizontal tongued-and-grooved side boards, *A, A, A*, making a well, *W*, in each end of the pier. After the caissons were sunk and concreted, completing the piers, the earth and sand between them was excavated to the depth of 1 ft., and the first pair of boards, *A, A*, was taken out, cut and nailed on again in the position, A^1, A^1, at right angles to their first positions. This operation was repeated, thus completing the octagonal well between the two piers. When the well was deep enough, a short vertical section of a steel air-shaft cylinder was set in it and concreted, and an air-lock was assembled to it and pressure put on. Slots, *S, S*, in the pier concrete were filled with the shaft concrete, thus acting as keys to prevent the pressure from blowing out the shaft. The men then entered and continued the excavation and removal of the boards, *A, A*. In this way, the excavation between the piers was carried to the bottom and afterward concreted, thus making a positive and efficient bond between the piers, the first time that it had been accomplished under pneumatic pressure.

A minor improvement in pneumatic-caisson work, but one which contributes materially to economy and rapidity, is the substitution of 1 000-lb. castings, with connections for hoisting tackles, for the small pieces of pig iron formerly used to ballast caissons. An equivalent device is a heavy rectangular box in which is placed 1 000 or 2 000 lb. of pig iron, kentledge, or its equivalent. These boxes are compact, and easily handled and piled. Either form of ballast can be much more advantageously piled around the air-shaft or on the pier than pig iron, and can be handled very rapidly by a small hoisting engine, thus eliminating hand labor.

The reference to the connection between concrete piles and pneumatic caissons, in the latter part of the paper, is very interesting. The two are opposite extremes of difficult foundation work, but there is an

Mr. Skinner. important space intervening between them. The usefulness of a concrete pile terminates with the requirements for bearing strength greater than that of a single pile of such dimensions that it can be advantageously driven. The perfect concrete pile has not yet been devised, and, for loads of more than about 20 tons, there are few examples of anything except pneumatic caissons for many cases of pier foundations in soft ground.

Caissons are so expensive and excavation so difficult for piers from 3 to 5 ft. square that there is a very important gap to be filled in the construction of small piers having a capacity greater than that of a single pile and less than can be made economically with the pneumatic caisson. The speaker is not aware of any examples of satisfactory construction for such cases, but he knows that simple designs have been made, which appear to be entirely practicable and very economical, for sinking small concrete piers 3 or 4 ft. in diameter, in soft and wet ground, without the necessity of pneumatic-caisson or coffer-dam work.

On general principles, the reinforcement of either a pile or a pneumatic-caisson pier with steel rods in compression is to be avoided. Ordinarily, the pier or other foundation should be essentially a masonry structure; steel reinforcement should only be tolerated when a bending moment or flexure is unavoidable. For this reason the speaker does not believe in the use of steel reinforcement for compression stresses in piles or in piers; he is aware that it has been used, and, further, he is aware that extremely high values have been permitted for steel used in this way in compression; but he has strong objections to it.

Although the speaker wishes to express the utmost admiration for, and satisfaction with, the splendid work that has been done in pneumatic caissons, yet he thinks the tendency has sometimes been to overdo it. Pneumatic-caisson foundations are a form of construction essentially and inherently very costly, and, when not indispensable, very extravagant. Where equal security can be otherwise obtained, pneumatic caissons should not be used, although in many, perhaps in most, cases where they have been adopted their use has been unavoidable or justifiable. In some cases other forms of construction would have served equally well, and would have avoided excessive expense. The construction of the caisson is costly, and sinking it is costly. Elaborate and expensive plants have to be maintained for it, and in some cases the extreme cost could be obviated by the substitution of piers, sunk by open coffer-dam work and by other methods.

The use of improved steel sheet-piling will go a great way toward solving that problem, and reducing the cost of many difficult substructures. Up to the present time, steel sheet-piling, although it has been used in large quantities, has been of very heavy weight, has not afforded absolutely water-tight joints, has been subject to difficulty in driving, has cost from 75 cents to $1 per square foot and upward, and, therefore, has often been "thrown out of court," not only on account of its excessive first cost, but on account of uncertainty in driving. Mr. Skinner.

Recent improvements in design and construction have very materially reduced the weight and cost of steel sheet-piling, and they insure absolutely water-tight joints, without caulking or silting. Such piling can also be driven with perfect protection, so that, no matter whether the driving be hard or easy, or whether or not there are moderate obstructions, the engagement of the piles and their perfect position can be assured when installed and in service. The piles, with ¼-in. webs, can bear a load of 100 lb. per sq. ft. with supports 6 ft. apart, and may be of any desired width, up to the limits of ordinary independent driving, say 24 in.

The most important elements of cost for steel sheet-piles are the joints, the spacing of which has heretofore been determined by the widths of standard rolled sections, and the practicable dimensions for driving. Both these considerations have been eliminated in recent improvements, by which steel sheeting can be installed, before excavation, in units of any width desired, thus reducing its cost almost one-half, without materially increasing the cost of driving, and by a method applicable in hard, soft, wet, or dry, soils. It can be used wherever it is possible to drive any other kind of sheet-piling, and provides for sheeting of any dimensions and any degree of strength and stiffness.

The significance of this fact is very important and far reaching; it means that, not only stiff sheeting can be installed more perfectly and cheaply than before, but that, where great permanent strength is not required, a continuous and perfectly water-tight steel surface, either temporary or permanent, can be installed, which has a minimum weight and, even with present facilities for manufacture—which can easily be materially improved—will cost less than wooden sheet-piling. This piling can be designed to have exactly the strength required, without excess of metal. If very thin webs suffice, they can be strengthened temporarily, to receive the reaction of braces and distribute pressure, by a facing of loose planks placed as the excavation progresses, and afterward removed, thus leaving only the minimum structure permanently engaged after the work is completed.

There is another point which has not been fully considered: In many cases it might be quite advantageous and entirely practicable to eliminate costly pneumatic-caisson work by enclosing one, two, or even three or four, sides of the building site by a continuous and perfect wall of steel sheet-piling. Suppose that 200 ft. of such a wall are necessary, and that it is required to be 30 ft. deep. It could be installed at a total cost of from $4 000 to $5 000, exclusive of salvage, and in many cases would not only protect the foundations of adjoining

Mr. Skinner. buildings from undermining and settlement—obviating the necessity for costly underpinning, in itself much more expensive than the total cost of the piling—but would also serve as a coffer-dam, excluding a large quantity of ground-water, so that the foundation piers could be constructed in open pits, much more rapidly and cheaply than with pneumatic caissons.

The pneumatic caisson is one of the most indispensable and important appliances for difficult substructure work; splendid courage, skill, and energy have been shown in the developments by which it has been brought to a high state of perfection and simplicity, enabling the erection of structures which would otherwise have been impossible. It will continue to be used advantageously in an increasing number of cases, but there will be other cases where the great cost of either pneumatic or rigid open caissons may be avoided, and perhaps structures built which would otherwise have been considered too costly, by the use of steel sheeting, which, besides the advantages mentioned, can be assembled before driving to form the walls of a complete caisson and driven sectionally in small units where it would be impossible to overcome the friction and resistance for the whole caisson at once, and where the driving effort can be concentrated on a small area of the structure, thus practically multiplying it greatly and also allowing for adjustment to conform to the rock surface and obstructions.

Mr. Thomson. T. Kennard Thomson, M. Am. Soc. C. E.—The author is to be congratulated on giving such a clear demonstration of the art of caisson design as it existed in the early part of 1907.

The design of pneumatic caissons has been completely revolutionized about four or five times in the last ten years, and the result is that what was the best two years ago was out of date last year, and last year's best is already out of date, notwithstanding the fact that there is no construction going on at present.

In trying to originate or improve, it is very common to revert unconsciously to original or antiquated types, and, after using these designs several times, the same objections which caused the original to be abandoned become apparent again. For example, the early caissons were built without coffer-dams on top, as the masonry piers were generally started on top of the massive wooden structures. To do this without any coffer-dam, of course, means that the masonry must be built as fast as the caisson sinks, as it always has to be kept above water. This means, first, delay in waiting for the masonry, and secondly, that the weight of masonry is often greater than required for sinking, which is dangerous.

It also means that the friction on the sides of the masonry from the surrounding material is often sufficient to break the fresh masonry joints; and again, stone masonry causes greater frictional resistance than plain greased boards. These are undoubtedly the reasons for giving up the attempt to build caissons without the use of coffer-dams.

The first pneumatic caissons under a sky-scraper were sunk for the Manhattan Life Building, at 66 Broadway, New York City, as Mr. Francis H. Kimball, the architect, was far-sighted enough to insist on a rock foundation for his building, and, though he was very severely criticised at the time, most people have become convinced of his good judgment, for pneumatic caissons are the only means by which sky-scraper foundations can be carried to good hardpan or bed-rock, in lower New York City, without serious danger to surrounding property. The only material above the hardpan is quicksand which runs like water, and anybody who attempts to hold back from 25 to 80 ft. of this stuff by sheeting, etc., will find the results disastrous to the pockets of the owners of the property. In fact, a great many thousands of dollars have already been lost through the experiments of novices, both by the novices themselves and their clients. Mr. Thomson.

The caissons of the Manhattan Life Building were built of iron plates and angles, without coffer-dams, the brick piers being started on top of the deck.

Brick masonry has also been found to open at the joints, and allow the caissons and coffer-dams to separate. Brickwork and stone masonry, good in their day, have now, it is to be hoped, been entirely displaced by concrete for caisson work, both above and below the deck. Wet concrete requires little or no ramming or inspection after mixing, while masonry of stone or brick cannot be given sufficient inspection.

As timber costs much more than concrete, the less timber used, of course, seemed to show the greater saving, and it was thought feasible to replace the timber coffer-dams by temporary forms, as described by the author.

It was soon found, however, that the apparent saving was very deceptive, for, in the first place, the concrete has to be allowed from 24 to 48 hours to set, and the result is that the sinking has to be interrupted several times, for several days, the men being put to work elsewhere. To change the men from one caisson to another in the middle of a shift is, of course, a waste of time; and to keep the air pressure on a caisson about twice as long as would be necessary with coffer-dams is obviously expensive, as well as dangerous, for when no one is working in the air chamber the gauge tender is likely to, and often has, become careless, allowing the pressure to increase or decrease, with disastrous results, especially to adjoining property.

Another reason is that, for economical sinking, it is necessary that the penetration should be gradual but continuous; plunging a couple of feet at a time and then waiting for several hours gives the quicksand a chance to flow against the sides of the caisson and then adhere to it. Naturally, if the caisson stands still for a couple of days or longer, this trouble is much increased, and it is quite difficult to start

Mr. Thomson. it again, and often requires a great deal of additional weight, usually in the form of pig iron, thus increasing the expense.

A minor objection to the "forms" is that they are usually connected by iron angles bolted together, and in New York City the Unions insist upon this being done by the Iron Union, thus making it necessary to keep a high-priced gang for unskilled work. Therefore, the probabilities are that timber coffer-dams, from the top of the caisson to the surface of the ground, will be retained.

The omission of the timber or steel roof is often a very decided saving; but in some cases it would not pay and in others it would be dangerous, especially where it would make the caisson too heavy to be handled to advantage. If the caisson with its load is too light, of course, it will not go down, whereas if it is too heavy it will penetrate more rapidly than desired, and, as a matter of fact, the cutting edge has frequently been forced into the ground until the entire working chamber has been filled with earth, etc. This, of course, would kill any of the men who were unable to escape, and would make the resumption of work quite tedious, as it would be necessary to remove enough material to make room for the men and the bucket.

As for the steel shafting, which is expensive, it is often economical to omit or remove this; but here, again, the question of extra labor and time will often overcome the advantages, and, in the case of a 5-ft. circular caisson with a 3-ft. shaft, it can easily be understood that the omission of the steel shaft would be attended with considerable risk, as experience has proved, where the caisson and concrete have broken apart—an accident almost impossible to rectify when the caisson is from 20 to 40 ft. in the ground.

There have been several cases where this has occurred; and what assurance is there that similar accidents have not occurred without being detected?

In short, even the most experienced men are often compelled to learn by experiment, while novices in foundation work must cultivate a very profound respect for earth and water pressures, or pay the penalty, which on one contract known to the speaker amounted to about $75 000.

Mr. Brown. LOUIS L. BROWN, M. AM. SOC. C. E.—There is little than can be added to this paper, especially considering the remarks made by Mr. F. W. Skinner and Mr. T. K. Thomson, both of whom are very conversant with the subject.

From the standpoint of one whose experience has been closely connected with the actual execution of work of this class, the fact of the recent improvements in methods cannot be given too much importance.

While it is true, as stated by Mr. Thomson, that there have been a great many improvements and changes in a very short period of time, and that sometimes new ideas are tried and found to be unsatisfactory, still, to make sure on such points, actual trials are necessary.

One company in New York City has tried and is continually trying new methods, for the purpose of cheapening and bettering caisson construction, and out of the many trials and investigations which this company has made have come some of the recent improvements such as Mr. Usina has illustrated. Mr. Brown.

Not only have the cost of the work and the excellence of construction been considered, but also the safety of the workers. There is now in use a shaft which, besides having the features described by Mr. Usina, is of oval cross-section, with space at one end for the passage of the bucket and at the other for a ladderway to permit the men to pass in and out without the risk of being hit by a descending bucket. This has averted many accidents.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

CHARLES HAYNES HASWELL, Hon. M. Am. Soc. C. E.*

DIED MAY 12TH, 1907.

A life of remarkable professional activity, characterized by conspicuous public service, came to its close when, on May 12th, 1907, Charles Haynes Haswell died at his home, 324 West 78th Street, in New York City.

Mr. Haswell was undoubtedly the oldest civil engineer in the world, and had he lived ten days longer, he would have entered upon his 99th year. He was elected a Member of the American Society of Civil Engineers on January 29th, 1868, and on May 12th, 1905, he was made an Honorary Member, being one of the forty men upon whom this distinction has been conferred since the organization of the Society.

A son of Charles Haswell, a native of Dublin, who was in the British diplomatic service, and of Dorthea Haynes, a member of a prominent family in the Barbadoes, he was born on May 22d, 1809, in a house which is still standing on North Moore Street, in New York City. His education was obtained in the best New York schools of the time, and was liberal or classical in its character, as no school of applied science had yet been established in the United States.

At the age of nineteen he entered the service of James P. Allaire, who was the owner of what was then the greatest steam engine works in the United States. By close application, he acquired a practical and thorough knowledge of mechanical and marine engineering, and his excellent work coming to the attention of the United States Navy Department, he was, in 1835, employed to prepare designs for the machinery of the United States Steam Frigate *Fulton*. He had the satisfaction of superintending the construction of the engines and boilers of this vessel, as Chief Engineer, under a commission signed by President Jackson. Afterward, he designed or superintended the building of the war-ships *Missouri, Mississippi, Michigan* and *Allegheny,* and a number of revenue cutters.

In 1843 Mr. Haswell was appointed the first Engineer-in-Chief of the United States Navy, his administration of which office was characterized by a devotion to the highest professional ideals, absolute integrity, and rare efficiency. His uncompromising fidelity to duty, as interpreted by his ideals and standards, was not always appreciated by those with whom he had official relations, and in 1851 he resigned from the

* Memoir prepared by Nelson P. Lewis, M. Am. Soc. C. E.

naval service to engage in private practice in New York City, as a civil and marine engineer. The modern steam yacht is said to have been created by Mr. Haswell, as the *Sweetheart*, which was probably the first vessel of this type, was designed and built by him in Brooklyn, some time before 1840.

While best known through his connection with steam and marine engineering, his work covered nearly all branches of civil and mechanical engineering, and as the author of the "Engineers and Mechanics Pocket-Book," which bears his name, he was known throughout the world. This book was a standard work of its kind for more than sixty years, and passed through seventy or more editions. He was working at his desk upon material for a new edition, when, rising from his chair, he fell and sustained injuries which resulted in his death the following day.

During the Civil War, Mr. Haswell was an enthusiastic supporter of the Union cause. Not only did he go to the front as the representative of a "Committee of Citizens of New York" and direct the disbursement of funds raised by the Committee, a mission requiring much tact and discretion, but he was in active service under General Burnside, who recognized his excellent work in his reports to the Secretary of War.

His interest in the affairs of the city which was his birthplace and home was always keen and unselfish. From 1855 to 1858 he was a member of the City Board of Councilmen, and, during his last year of service, he presided over that body. He served as member of a number of important commissions, and was one of the Trustees of the Brooklyn Bridge. From 1898 until the time of his death he was Consulting Engineer to the Board of Public Improvements and the Board of Estimate and Apportionment. He personally made the plans for and supervised the installation of the heating and power plants for the public institutions on Hart's Island, and prepared plans for the enlargement and improvement of Riker's Island. Until within a few months of his death, he was regular in his attendance at his office, where a large part of his time was spent over the mahogany drawing board, concerning which Mr. Haswell wrote in 1904:

"It has been in use 53 years without requiring to be trued. On it was executed the feat that has become historical both here and in Europe, that of the delineation of the entire working drawings of the engines and boilers of the U. S. Steamer *Powhattan,* cylinders 70 inches by 10 feet stroke of piston, the demands upon my time not admitting of the delay of making a general drawing before furnishing those of the detailed parts."

In the summer of 1904, then in his 96th year, he was retained as an expert to examine and report upon a boiler plant in Chicago, where he spent a week in making tests and preparing his report.

Appreciation of Mr. Haswell's professional work was not confined to his own country. More than half a century ago the Czar of Russia sent him a diamond ring, accompanied by an expression of his thanks for services rendered to the Imperial Government in sending to it a number of plans and drawings. The engineers and naval architects of Great Britain have frequently indicated their high regard for and deep obligations to him, and during the visit of members of the Institution of Civil Engineers to America in 1904, he was the recipient of conspicuous attention from them and their President, Sir William H. White. Those who went to West Point with the visiting engineers will recall the fact that he walked unaided, down the long line of cadets, with the reviewing officers and guests, and his tall, erect figure was the most conspicuous in the party.

He contributed several papers and discussions to the *Transactions* of this Society and to the *Minutes of Proceedings* of the Institution of Civil Engineers. In 1897 he published his "Reminiscences of an Octogenarian," a book which, while lacking continuity of narrative, gives some admirable and interesting sketches of New York City between the years 1816 and 1860, and affords evidence of the refined tastes and admirable public spirit of the author.

Owing to Mr. Haswell's great modesty, he rarely spoke of his personal achievements or of incidents in which he figured, and it was difficult to realize the important service which he had rendered to his profession and to his country. On the rare occasions when he would indulge in conversational reminiscences, he was delightful. His early education, as already noted, was of the liberal sort, and there was a refinement in his manner and conversation which showed the influence of that training. He was familiar with the best literature, and his Latin quotations, while used without a suggestion of pedantry, frequently gave force to his illustrations and charm to his conversation. His bearing toward his associates and toward those who were many years his junior was characterized by a gentleness and uniform courtesy which made him a delightful companion and an always welcome visitor, while his tall, slender figure made him conspicuous in any assembly.

With his keen appreciation of the dignity of his profession, his high sense of personal honor, and his rare consideration for the feelings of others, he was an admirable example of the old-school, courtly gentleman of the type which has become all too rare.

Mr. Haswell, in addition to his membership in this Society, was also a member of the following technical and social organizations: The Institution of Civil Engineers; The Institution of Naval Engineers of Great Britain; The Naval Engineers of the United States; The Municipal Engineers of The City of New York; The American Institute of Architects; The New York Academy of Science; The New York Microscopical Society; The Society of Authors; The Engineers' Club of New York; The Engineers' Club of Philadelphia; and the Union Club of New York.

JAMES DUN, M. Am. Soc. C. E.*

DIED FEBRUARY 23D, 1908.

James Dun was born on September 8th, 1844, in Chillicothe, Ohio, and there his early education was obtained. After being graduated from the Chillicothe Central High School, he attended a private school at St. Catherines, Ontario, Canada. Later, he returned to Ohio and received his finishing education at Miami University, Oxford, Ohio.

Mr. Dun began his professional career in 1866, as a member of an engineering corps working near Indianapolis, Indiana; later, he was Instrumentman on the survey for the old Louisiana and Missouri River Railway, between Louisiana and Cedar City, Missouri, now a part of the Chicago and Alton System.

In 1867 he was appointed Assistant Engineer of the Atlantic and Pacific Railway, under Mr. Thomas McKissock, Chief Engineer. This road is now a part of the Frisco System.

From 1871 to 1873 Mr. Dun was Assistant Engineer of the Missouri Pacific Railway, under Mr. James W. Way, Chief Engineer. From 1874 to 1877 he was Chief Engineer of the Union Depot Company, and built its yards, and freight and passenger station in the vicinity of Twelfth Street, St. Louis, Missouri. In 1877 he was appointed Superintendent of Bridges and Buildings of the St. Louis and San Francisco Railway Company, and in 1878 was appointed Chief Engineer of the same system, also filling, for a part of the time, the position of Acting General Manager, during the last illness of Mr. C. W. Rogers, Vice-President and General Manager.

In 1890 Mr. Dun was appointed Chief Engineer of the Atchison, Topeka and Santa Fe Railway, and in 1900 was appointed Chief Engineer of the Santa Fe System. In 1906 he was appointed Consulting Engineer of the same system, which position he held at the time of his death, which occurred on February 23d, 1908, at St. Augustine, Florida.

Mr. Dun was elected a Member of this Society on June 7th, 1876. He was also a member of various other technical societies, including the Engineers Club of St. Louis, and the Western Society of Civil Engineers, of Chicago.

His professional reputation was international, and the Frisco and Santa Fe Systems show to-day the characteristics of his work. The writer sustained social and official relations with Mr. Dun from 1869—not continuously, but nearly so—and in all those years has never seen Mr. Dun's enemy nor heard an unkind criticism.

Broad-gauged, liberal-minded, and with spotless integrity, he recognized his work and performed it to the letter. Accurate and resource-

* Memoir prepared by J. F. Hinckley, M. Am. Soc. C. E.

ful, with a keen mind for details, and a fund of information acquired by long and varied experience, he was especially well qualified to sustain the confidential relations with his superior officers which he held for so many years before his death. Loyal, kind and generous, he was a most charming companion.

Born a gentleman, he developed into the highest type of manhood. Full of sympathy for all with whom he came in contact, he was never too busy to give time to the trouble of his friends, nor counsel to the young graduate who was looking for an opportunity.

One of his friends—a prominent railroad man who had known him intimately for more than thirty years—writes as follows:

"I never knew him to do a wrong. His integrity was like the sun's rays; it came swift from a soul of fire. Nothing deflected him from the straight course. * * * He sympathized with all men in trouble, and appreciated the infirmities of human nature; but he never could understand why men were dishonest. The men in this world who have never been swerved from the right by either passion or covetousness have been few: James Dun is the only one I ever knew."

The high esteem in which he was held by his associates cannot better be summed up than in the words of an old roadmaster who worked under his direction for many years; upon being told of Mr. Dun's death, he remarked: "There may have been a better man, but I never met him."