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WATER-SUPPLY ENGINEERING.

THE DESIGNING, CONSTRUCTION, AND MAINTENANCE OF WATER-SUPPLY SYSTEMS, BOTH CITY AND IRRIGATION.

BY

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FIRST EDITION.

FIRST THOUSAND.

NEW YORK:
JOHN WILEY & SONS.
LONDON: CHAPMAN & HALL, LIMITED.
1900.

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ROBERT DRUMMOND, PRINTER, NEW YORK.

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WATER-SUPPLY ENGINEERING.

PART I. DESIGNING.

CHAPTER I.

SYNOPSIS.

ARTICLE 1. SOURCES OF SUPPLY.

ALL mankind, whether wandering savage, isolated pioneer, or dweller in a crowded city, must have water for drinking, needs it for cleansing himself and his belongings, and for irrigation; and under many circumstances he uses it for power or in manufacturing, as well as for extinguishing fires, sprinkling lawns and streets, flushing sewers, and many other purposes. The water for all these uses can have but one first source—the moisture in the atmosphere. This generally becomes available as rain (or snow), but dew is in some cases an important consideration.

Water falling as rain or snow may be caught before reaching the ground and stored in basins or cisterns hewn from the rock, dug in the soil, or in the shape of tanks of wood or iron. Or it may be taken from rivers or smaller streams direct, or these may be intercepted and stored in large artificial reservoirs, or in natural reservoirs, i.e., lakes. Or that which soaks into the ground may be obtained by wells, shallow or deep, dug or driven; or at its emergence in the form of springs.

Art. 2. Quality of Water.

Water does not exist in nature chemically pure—(H_2O)—but, owing to its almost universal solvent powers, it contains many foreign matters in solution; and, when flowing in streams or lakes, in suspension also. Many of these matters in solution are harmless, some are beneficial, and a few are injurious. Rain-water washes impurities from the air, river-water receives much organic and some mineral matter from the surface flow or "run-off" of fallen rain, and ground-waters absorb much mineral and some organic matter from the strata through which they pass. In only exceptional cases are these matters injurious to man before the country has been cleared and occupied by him. But the enormous amounts of waste matters from cities overtax Nature's arrangements for their immediate destruction, and rain-, surface-, and ground-waters are all contaminated by animal and manufacturing wastes. It is probable that in very few cases is water from air, stream, or well so impure as to be injurious when applied to the skin, but only when taken into the stomach. It may, however, contain so much matter in solution and suspension as to unfit it for many manufacturing purposes, such as paper-making and the textile industries; and for use in these and in washing kitchen utensils, as well as to render it potable, it must be uncontaminated by objectionable matters or must be purified of them. For other purposes, such as extinguishing fires, sprinkling streets, flushing sewers, etc., almost any water is adapted which does not contain large amounts of suspended matter.

Until within a very few years the principal aim of most American communities has been to obtain quantity of water, with little regard to its quality; and they have so well succeeded that we are supplying on an average five or six times as much water to each citizen as do European cities. However, compilation of mortuary statistics by Health Boards of different states and cities, and comparisons of these with each other and with those of foreign cities, together with a more definite and wide-spread information on the causation of disease, has led to a realization of the importance of pure water, food, and air.

Largely for the purpose of emphasizing the importance of this phase of the subject the *quality* of water-supplies has been the subject first treated of. While this is more a chemical and bacteriological than a strictly engineering subject, no engineer should attempt to select a supply without calling these branches of science to his aid, and should be able to understand and weigh the information thus obtained. In fact, many of the problems not only of design but of maintenance also will require for their proper solution more or less intimate knowledge of chemistry and biology.

Art. 3. Elements of a System.

Before considering in detail the design of a system it will be well to understand what elements go to make up the complex whole. In general the water which falls is to be rendered available for whatever purposes man may desire it. The enlarging and walling up of a spring may be considered as the first reservoir construction. The roof from which the water-butt receives its supply is an elementary catchment area. The engineering features of these and of dug wells for private supply are extremely simple; but public supplies become more and more complex as the number of persons and purposes to be served increase; and the supply of a modern city with the best water in the best way calls for a high character of engineering skill.

A supply being found which is satisfactory in quality and quantity—or the nearest to this obtainable—it must be supplied to each consumer continuously (although European cities did for years supply water but a few hours daily), at a rate always sufficient for his hourly needs, and of the necessary purity. This may call for storage to tide over seasons of the year when the natural supply is deficient; or for purification to improve the quality. If the source is not higher than all points where it is to be used, the water must be raised to these. It must be conducted from its source to each consumer, and the conduits provided with the proper contrivances for supplying all public needs, such as fire-service, street-sprinkling, and the like. Since water is an absolute daily necessity, and in a crowded city the public supply is generally the only one available, there must be no possibility of an interruption of the supply from any cause whatsoever. The quality also must be preserved, and the water as delivered be both wholesome and unobjectionable to any of the senses. In some instances where such water cannot be obtained in sufficient quantity for all purposes, a secondary supply of less pure water is used for extinguishing fires, street-sprinkling, and sewer-flushing.

Such a system having been provided, it must be so used and maintained as to deteriorate as little as possible both in the quality and quantity of water furnished and in the efficiency of the plant; and, whether a private or public enterprise, should be conducted on sound business principles.

The subject of water-supply can best be treated under the general heads of Designing, Construction, and Maintenance. Each of these considers the water to be supplied, its quality and quantity; the means for obtaining it or making it always available—wells, dams, reservoirs, pumps, etc., valve-gates, fire-hydrants, and other contrivances; and of Maintenance, the conduct of the works from a business point of view is an important branch.

CHAPTER II.

REQUISITES OF A SUPPLY. QUALITY.

ART. 4. VALUE OF WATER ANALYSES.

SINCE the quality of a water-supply is of the greatest importance to all consumers, any water proposed for use should be most thoroughly investigated before being finally adopted for a supply. And the engineer, as he generally must decide concerning any given supply, should be capable of doing so intelligently.

While it is not necessary and is seldom possible that a water-works engineer should be a chemist or bacteriologist, he should understand the principles and aims of analyses and be able to interpret these when made by experts. And first he should realize that, in the present state of these sciences, neither chemistry nor biology can, alone or together, decide finally as to whether a water is or is not injurious if used as a beverage. On the other hand, in very few cases should such a decision be made without their aid; and they alone are generally sufficient to determine the fitness of a supply for use in any given manufacturing process, or for irrigation.

The characteristics to be considered are: impurities (injurious or otherwise), appearance, taste, color, and odor.

The impurities existing in water are either organic or mineral, in either solution or suspension; the organic may be living or dead organic matter, and may be animal or vegetable. Mineral matter can generally be determined quite accurately as to both kind and quantity. The organic can be quite accurately determined as to quantity either in solution or suspension; the living organisms can be classified by the microscopist with considerable minuteness and certainty, although much yet remains to be learned in this line; but of dead and putrescible organic matter, and the products of decomposition of this, little can be learned through the microscope, and chemistry can but discover the elements or mineral compounds which enter into the composition of such matter. Thus the chemist can determine the amount of nitrogen in a given water, and, knowing that sewage contains a large amount of nitrogen, may suspect sewage contamination. But he cannot say that this mineral was not washed out of the air by rain, or from peat deposits in the earth by ground-water.

In almost no case is the actual matter discovered by the chemist injurious in quantity. In the average sewage the total impurity is less than one one-thousandth the amount of the pure water which carries it; and a river-water which the senses would not condemn would generally contain not more than $\frac{1}{50000}$ of impurities. It takes about 1000 drops of water to make one glassful ($\frac{1}{2}$ pint), so that the amount of impurity, harmless and otherwise, in a glass of such water would be but $\frac{1}{50}$ the size of a drop. It is not therefore the matter found, but the inference from its presence, which is important.

Similarly the bacteriologist may determine that a cubic centimeter of water contains a certain number of bacteria, but in few if any cases have pathogenic bacteria been recognized with certainty in drinking-water; and the number of all kinds present is but an indication of the amount of organic impurity, which may all be harmless. "A water analysis . . . is really not an analysis at all, properly so called, but is a series of experiments undertaken with a view to assist the judgment in determining the potability of the supply. The methods of conducting these experiments are largely influenced by the individual preference of the analyst, and are far from being uniform or always capable of comparison, thus often introducing elements of confusion where two or more chemists are employed to analyze the same water. Some of the substances reported—'albuminoid ammonia,' for instance —do not exist ready formed in the water at all, and are but the imperfect experimental measures of the objectionable organic constituents, which our present lack of knowledge prevents our estimating directly.

"Thus the numerical results of a water-analysis are not only unintelligible to the general public, but are not always capable of interpretation by a chemist, unless he be acquainted with the surroundings of the spot whence the sample was drawn, and be posted as to the analytical methods employed." (Mason's "Water-supply.")

Because the chemist cannot certainly find specific injurious matter, except by inference, however, is no reason for neglecting or rejecting his services; but these should be used for all that they are worth—and this may be considerable—and interpreted in comparison with all other available data.

ART. 5. INTERPRETATION OF ANALYSES.

"Substances found in water may be classified as follows:

- "1. Suspended Matter
  - Inorganic or Mineral
  - Organic
    - Animal
    - Vegetable
- "2. Dissolved Matter
  - Gaseous
  - Solid
    - Inorganic
    - Organic
      - Animal
      - Vegetable."

(Nichols' "Water-supply, Chemical and Sanitary.")

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Much of the suspended matter may become dissolved if time be allowed; organic may be resolved into its inorganic elements; and these may alter their combinations.

Inorganic matter is seldom present in such quantities as to render injurious a water otherwise suitable for drinking; excepting the salt found in the ocean and in certain underground waters. Lime, salt, and iron are the three mineral substances most commonly found. Whatever the mineral, it is generally stated in the analysis directly in the number of parts found per 70,000 (grains per imperial gallon); per 58,372 (grains per U. S. gallon); per 10,000, 100,000, or 1,000,000 parts by weight; the last being equivalent to milligrams per liter when this amount of the water in question weighs 1000 grams. "Parts per 100,000" is most generally used in this country for both organic and inorganic matter, and will be the unit adopted in this work except where otherwise stated.

Organic matter in water appears as living organisms, animal or vegetable; products of organic life, as albumen, urea, tissue, etc., dissolved or suspended; and products of decomposition of organic matter, including mineral matters, as salts of ammonia and carbonic and nitric acids. Carbon and nitrogen oscillate between the organic and the inorganic state. Organic matter cannot be determined directly by chemical analyses, but only by indirect methods; the nitrogen compounds being generally taken as an index of the amount present, since "it is the nitrogenous organic matter which has the greatest sanitary importance, owing not only to the facility with which it undergoes decomposition, but also to the fact that nitrogen is an essential element in all living matter. Analytical processes of great accuracy enable us to determine nitrogen in four forms; namely, as organic nitrogen ('albuminoid ammonia'), as ammonia, as nitrous acid, and as nitric acid." (Mass. State Board of Health.)

Organic matter consists chiefly of carbon, hydrogen, nitrogen, and oxygen. When its life departs it begins a decomposition, first by oxidation of the carbon, which leaves the nitrogen combined with hydrogen in the form of ammonia; and subsequently by the oxidation of the ammonia to nitric acid (NH_3 to HNO_3), which generally combines with some mineral base in the water. The ammonia (free ammonia) discovered by chemical analysis indicates that organic matter once present has begun rapid decomposition. That not yet decomposed, whether living or dead, is changed by the addition of chemicals to ammonia and given as "albuminoid ammonia"; while ammonia which has been oxidized is recorded as "nitrites" or "nitrates," according as nitrous or nitric acid has been formed. Albuminoid ammonia is about $\frac{14}{17}$ nitrogen, and the amount of this obtained by the analysis is about one half the organic nitrogen present. About $\frac{4}{25}$ of animal matter and a much smaller part of vegetable matter are nitrogen; algæ containing about $\frac{1}{15}$ nitrogen. Hence the albuminoid ammonia, if derived wholly from algæ, times $\frac{14}{17} \times 2 \times 15$ would give approximately the amount of algæ present.

The absorption of oxygen in the formation of nitrous and nitric acid generally uses up much of the free oxygen in the water, and hence the absence of oxygen in the water is sometimes considered as an index of the organic matter present. The amount of oxygen absorbed from a permanganate or other oxidizing agent by the water to replace that used up is generally stated as "oxygen absorbed."

When saturated with oxygen, water at 32° Fahr. contains 1.47 parts of this by weight per 100,000; and at 80°, 0.81 parts.

One other determination is generally made in investigating organic pollution—the chlorine; which, since it is found in all urine, is taken as an indication of sewage pollution, if existing in quantities greater than normal. Chlorine cannot be removed from water by any method of filtration or oxidation, and hence any increase in the amount present in a given water is an indication that such water has received the addition of a less or greater amount of chlorine, or has been reduced in amount by evaporation.

High ammonia, nitrites, and chlorine together form an almost sure indication of sewage pollution. The excreta from each person have been found to contribute daily to sewage an average of .015 lbs. of free ammonia, .003 lbs. of albuminoid ammonia, .218 lbs. of dissolved solids, and .042 lbs. of chlorine. These amounts will of course vary somewhat with the age, sex, and food-matter of each contributor; but by their use an approximate idea can be formed of the pollution of a stream of given flow which is caused by a given number of sewage contributors.

By the microscope living organic matter is investigated, and in certain cases the dead also. But for studying dissolved or disintegrated tissue or other organic matter the microscope is not adapted. There are very few animal or vegetable organisms occurring in water which are injurious to human beings, except when taken in such numbers as to render the water repulsive. They may, however, give rise to unpleasant tastes in the water, and for this reason their presence in considerable numbers should not be overlooked. There is a class of organic matter, however, called bacteria, certain of which are thought to be the causes of several diseases, the most important being typhoid fever and cholera; although malaria and some other common diseases are by many attributed to water-borne bacteria. A bacterium is about $\frac{1}{100000}$ to $\frac{1}{10000}$ of an inch in diameter, and one cubic centimeter may contain many thousands. Since but one drop of water can be examined at a time, the absence from any given drop of dangerous bacteria is no indication that some other of the million-and-a-half drops supplied daily on an average to each consumer may not contain such. Moreover, in the present state of bacteriology we cannot certainly identify the particular bacterium of typhoid or of cholera, and about the only determination which can be made is a quantitative one. Of 100,000 bacteria in a cubic centimeter of water, not one may be morbific; but the presence of this number indicates organic matter which may furnish food for, or be otherwise indicative of, the presence of such bacteria; and hence water high in bacteria is generally looked upon with suspicion, particularly if known to receive sewage, which may contain the dejecta of typhoid and cholera patients.

ART. 6. INORGANIC MATTER.

Iron, lime, magnesia, chlorine (usually as chloride of sodium or common salt), sulphuric acid (generally as sulphate of lime, magnesia, sodium, or potassium), carbonic acid (both free and as carbonate of lime or of magnesia), are the mineral matters most commonly found in solution in natural waters. In suspension may be found clay, fine sand, and other soil constituents, varying in size from $\frac{1}{100000}$ of an inch to large sand or gravel.

Iron is not unhealthful when occurring in drinking-water in relatively large amounts; but since one part to 200,000 of water is appreciable to the taste, more than this should not be permitted in a public supply; more especially since less than this amount in washing water will discolor clothes, and will cause a brown sediment when left standing in the open air. Iron is derived from iron deposits in the soil and deeper strata, dissolved out by surface- and ground-waters, and from peaty and other vegetable matter. Most vegetable matter contains more or less iron,—leaves from .01% to .06%, grass about .006%, and black peaty muck 0.9% (Boston Water Board experiments), and much of this is dissolvable by water, and probably is largely responsible for the color in swamp and peaty waters. Such discolored peaty waters are often wholesome, and some of the purest water-supplies in southern New Jersey are highly colored. Peaty waters are not always harmless, however, but have been known to cause enteric diseases, probably because of too large quantities of organic matter.

Lime and magnesia occur in water as carbonates or sulphates, or other soluble salts. The carbonates cause a "temporary hardness," the other salts "permanent hardness." Carbonates are caused by the carbonic acid contained in a water dissolving lime or magnesium from the rock it passes over or through. When water is boiled, carbonic acid is expelled and the lime or magnesium is left in suspension, and forms a deposit, in boilers a "scale," the water being thus rendered "soft." In permanently hard water the salts are dissolved by the water itself and do not separate out by boiling. The same water may be both temporarily and permanently hard. Such hardness as is ordinarily found in water is not unhealthy, although slight intestinal trouble may be caused by the change from soft water to hard, but also from hard to soft. It is not probable that hard water causes deposits of calcareous matter in the bladder. The serious objection to hard water is the formation of scale in boilers and the waste of soap resulting from its use. In general it may be said that each grain of carbonate of lime per gallon of water compels the use of two additional ounces of soap per 100 gallons of water. The expense to boiler-users is even greater. The sulphate scale, which is deposited at about 260° Fahr., is much more injurious than the carbonate, because it forms a denser and harder incrustation. The expense of cleaning and repairing boilers, due to this scale, for a mileage of 362,756 on the Nebraska Division of the U. P. R. R. in 1897–8 was $2860, or 8 cents per mile, in addition to which the total mileage of each engine was reduced probably one half.

Sulphates of sodium and potassium occur in small quantities in some waters and cause foaming in boilers, but no deposits. Sulphate of magnesium forms a boiler-scale, and the sulphuric acid is set free as a corrosive element. Chlorine generally occurs in water in the form of common salt ($NaCl$). Besides the urine of man and other animals, the sources of most chlorine in water are the ocean, from which it is carried by evaporation and held in solution by the rain; and salt deposits on the surface, as on the western desert, or in the ground, as in western New York, Ohio, and Kentucky. Rain-water in certain parts of Massachusetts contains 2.5 parts of chlorine per 100,000; and 7 parts have been found in rain-water in England. A well in St. Augustine, Fla., contains 196 parts per 100,000 of salt, and those in Onondaga County, N. Y., contain about 15,000 parts per 100,000; while the Great Salt Lake contains 20,000 parts. The waters last mentioned are not potable, and cannot be used for boilers, or probably for any purpose for which water is required. But the amount contained in any rain-water or surface-water deriving all its salt from the atmosphere is neither injurious nor unpalatable, nor harmful to boilers; although it may prevent its use for certain manufacturing purposes.

If it were not for the fact that salt is contained in sewage, it would not generally be necessary to determine its quantity in water. To decide how much of that found in a given water is due to sewage pollution it is necessary to know the amount present in uncontaminated ("normal") water in the locality in question. This is generally determined by analyzing surface-water (if such be the water whose analysis is desired) taken above any possible sewage pollution, and comparing its chlorine with that of the water in question.

(If it be ground-water little can be learned as to sewage pollution from the chlorine determination.) Several State Boards of Health have made such analyses in all parts of their States, and thus determined the "normal chlorine" for each section. This is found to vary quite closely with the distance from the ocean. Plate I shows, by means of "isochlors," the normal chlorine throughout Massachusetts and Connecticut, as determined by their respective State Boards.

If the water at any place contains more than the normal chlorine it should be looked upon with suspicion until further examination reveals the source of the excess.

Lead and zinc are dissolved from service-pipes by some waters, and in relatively large quantities may cause serious poisoning. In Lowell, Mass., water high in carbonic acid has recently given trouble when delivered through lead or galvanized pipe. 23 parts of lead per 100,000 were found in one sample which flowed through 285 feet of lead service-pipe

ART. 7. ORGANIC MATTER.

Living organic matter, or such as has not begun or does not accompany putrefaction, in the quantities ordinarily found in water is not often injurious. These organisms vary in size from the smallest bacterium to the largest fish. There are perhaps a few vegetable organisms which are poisonous; many which, if taken into the stomach in large quantities, would produce nausea or more serious enteric troubles; but no water which would be used for drinking purposes except under the most extreme compulsion contains a sufficient amount of such matter to be seriously injurious; always excepting certain bacteria. Many organisms both animal and vegetable are beneficial to the water in that they purify it from dead organic matter. The most numerous of

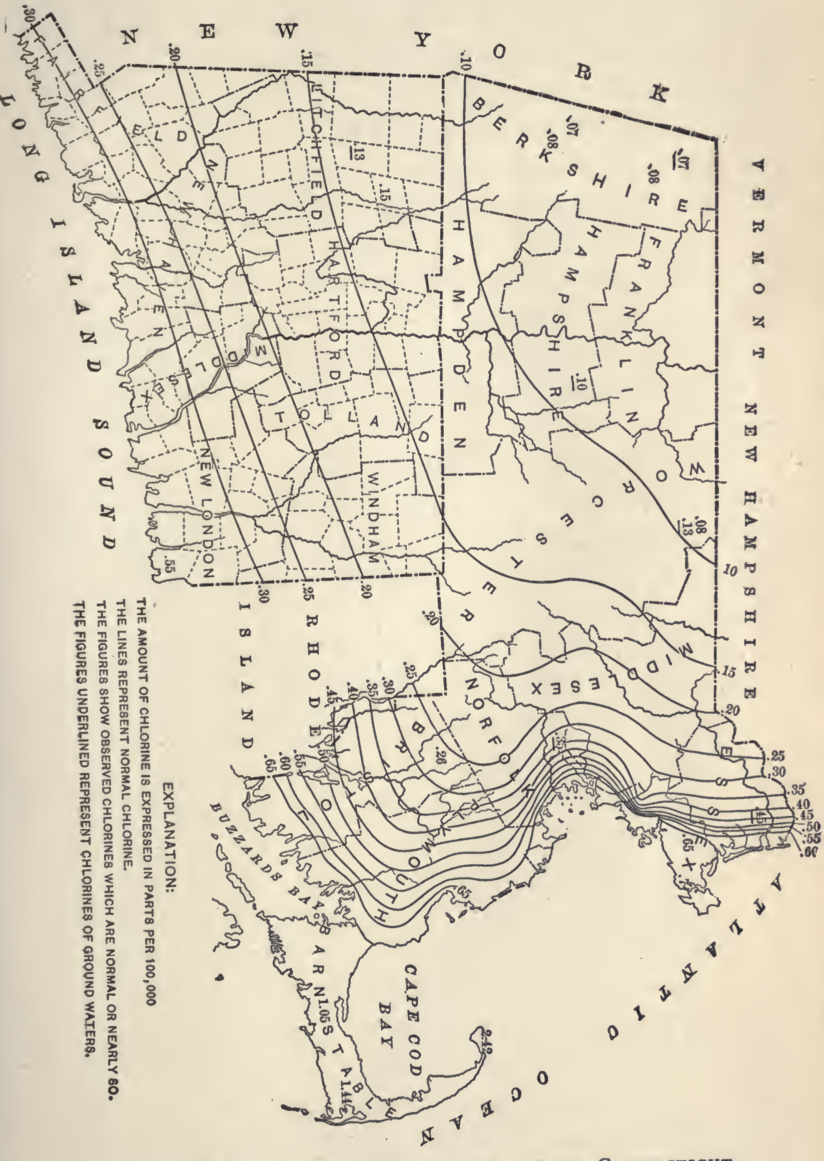

Plate I.—Isochlors of Massachusetts and Connecticut.
(From Reports of the State Boards of Health.)

the smaller vegetable organisms found are algæ, fungi, and bacteria—the two former being generally visible to the naked eye. Of animals other than fishes the protozoa, spongiana, rotifera, entomostraca, and certain mollusks are those most often found. Of these the animals live on vegetable matter and other animals, the vegetable organisms upon "mineralized" organic matter (bacteria, however, seem to possess the animal characteristic of feeding upon organic matter). Both thus assist in purifying the water, not only of organic impurities, but of mineral ones also. Many ground-waters furnish the requisite food, generally in the form of nitrogen, for immense numbers of algæ, of which over 2000 species are known.

These organisms, whether living or dead, seldom give any injurious properties to water; but many species of algæ, including some of the green ones, give off when dead an odor described as "fishy," "cucumber," etc., which is not injurious, but is very unpleasant.

The way in which certain bacteria produce disease is not understood; nor is it known whether a single bacterium may suffice for this purpose, although it is certain that considerable numbers of pathogenic ones may, under some circumstances, be taken into the stomach without apparent effect. We do know that in many instances the contamination of a water-supply with the excreta of typhoid-fever patients has been followed by a great number of cases of such fever among its consumers; and that the only apparent difference always found between the excreta of a typhoid patient and that of any others is the presence of a certain bacterium, the bacillus typhosus, in such excreta. The connection between cholera and the spirillum choleræ Asiaticæ seems to be similarly intimate. The former, however, is the only disease common in this country whose bacterial source seems proven.

About 10% of typhoid cases are fatal. The majority of cases are persons between 18 and 35 years of age. It takes from 9 to 25 days for the disease to make itself apparent—14 or 15 days is the most common period—and the average length of sickness is 40 to 50 days. Typhoid fever occurs most frequently in the autumn; as illustrated by the following table:

DEATHS FROM TYPHOID AND TYPHO-MALARIAL FEVERS; BY MONTHS.

	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
Connecticut; average for eight years. (State Bd. of Health.)........... ...	19.9	13.1	16.4	15.5	16.4	11.9	20.9	43.6	49.2	55.0	36.9	24.9
Ohio; 1892. (State Board of Health.)................	38	27	32	19	27	36	52	71	105	78	86	40

Although a temperature somewhat under 150°, if continued for a few minutes, is fatal to typhoid bacteria, they have been known to live for days and weeks in both pure and impure water and in ice. It thus appears that, once introduced into a river, the danger exists of their being in any given glass of water taken therefrom; although this danger is reduced by reducing the average number of such bacteria to each gallon of the water. A given river-water may contain millions of bacteria per cubic centimeter, of which but a few are pathogenic; and these few cannot often be recognized with certainty; but it is considered probable, from known data, that the smaller the total number of bacteria the smaller the number of pathogenic ones. It is impossible to set a limit to the number of bacteria permissible in drinking-water, but this should be the lowest possible. One hundred per cubic centimeter is the provisional limit set for filtered water by the German Imperial Board of Health in 1892.

Lifeless organic matter in the presence of moisture very quickly begins to decompose, and if oxygen be present in sufficient quantities "mineralizes" into nitrates, this process being assisted by bacteria. These nitrates are of themselves harmless, but unoxidized putrescible or putrescent matter, or the bacteria of putrefaction, are thought to produce enteric diseases which may be of a serious nature if such matter exists in relatively large quantities. 18.7 parts of solids, of which 9.2 were organic, in the water-supply of Long Branch, N. J., in 1887 caused a severe epidemic of diarrhœa.

It is neither possible nor desirable to place a limit to the amount of organic matter permissible in a drinking-water, since it is more the quality than the quantity of such matter which is important. Such matter, when from leaves and other vegetable bodies, may be perfectly harmless while fresh; a little later, while decomposing, it may be injurious; and later still, when oxidized, it may again be harmless. If the albuminoid ammonia in a river below a town be much greater than in the same river above possible sewage contamination, such water should be avoided for its organic matter; and if either this, free ammonia, or nitrogen be present in large quantities, it should be avoided for the bacteria probably accompanying these.

Organic matter seldom renders water objectionable from other than a sanitary point of view, except in the physical condition and character of suspended matter.

Art. 8. Physical Properties.

The matter in suspension in a water may be so considerable as to interfere with its use for manufacturing purposes or boiler supply, and render it unattractive in appearance. When such matter consists of clay, sand, etc., it may in many cases be drunk with impunity, especially when the stomach becomes habituated to it. For household purposes, however, a turbid water is undesirable; although many American cities use water which at times appears tawny yellow or dark gray in a drinking-glass, and contains 1800 parts of matter in suspension per 100,000 for a short time during floods.

While the clay and sand washed into a river are not themselves particularly injurious to health, they serve to call attention to the fact that more objectionable impurities may have been washed from the ground-surface at the same time, such as human excreta, manure, fertilizers, etc. The amount of turbidity is usually expressed as the reciprocal of the greatest distances from the surface of the water, expressed in inches, at which a platinum wire .04 in. thick can be seen. Thus, if such wire just disappears at 5 inches the turbidity is 0.2.

The color of a water tells little of its characteristics, since an unobjectionable one may have considerable color, while another seriously polluted may be clear and sparkling. But color is apt to prejudice consumers against a supply, and this should be taken into consideration. Many waters have a higher color at one time than at another, without any change in the quantity of impurities present. Thus ferrous oxide in solution may, by absorbing oxygen, become insoluble ferric oxide causing a dark color, which again disappears when this settles out or yields part of the oxygen to organic matter in the water. Color is determined by comparison with a fixed arbitrary standard or set of standards, the depth of either water or standard being varied to cause the color of the two to agree. A Nessler scale of 17 standards, recommended by Prof. A. R. Leeds, and the platinum standard of Allen Hazen, are the most commonly used. These standards correspond at 0.40; but 1.50 by the platinum scale equals 2.00 by the Nessler. The greater the number the darker the color.

No standard for odor and taste have been or can well be set. These, like color, may or may not be relative to the impurity of the water, but may easily influence popular opinion, and even produce nausea in sensitive constitutions. A considerable odor is, however, generally indicative of undesirable pollution.

A cool water is pleasanter as a beverage than a warm, and is less liable to foster the growth of vegetable organisms. But this is a question more of the construction of the reservoirs, piping, etc., than of the supply at its source.

Art. 9. Statistics of Disease in Relation to Drinking-water.

The bacillus typhosis cannot be followed in its course from the dejecta of a typhoid patient along a watercourse or in surface-water to the intestines of its victim; but its responsibility for the infection is proved by indirect evidence only. The relation most important to the engineer is that appearing to exist between sewage-polluted water and typhoid fever. There are other mediums of typhoid infection besides drinking-water; but the fact that purification of the water-supply in almost, if not quite, every case greatly reduces the typhoid rate seems to argue that drinking-water is by far the most important one. The two well-known instances of Hamburg and Altona in Germany, and Lawrence in this country, are but pronounced illustrations among a great number of cases of the effect of water on typhoid death-rates. Of Altona and Hamburg—two cities really one but for their distinct government and public works, using the same river for a water-supply, that of Altona, however, being polluted with Hamburg's sewage, but filtered carefully before use—Hamburg in 1892 lost 8605 by cholera, a rate of 1344 per 100,000 inhabitants, while Altona lost but 328, a rate of 230. Prof. Koch says: "Cholera in Hamburg went right up to the boundary of Altona and stopped. In one street, which for a long way forms the boundary, there was cholera on the

Hamburg side, whereas the Altona side was free from it"—undoubtedly because of the different water-supplies.

Lawrence, Mass., has drawn its water-supply from the Merrimac River since 1875. In September, 1893, sand-filtration was begun, but the use of unfiltered water in the mills has been only gradually abandoned since then. The typhoid death-rates before and since 1893 have been as follows (per 100,000):

1890	1891	1892	1893	1894	1895	1896	1897
123	115	95	69	48	31	18.6	16.2

Referring to Plate II, page 24, it is seen that a considerable reduction in the typhoid rate followed the introduction into Newark of the Pequannock water, derived from a sparsely settled watershed, the filthy Passaic having been the former source of supply. The introduction of a purer supply and settling-basins in St. Louis in 1894 resulted in the immediate reduction of typhoid rates; as was also the case when Chicago began the use of the four-mile intake instead of the short one formerly used. The same result appears also in the records of Lowell and Atlanta.

In the winter of 1890 and 1891 a typhoid epidemic travelled progressively down the Mohawk and Hudson rivers, appearing at Amsterdam and Schenectady (on the Mohawk) during the latter part of 1890 and at Cohoes, Green Island, Troy, Albany, and Catskill in the early part of 1891. All of these cities derive their water from and empty their sewage into the Hudson, or its tributary, the Mohawk.

"If the drinking-water supplied to cities is an index of their healthfulness as measured by the typhoid-fever death-rates, we should expect to find some sort of relation existing between the quality of the water and the typhoid death-rates. Of course it is unreasonable to expect to find such an exact relation that there would be no variation from the rate for each different classification; there must be overlapping, as local conditions may make a certain water more subject to pollution than another. As a general thing, we should expect to find that cities whose water is kept perfectly secure from contamination, such as use spring-water secured in mountains where no pollution is possible, would have the lowest death-rates, and the range of fluctuation would be very small, comparatively speaking. Next we should expect to find that water properly filtered, as is done extensively in Europe, would show a low rate and one without much fluctuation, provided the operation is carefully and intelligently carried on. Next in purity we should expect to find ground-waters, and in these there might be accidental pollution that would cause considerable fluctuation. Then follow, in the order of their liability to contamination, large impounding reservoirs, where legal measures are taken to restrict pollution; large rivers, either normal, or where great volume and absence of any considerable evident place of pollution within a great distance, coupled with dispersion, sedimentation, and nitrification, may have brought a previously polluted river back to its normal condition. Then follow great lakes, whose waters at great distances from polluting sources are pure, but which are liable to pollution near the shores (in this class, from the relative positions of the intakes, we should expect to find rates varying from those of a very healthful city to those of the most infected); upland streams and small gathering-grounds where no special precautions may be taken to restrict pollution of the watersheds; and, finally, rivers and sources known to be polluted with sewage. In cities using such supplies we should expect to find high typhoid-fever death-rates, and a considerable variation of rates from year to year." (Fuertes' "Water and Public Health.") J. H. Fuertes has collected and arranged data from a large number of cities, both American and

European, and these data are shown on Plate II, page 24, as classified by him. His classification is as follows:

"Class A. Mountain springs with sources undoubtedly beyond the danger of pollution.

"Class B. Water properly purified by slow sand-filtration.

"Class C. Pure ground-water supplies.

"Class D. Surface-water supplies with large impounding reservoirs and legal provisions against pollution.

"Class E. Large normal rivers, or rivers in which the pollution may be considered to have greatly vanished through the agency of sedimentation, dilution, and other causes.

"Class F. Large inland lakes, which may be more or less subject to pollution.

"Class G. Upland streams and small lakes with limited watersheds which are more or less inhabited.

"Class H. All rivers and public and private wells which are known to be polluted with sewage and other infectious matter to varying degrees."

From a study of these data Fuertes finds that in cities using *filtered water* 94% of the death-rates per 100,000 are more than 3, 83% are less than 20, and 77% are between 3 and 20; that in cities using *ground-waters* 98% of the death-rates per 100,000 are more than 5, 77% are less than 32, and 75% are between 5 and 32; that in cities using *impounded waters* 97% of the death-rates per 100,000 are above 15, 80% are less than 35, and 77% fall between 15 and 35; that in cities using the *waters of large normal rivers* 90% of the death-rates per 100,000 are above 17, 85% are less than 38, and 75% are between 17 and 38; that in cities using the waters of the *great lakes* 93% of the death-rates per 100,000 are over 18, 80% are less than 54, and 73% are between 18 and 54; that in cities using the waters of *upland streams*, etc., 92% of the death-rates per 100,000 are over 29, 80% are less than 58, and 72% are between 29 and 58; that in cities using

PLATE II.—DEATH-RATES PER 100,000 PER ANNUM FROM TYPHOID FEVER.

polluted waters 95% of the death-rates per 100,000 are over 40, and only 65% fall between 50 and 100, the upper limit frequently exceeding 300. ("Water and Public Health.") Stated in another way, 75% of the death-rates in American cities fall between the following limits for each class of supplies:

Mountain Springs.	Filtered Water.	Ground-Water.	Impounded Waters.	Large Normal Rivers.	Great Lakes.	Upland Streams.	Polluted Waters.
2 to 10	3 to 20	5 to 32	15 to 35	17 to 38	19 to 53	28 to 57	39–100 +

The above are deaths only; but since only 9 to 12 per cent of typhoid cases ordinarily prove fatal, the chance of any citizen being infected with this disease is, say, 10 times these rates. From a humanitarian point of view it is the duty of all concerned to prevent as many deaths as possible; from the utilitarian, the value to the community of the lives lost must be considered. Rochard gives this as the sum "that he has cost his family, the community, or the state for his living, development, and education. It is the loan which the individual has made from the social capital in order to reach the age when he can restore it by his labor." This sum may vary from $500 to $50,000, but a conservative average would probably be $5000. Since by far the greater part of the typhoid victims are in their prime, and the majority are among the wealthier class, this value is above rather than below the average. Thus, where polluted waters are used each citizen, as part of the community, loses annually by deaths from $1.50 to $15 worth of potential labor, which might be reduced by filtration or the use of mountain springs to from 10 cents to $1.00. Moreover, the average length of sickness being, say, 45 days, each citizen would, with polluted supply, lose annually from .135 to 1.35 days of his fellow citizen's labor, which at $2 and the doctor's bills at a like sum would amount to from \$0.54 to \$5.40; making a total annual loss of from \$2 to \$20 per capita, as against a possible \$0.14 to \$1.36 with a pure water-supply. At Plymouth, Pa., in 1885 (population 8000) 1104 cases of typhoid fever and 114 deaths resulted from discharging the dejecta of one patient into the water of a small impounding reservoir. The care of these patients cost \$67,100.17, and the loss of wages by those who recovered \$30,020.08; an actual money loss in one year of \$12.14 per capita; or \$83.39 per capita if the value of each life be placed at \$5000.

ART. 10. SUMMARY: REQUISITE QUALITY.

It appears to have been demonstrated that an impure drinking-water is a very important cause of diarrhœa, typhoid fever, cholera, and probably a number of other diseases; typhoid fever and cholera being communicated by sewage-pollution. It is probable also that the immediate cause of these two diseases is found in water-borne parasitic bacteria; while that of the other enteric diseases may be either bacteria or matter which acts mechanically as an irritant. By its morbific qualities impure water may be considered to cause a loss to the community of from \$2 to \$20 per capita per annum; or \$1.86 to \$18.64 more than if pure water were used; and hence this sum capitalized, or, say, \$46.50 to \$465 per capita, could, from a utilitarian point of view, be profitably spent in purifying the water or obtaining it from an uncontaminated source. In deciding what can be called a pure water, or what limit of impurity can be fixed for potable water, great difficulty arises. Bacteriological analysis can inform us of the approximate number of bacteria per cubic centimeter in a water, but not certainly as to the presence of pathogenic ones. Chemical analysis can inform us of the amount of organic matter which is and which has been present, but not as to whether this is animal or vegetable,

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morbific or otherwise. Hence no absolute standards can be based upon the determinations of either or both of these alone. The probability of either bacteria or organic matter being of such a character as to render the water dangerous must be judged from their probable source and origin. A great increase of organic matter in a river below a city as compared with the same river above indicates sewage-contamination and danger; while a like increase in flowing through a forest is probably due to leaves and other vegetable matter only. It can be said, however, that a water low in bacteria and in organic impurities is in all probability a safe one; that a water with only a moderate amount of these present which comes from an unpopulated watershed or a deep well is likewise probably safe; but that a water should be treated with suspicion which is high in bacteria and organic matter, or which, containing a moderate amount of these, is subject to sewage or any human contamination.

For other than sanitary reasons a water-supply should contain little sediment or dissolved mineral matter or acids, and should have no appreciable color, taste, or odor; and the cooler it can be delivered to the consumer the better.

For water which is to be used for irrigation the requirements as to quality are seldom considered, since few waters available for this purpose contain matters injurious to vegetable life. Large amounts of alkaline salts, alum, and most other mineral impurities are not permissible; but excepting the first (chloride, sulphate, and particularly carbonate of sodium) and lime none of these are often found in appreciable quantities in large volumes of waters, and the lime is beneficial in many instances. Sediment is desirable for most soils, as it has considerable fertilizing value. "In the valley of Moselle, France, on land absolutely barren and worthless without fertilization, the alluvial matter deposited by irrigation from turbid water renders the soil capable of producing two crops a year. In the valley of the Durance, France, the turbid waters of that stream bring a price for irrigation which is ten and twelve times greater than that paid for the clear cold water of the Sorgues River. It has been estimated that on the line of the Calloway Canal in California land which has been irrigated with the muddy river-water gives 18% better results after the fifth year than the same land which has been irrigated with clear artesian water." (Wilson's "Manual of Irrigation Engineering.") Sewage is used for irrigation with excellent results.

QUERIES.

1. An analysis of the sewage of Meriden, Conn., showed 4.27 parts of chlorine, 0.840 of free ammonia, 0.976 of albuminoid ammonia. What was probably the number of gallons of flow to each contributor?

2. If Newark had a population of 160,000 in 1892, what sum could have been paid by her for the Pequannock water-supply in 4% 40-year bonds, without any net loss to herself, including expenses of sickness, and counting each life as worth $5000 to her?

CHAPTER III.

REQUISITES OF A SUPPLY. QUANTITY.

ART. 11. FOR IRRIGATION.

FOR irrigation about one to three cubic feet per year is required for each square foot of irrigated surface, varying with the crop, the porosity of the soil, the amount and character of vegetation thereon (as affecting evaporation), temperature and precipitation, and the skill of the irrigator. Probably with care 2 cubic feet should suffice for the most demanding crop on any soil. The water required decreases as the land is cultivated and as the ground-water surface rises owing to previous irrigation. The quantity of water used for irrigation is customarily stated in acre-feet, one acre-foot being the amount necessary to cover one acre one foot deep, or 43,560 cubic feet. The water is not used constantly, but is turned into the fields from two to ten times, or "services," a year. Vegetables are generally given more services than grain or grass, requiring also a greater total amount of water. The quantity stated is for the net area irrigated, and this will probably average about three fourths of the total area of a district. Of the run-off collected 15 to 50 or more per cent will be lost by evaporation and seepage from the reservoirs and canals. The number of acres which can be irrigated by the yield from a given area will therefore be approximately the quotient obtained by dividing the required depth of irrigating water (1 to 3 feet) into the product of the catchment area (in acres), the annual rainfall (in feet), the proportion of this collected, and the proportion of this last delivered (generally about .75). Thus, on the Sweetwater drainage area, in Southern California, the mean rainfall is 20.64 inches, average run-off 9.3%: the duty is 1.6 acre-feet per year. One acre of watershed would therefore furnish irrigation for $\frac{20.64 \times .093 \times .75}{12 \times 1.6} = 0.075$ acres. The watershed of 186 square miles would hence irrigate 13.95 square miles or 8928 acres. In 1896, 4580 acres were irrigated and 2600 people were dependent upon the reservoir for domestic water. (Schuyler, Eighteenth Annual Report of U. S. Geol. Survey.)

ART. 12. POPULATION TO BE SUPPLIED.

With very few exceptions American cities and villages have increased in population in the past, and will continue to do so in the future so long as commercial and social conditions retain their present character and tendencies. The growth of the United States follows very closely a general law, viz., about 30% increase per decade; that of each of the larger cities follows a uniform law with fair regularity; while smaller cities and villages may follow one law of growth for a series of years, when some change in condition may cause a considerable change in the law. It follows that the future population of a State or large city can be predicted with more certainty than can that of a small community. The percentage of increase per decade of New York City has been about 33; of Chicago, 106; of Philadelphia, 25; of Boston, 30; of Cleveland, 70; of Salem, Mass., 10.

The method of predicting future growths generally

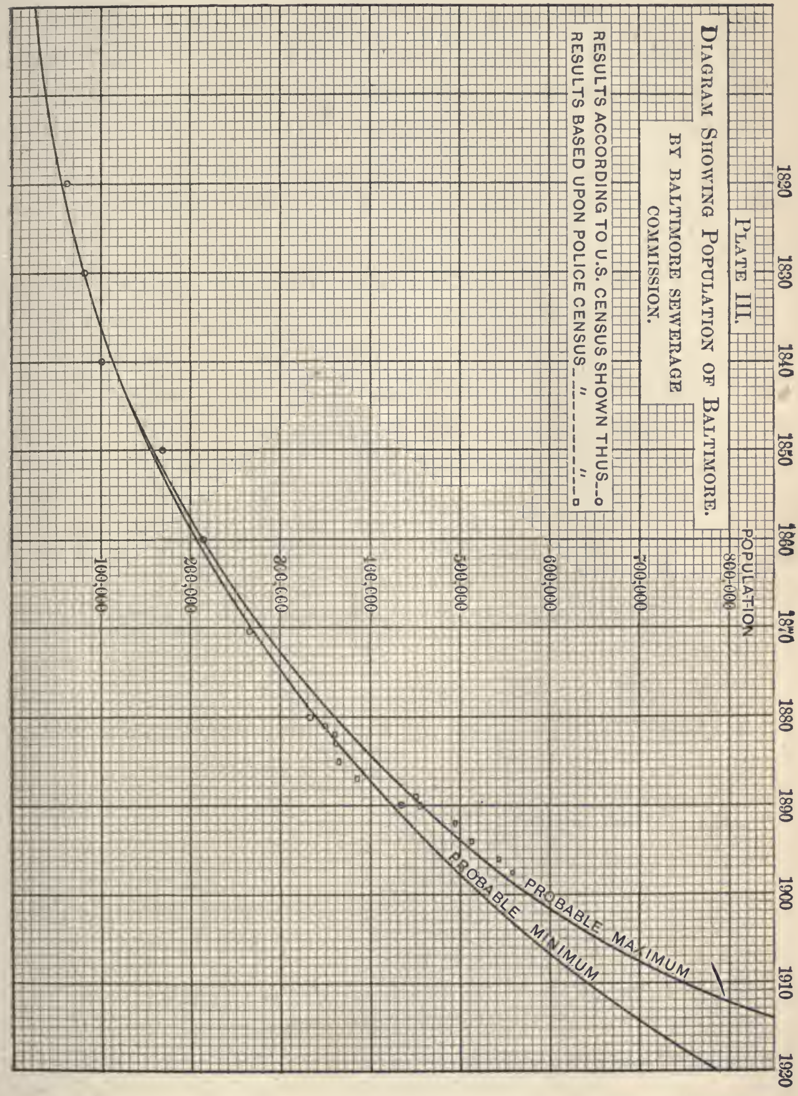
PLATE III.
DIAGRAM SHOWING POPULATION OF BALTIMORE.
BY BALTIMORE SEWERAGE COMMISSION.
RESULTS ACCORDING TO U.S. CENSUS SHOWN THUS...o
RESULTS BASED UPON POLICE CENSUS " " ...□
POPULATION
100,000
200,000
300,000
400,000
500,000
600,000
700,000
800,000
1820
1830
1840
1850
1860
1870
1880
1890
1900
1910
1920
PROBABLE MAXIMUM
PROBABLE MINIMUM

adopted is to plot all past known populations, using the year as one ordinate and the corresponding population as the other, pass a curve as nearly through these points as possible and, finding its formula, project it into the future years. This curve cannot of course return upon itself, but must approach some asymptote as a limit. It may, from unforeseen circumstances, reverse, even so far as to show a loss rather than gain of population. Such a curve for the city of Baltimore is shown in Plate III.

If the city is new, or no population data are available, the future population must be more or less of a guess pure and simple. Certain known facts may assist this, however. For instance, no modern city can reach any considerable size which is not well served by transportation facilities such as railroads, canals, or ocean steamers. Railroads and canals in a hilly or mountainous country follow the valleys, and the junction of several valleys, or a good harbor, is hence a locality favorable to a large population.

The U. S. Census shows that the density of population is greatest on the seacoast and diminishes gradually and quite uniformly up to the 2000-foot contour, above which the population is quite sparse; more than three fourths of the population living below the 1000-foot contour.

The table on page 33 shows the distribution of population relative to altitude, as determined by the U. S. Census.

The necessity for estimating the future population arises from the fact that the water-works should be of a capacity sufficient not only for the present, but for the future also; this being particularly true of the supply of water. For how distant a future is largely a matter of judgment, and depends somewhat upon the character of the works; but it is probable that forty or fifty years ahead would suffice for the majority of cases. This would generally mean a capacity three or four times that required for the present.

TABLE No. 1.

DENSITY OF POPULATION IN THE UNITED STATES RELATIVE TO ALTITUDE.

(From *Engineering News*, vol. XXVI. page 71.)

Altitude in Feet.	Population in Thousands.			Population per Square Mile.			Increase in Population per Square Mile.	
	1890	1880	1870	1890	1880	1870	1880 to 1890	1870 to 1880
0 to 100	10,387	8,273	6,441	51.8	41.3	32.2	10.5	9.1
100 to 500	13,838	11,654	9,240	35.6	30.0	23.8	5.6	6.2
500 to 1000	23,947	19,813	15,914	43.9	36.3	29.1	7.6	7.2
1000 to 1500	9,431	7,256	5,136	23.8	18.3	13.0	5.5	5.3
1500 to 2000	2,354	1,597	978	9.8	6.7	4.1	3.1	2.6
2000 to 3000	1,154	723	405	4.4	2.7	1.5	1.7	1.2
3000 to 4000	381	185	124	2.1	1.0	0.7	1.1	0.3
4000 to 5000	296	135	75	1.1	0.5	0.3	0.6	0.2
5000 to 6000	487	270	137	2.2	1.2	0.6	1.0	0.6
6000 to 7000	161	98	56	1.0	0.6	0.3	0.4	0.3
7000 to 8000	94	59	33	1.0	0.6	0.4	0.4	0.2
8000 to 9000	43	39	14	1.1	0.9	0.3	0.2	0.6
9000 to 10,000	39	45	3	2.0	2.3	0.2	—0.3	2.1
Above 10,000	10	9	2	0.5	0.5	0.1	.....	0.4

The future population is that within the future limits of the city, which is not necessarily the area now occupied. The population on any given area will probably approach, and in many districts will have already reached, a maximum limit. This may be from thirty to sixty per acre in suburban and the better class of residence districts, and several hundred per acre in crowded tenement blocks of large cities.

ART. 13. QUANTITY FOR CITY AND SUBURBAN USE.

In a private residence the water furnished is used for drinking, cooking, washing, and other domestic purposes; for watering horses and cattle, washing carriages and other stable duties; and for sprinkling lawns, flower-beds, etc. The approximate quantities so used on an American suburban property probably average about as follows:

For ordinary indoor use........	15 to 25 gals. per capita per day.
" stable use..................	50 to 200 gals. for each horse and carriage.
" sprinkling lawns in summer, and wasted to prevent freezing in winter.	200 to 2000 gals per house per day, during the coldest and dryest weather—say four months of the year.

For city residences, or those having small yards, and where stables are few, a yearly allowance of 20 to 30 gals. per capita per day should be ample for domestic use; 1 to 2 gals. per capita for stable use; 20 to 40 gals. per capita for office buildings, stores, restaurants, hotels, elevators, factories, etc.; for public schools, street-sprinkling, sewer-flushing, fountains, and extinguishing fires, 3 to 5 gals.; and for losses, 3 to 7 gals.; or, say, 45 to 85 gals. per capita per day. There may be numerous factories, breweries, or other industries using large amounts of public water which will greatly increase this per capita average; but the tendency is for such establishments to secure a private water-supply, as being cheaper for them in the end. The amounts used for various purposes in Boston in 1880 and 1892 are shown in the table on page 35.

The figures given above—45 to 85 gals. per capita—as ample for most cities are in a great number of cases exceeded, and in most such cases the excess is careless or wilful waste. The table on page 36 shows the consumption in a number of American cities.

The last two cities in the list, with practically the same population, show extreme rates of consumption. Nanticoke must waste at least one half its supply; while at Baton Rouge the low rate is accounted for by the small proportion of the population using the supply. New York, Brooklyn, Boston, St. Louis, Salem, Keokuk, and Brookline show by their 1890 record that even with some waste the rate can be kept under or near 75 gals.; while Providence and Fall River show that with careful administration it may be much lower than this. The great increase in New York's rate between 1890 and 1897 was due to the completion and use of the new Croton Aqueduct, furnishing increased quantity and head of water. Immediately before its completion the rate had fallen to 70 gals.

TABLE NO. 2.

CONSUMPTION IN GALLONS PER CAPITA FOR VARIOUS PURPOSES FROM THE COCHITUATE WORKS IN BOSTON IN 1880 AND 1892. (BRACKETT.)

	Population in 1880, 306,000.			Population in 1892, 430,000.		
	Metered.	Un-metered.	Total.	Metered.	Un-metered.	Total.
MANUFACTURE AND TRADES.						
Office buildings and stores ..	0.844	10.200	11.044	5.63	5.54	11.17
Steam railroads............	1.139	....	1.139	2.26	....	2.26
Sugar refineries............	0.811	....	0.811	1.70	....	1.70
Factories, machine-shops, mills, and engines	0.966	2.120	3.086	2.15	....	2.15
Iron-works and -foundries ..	0.573	....	0.573	0.24	....	0.24
Marble- and stone-works....	0.143	....	0.143	0.12	....	0.12
Gas companies..............	0.324	....	0.324	0.75	....	0.75
Electric light companies	....	....	....	0.69	....	0.69
Breweries..................	0.556	....	0.556	0.89	....	0.89
Oil- and chemical-works	0.214	....	0.214	0.19	....	0.19
Laundries..................	....	0.150	0.150	0.15	0.35	0.50
Restaurants	0.129	0.650	0.779	0.37	0.29	0.66
Stables....................	0.443	....	0.443	0.60	....	0.60
Steamers and shipping......	0.325	1.000	1.325	0.82	0.08	0.90
Elevators and motors	1.033	....	1.033	2.95	....	2.95
Street railways	....	....	....	0.90	....	0.90
Saloons	....	1.500	1.500	0.27	0.89	1.16
Hotels.....................	1.454	0.150	1.604	1.55	0.07	1.62
Theatres and halls.........	....	....	....	0.10	0.09	0.19
Markets and cellars.........	....	0.150	0.150	....	....	....
Greenhouses	....	0.160	0.160	....	0.08	0.08
Miscellaneous	0.314	....	0.314	0.27	0.28	0.55
Totals..............	9.268	16.080	25.348	22.60	7.67	30.27
DOMESTIC USES.						
Apartment hotels...........	0.047	5.850	5.897	1.72	12.34	14.06
Dwelling-houses............	....	50.000	50.000	....	43.90	43.90
Stables	....	1.500	1.500	....	1.38	1.38
Hand hose..................	....	1.250	1.250	....	2.25	2.25
Club-houses................	....	0.040	0.040	0.18	0.07	0.25
Churches...................	....	0.250	0.250	....	0.18	0.18
Miscellaneous	....	....	....	....	0.22	0.22
Totals..............	0.047	*58.890	58.937	1.90	*60.34	62.24
PUBLIC USES.						
Hospitals	....	0.200	0.200	0.30	0.21	0.51
Schools....................	0.505	0.400	0.905	0.30	0.12	0.42
City, State, and Government buildings................	....	0.400	0.400	0.83	0.52	1.35
Urinals, fountains, etc.	....	....	....	....	0.14	0.14
Miscellaneous....	....	0.500	0.500	....	....	....
Totals..............	0.505	1.500	2.005	1.43	0.99	2.42

* The quantity wasted is included in the unmetered consumption.

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Table No. 3.

POPULATION AND PER CAPITA WATER CONSUMPTION IN VARIOUS CITIES.

Consumption in Gallons per Day.

Date...	1850.		1860.		1870.		1880.		1890.		1897.	
City.	Population.	Consumption.	Population.	Consumption.	Population.	Consumption.	Population.	Consumption.	Population.	Consumption.	Population.	Consumption.
New York	......	...	......	..	942,292	82	1,206,590	77	1,515,301	85	1,700,000	112
Chicago	29,963	...	112,172	43	298,997	73	503,185	112	1,099,850	127	....	....
Philadelphia	121,376	...	565,529	36	674,022	55	847,170	68	1,046,964	132	1,385,734	215
Brooklyn	96,838	..	266,661	12	396,099	47	566,663	54	853,945	67	....	80
Boston	139,800	42	177,900	97	312,171	60	413,700	87	527,630	80	....	....
St. Louis	77,860	...	160,773	...	310,864	35	350,518	72	451,770	78	....	....
Cincinnati	115,435	20	161,044	30	216,039	48	255,139	76	296,908	115	....	....
Cleveland	17,034	...	43,417	14	92,829	31	160,146	65	261,353	106	....	....
Detroit	21,019	...	45,619	52	79,577	64	116,340	130	205,876	155	279,107	125
Providence	41,513	...	50,666	...	68,904	...	104,857	34	132,146	46	....	....
Fall River	11,524	...	14,026	...	26,766	...	48,961	30	74,308	28	....	36
Salem	20,264	...	22,252	...	24,117	31	27,563	55	30,801	69	....	....
Wilmington, N. C.	......	...	......	...	......	...	....	....	20,056	22	....	....
San José, Cal.	......	...	......	...	......	...	....	....	18,060	194	....	....
Keokuk, Ia.	......	...	......	...	......	...	....	....	14,101	78	....	....
Brookline, Mass.	......	...	......	...	......	...	....	....	12,103	73	....	....
Baton Rouge, La.	......	...	......	...	......	...	....	....	10,478	19	....	....
Nanticoke, Pa.	......	...	......	...	......	...	....	...	10,044	199	....	....

The consumption in a number of English cities in 1897 was as follows:

Table No. 4.

WATER CONSUMPTION IN ENGLISH CITIES IN 1897.

City.	Population.	Consumption, Gallons per Day.
Worcester	45,000	43
Wigan	60,000	20
Plymouth	98,575	59
Swansea	100,000	35
Middlesborough	187,331	61
Sheffield	415,000	21
Manchester	849,093	40
Average of thirty-six cities	....	33

The rates in these tables are obtained by dividing the total consumption by the total population. A larger percentage than formerly of each city's population now uses the public supply, and the number of faucets, bath-tubs, water-closet flushes, etc., in each house is greater than formerly; also more water is used for flushing sewers, public fountains, etc.; and these facts to a large extent account for the increasing rate of consumption in many towns. This increase in fixtures in Boston is shown by Table No. 5. (Brackett, Transactions American Society of Civil Engineers, vol XXXIV. page 203.)

TABLE NO. 5.

NUMBER OF WATER-FIXTURES IN USE AND FIXTURES PER CAPITA, ETC., ON COCHITUATE WORKS, FROM 1870 TO 1892.

Name of Fixture.	1870.	Percentage Increase.	1880.	Percentage Increase.	1890.	Percentage Increase.	1892.
Taps	5,893	56.6	9,228	61.7	14,922	12.0	16,706
Sinks	53,010	59.4	84,498	39.7	118,066	6.0	125,151
Bowls	23,961	92.5	46,116	39.8	64,462	6.2	68,448
Bath-tubs	8,892	93.8	17,230	85.2	31,914	17.5	37,495
Water-closets	25,050	107.7	52,030	75.4	91,280	12.5	102,687
Urinals	2,447	65.1	4,041	20.7	4,879	− 2.6	4,754
Washtubs	9,615	99.1	19,139	126.7	43,389	23.0	53,360
Private hydrants	547	− 63.9	197	− 81.7	36	− 50.0	18
Slop-hoppers	723	32.2	956	56.2	1,493	− 4.1	1,432
Foot-baths	73	90.4	139	101.4	280	− 7.1	260
Hydraulic rams	13	−100.0	......	....	....	....	....
Totals	130,234	79.3	233,574	58.7	370,721	10.7	410,311
Total population	250,500	31.7	330,000	24.4	410,600	4.8	430,200
Daily average consumption	16,257,700	72.2	28,000,000	21.0	33,871,700	22.0	41,312,400
Fixtures per capita	0.520	36.2	0.708	27.5	0.903	5.6	0.954
Consumption pr. capita	64.9	31.0	85.0	− 2.9	82.5	16.4	96.0
Consumption pr. fixture	124.8	− 3.9	119.9	− 23.8	91.4	10.5	101.0

The consumption so far referred to is the *average* daily consumption; the actual daily consumption will vary from month to month, week to week, and day to day. The maximum consumption will generally be in the dryest summer weather. A maximum rate in urban districts may occur in very cold winters, due to waste through faucets to prevent freezing. Table No. 6 gives the average monthly rates for several cities.

For some purposes it is desirable to know the maximum rates of consumption; and Table No. 7 gives these for

TABLE NO. 6.

MONTHLY RATES OF WATER CONSUMPTION IN SEVERAL CITIES.

Average Percentage of Annual Rate.

City.	Population.	Duration of Record.	January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Average Daily Consumption.
Rock Island, Ill...	13,595 to 15,020	1889 to 1894	98.2	98.5	91.7	89.0	90.7	105.2	112.3	114.3	114.0	105.2	99.6	94.8	143
Average of 13 New England cities and towns... ..	....	......	87.2	89.0	88.6	89.7	99.8	114.0	123.0	113.5	109.4	103.0	92.1	88.7	.....
Brooklyn, N. Y...	690,000 to 940,000	1886 to 1895	97.1	100.1	98.4	95.9	97.5	102.4	104.3	103.3	104.4	100.2	96.8	99.4	70
Rockford, Ill.....	24,000	1890 1891	87.9	98.3	88.3	97.7	101.0	105.6	106.3	109.7	109.9	99.2	98.8	94.3	94
Taunton, Mass....	27,000	1893 1896	87.5	101.9	91.7	94.4	103.2	114.0	116.4	112.4	101.0	98.9	91.9	86.5	42
Newton, Mass....	27,590 in 1895	1880 to 1897	85.0	86.0	84.0	89.0	98.0	120.0	125.0	116.0	112.0	99.0	94.0	92.0	63
Philadelphia, Pa..	1,175,000	1893	89.9	85.0	82.4	89.6	99.9	110.4	111.7	110.8	106.4	109.8	98.3	106.7	152
A fair average will be about.......			87	90	87	90	98	115	121	115	110	101	94	92	

TABLE No. 7.

MAXIMUM RATES OF CONSUMPTION.

City.	Duration of Record.	Population.	Daily Consumption, Year in Question.	Monthly Maximum.		Weekly Maximum.		Daily Maximum.		Hourly Maximum.	
				Month.	Rate Per Cent of Average.	Month.	Rate Per Cent of Average.	Date.	Rate Per Cent of Average.	Hour.	Rate Per Cent of Average.
Taunton, Mass.	3 years	27,000	42	July	128						
Rockford, Ill.	2 years	25,400	96.5	August	117						
Philadelphia, Pa.	2 years	1,175,000	155	July	112	....	....	Sept.	124		
Detroit, Mich.	3 years	260,000	150	....	125	....	140	....	150	....	178
Attleboro, Mass., and Woonsocket, R. I.	1 year	7,577 20,830	40.5 26.1	....	122	....	134	....	155	8 A.M.	333
Brooklyn, N. Y.	10 years	700,000	65	July	108	....	....	Feb. 8	127		
Reading, Pa.	2 years	74,400	87.1	....	....	....	....	July 9	131		
Rock Island, Ill.	....	....	121	August	128						
Boston (Mystic supply)	1 week	117,000	73.6	....	....	....	....	....	....	8 A.M.	141
Newton, Mass.	18 years	10,000 to 30,000	60.1	August	167	July	226	July	296		

several places for the month, week, day, and hour, in percentages of the average rate for the year in question.

The maximum rate of consumption at any one point will probably be that due to use for extinguishing fires. The annual rate per capita for fire purposes will probably be about 0.08 to 0.12 gals. per day; but the rate during use will be much greater than this. The number of fire-streams of 175 to 250 gals. per minute each required simultaneously in American cities of various magnitudes is considered by different authorities to be as follows:

TABLE No. 8.

NUMBER OF FIRE-STREAMS REQUIRED SIMULTANEOUSLY IN CITIES OF VARIOUS MAGNITUDES.

Population.	Freeman.	Shedd.	Fanning.	Kuichling.	Gallons per Minute by Freeman.
1,000	2–3	....	....	3	350–750
4,000	(4–6)	....	7	6	700–1500
5,000	4–8	5	....	6	700–2000
10,000	6–12	7	10	9	1050–3000
20,000	8–15	10	....	12	1400–3750
40,000	12–18	14	....	18	2100–4500
50,000	(14–20)	....	14	20	2450–5000
60,000	15–22	17	....	22	2625–5500
100,000	20–30	22	18	23	3500–7500
150,000	(24–36)	....	25	34	4200–9000
180,000	(27–40)	30	....	38	4725–10000
200,000	30–50	....	....	40	5250–12500

One of the benefits of a public water-supply is the fire protection afforded, and unless this is ample the system is lacking in one of its essential functions. This affects not only the possibility of extinguishing fires, but also, on account of this, the rates of fire insurance throughout the city; and in most cases the interest on the cost of ample provision for such service will be more than covered by the reduction in insurance rates.

ART. 14. WASTE OF WATER.

There are very few cities in which a yearly average of 75 gals. per capita per day is not sufficient for all uses, and in most cities where the consumption exceeds this it might be brought within this limit by proper treatment. This being the case, it is evident that there must be a great amount of water wasted in many cities. In New York City this appears to be about 40 gals., in Detroit 50, and in Philadelphia more than 140 gals. per capita daily. If the supply is pumped, this means increased expense for coal-consumption and enlargement of pumping-plant; if the supply is from wells or reservoirs, it hastens the day when new wells must be sunk or new drainage areas sought; and in any case it means that either the mains must be unnecessarily large or the pressure will be decreased below that desired. Millions of dollars are being spent by many of our larger cities to so increase their supply that two thirds of it may be wasted.

This waste is either intentional, careless, or through ignorance. Under intentional waste may be classed the opening of faucets on winter nights to prevent freezing of poorly located plumbing; the lavish sprinkling of lawns and streets in summer; and unnecessary amounts used in automatic sewer flush-tanks, as well as in automatic water-closet flushes in hotels and office buildings. Carelessness usually takes the form of non-attention to leaky house-fixtures, thousands of which can be found in almost any city. More or less leakage in every system defies detection, being probably the result of very slight leaks in thousands of joints, in fire-hydrants, stop-valves, corporation-cocks, and other portions of the distributing system. This has been found to be 5 to 10 per cent of the consumption in some cities.

The winter waste due to the first-mentioned cause is shown in Table No. 6. In Rock Island this was greatest in

January and February; in New England, Brooklyn, and Rockford, Ill., in February. In Rockford the February consumption is seen to be about 8% greater than that of the other two winter months. Daily records would show this waste much more prominently. At Reading, Pa., the maximum winter rate during 1897–98 was on Dec. 10, being 125% of the average for that year. At Newton, Mass., during twenty years past the winter weekly maximum has fallen in December four times, in January four times, and in February nine times; the greatest excess being in the second week of December, when the average consumption for the week was 122% of the annual.

The summer waste is more prominent, the average consumption during June, July, and August running from 4 to 15 per cent above the yearly average.

The waste of water by the city in flushing sewers by automatic tanks is in some cases most excessive. From 500 to 800 gals. per day per tank should be sufficient for the proper maintenance of a lateral sewer; but carelessness or ignorance has been known to cause ten to thirty times this amount to be used. The Water Company of Racine, Wis., found the 111 flush-tanks of that city using 1,436,429 gals. per day, an average of about 13,000 gals. each—an actual waste of certainly more than 1,000,000 gals. per day. Street-sprinkling is also a common cause of much waste, one Massachusetts city using 7% of its entire supply for this purpose in 1897.

The loss through leaky house-fixtures is enormous. "From 10 to 15 per cent of the premises examined in Boston are found to have defective fixtures, and of 12,609 defective fixtures reported in 1893, 7314 were ball-cocks. The tank ball-cock is without doubt the source of the greater part of the waste from defective plumbing." (Dexter Brackett, "Consumption and Waste of Water," Transactions American Society of Civil Engineers, vol. XXXIV. page 196.) In 1897 there were 9211 defective fixtures on 47,778 premises in Boston, and 1600 leaks on 4651 metered services. Of the latter, 959 were ball-cocks and valves, 521 were sink-, hopper-, bowl-, and bath-faucets, and 120 were burst service-pipes.

In Philadelphia, of 782 appliances in a certain section containing 539 population, 22 were leaking slightly, and 32 were running continually; and it was found in one district that 63% of the water furnished was wasted by 17% of the consumers, and in another 86% was wasted by 7% of the consumers; this being, not lavish use, but absolute waste.

A leaky faucet may waste 75 to 300 gals. per day, and a leaky ball-cock may easily permit 1000 gals. per day to pass it. Unless this leakage inconveniences the householder he will seldom repair the defective fixtures if not compelled to. The greatest amount of leakage of this class is generally found in rented houses, particularly of the poorer class, where the plumbing is cheap, and money for repairing a leak is spent grudgingly.

Aside from these there are many unknown sources of waste; generally, leaking mains or house-connections. Different authorities consider that an average of 500 to 3000 gals. leaks through pipe-joints, hydrants, and valves on each mile of pipe in a well-constructed system which has been in use for some years; the leakage being generally so small at any one point as not to be detected. Although in Boston broken 4- and 6-inch pipes leaking 1000 to 4000 gals. per hour have gone for years undiscovered; and other cities have undoubtedly had similar experiences.

Most commercial articles are sold by the quantity, and there seems to be no good reason why water also should not be. "Water should be free as air," the catch-word in many cities, may be true; but its distribution should not and cannot be, nor would this be expected if a cart instead of pipes were the distributing medium.

ART. 15. STATISTICS.

The following water-works statistics are added as of interest in this connection, although not requiring special discussion.

TABLE No. 9.

PERSONS PER DWELLING IN VARIOUS CITIES.

City.	1870.		1880.	
	Population.	Persons to a Dwelling.	Population.	Persons to a Dwelling.
New York, N. Y.	942,292	14.72	1,206,299	16.37
Philadelphia, Pa.	674,022	6.01	847,170	5.79
Brooklyn, N. Y.	396,099	8.64	566,663	9.11
Chicago, Ill.	298,977	6.70	503,185	8.24
Boston, Mass.	250,526	8.46	362,839	8.26
St. Louis, Mo.	310,864	7.84	350,518	8.15
Baltimore, Md.	267,354	6.63	332,313	6.54
Cincinnati, O.	216,239	8.81	255,139	9.11
San Francisco, Cal.	149,473	5.77	233,959	6.86
New Orleans, La.	191,418	5.69	216,090	5.95
Cleveland, O.	92,829	5.56	160,146	5.89
Pittsburg, Pa.	86,076	6.05	156,389	6.44
Buffalo, N. Y.	117,714	6.44	155,134	6.55
Washington, D. C.	109,199	5.59	147,293	6.11
Newark, N. J.	105,059	7.32	136,508	7.26
Louisville, Ky.	100,753	6.87	123,758	6.55
Jersey City, N. J.	82,546	8.37	120,722	8.59
Detroit, Mich.	79,577	5.42	116,340	5.68
Milwaukee, Wis.	71,440	5.48	115,587	6.17
Providence, R. I.	68 904	7.46	104,857	7 41
Albany, N. Y.	69,422	7.94	90,758	6.85
Rochester, N. Y.	62,386	5.36	89,366	5.65
Allegheny, Pa.	53,180	6.37	78,682	6.59
Indianapolis, Ind.	48,244	6.17	75,056	5.47
Richmond, Va.	51,038	6.35	63.600	6.67
New Haven, Conn.	50,840	6.28	62,882	6.31
Lowell, Mass.	40,928	6.43	59,475	7.21
Kansas City, Mo.	32,260	5.95	55,785	6.48
Charleston, S. C.	48,956	7.14	49,984	7.63
Scranton, Pa.	35,092	6.21	45,850	6.25
Lawrence, Mass.	28,921	8.40	39,151	8.50
Denver, Col.	....	....	35,629	6.75
Memphis, Tenn.	40,226	6.28	33,592	4.68
Trenton, N. J.	....	....	29,910	5.85
Norfolk, Va.	....	....	24,966	6.70
Holyoke, Mass.	....	....	21,915	10.52
Springfield, Ill.	....	....	19,743	5.60

TABLE No. 10.

POPULATION PER MILE OF MAINS AND PER TAP, COST OF SYSTEMS PER MILE OF MAIN, AND PRESSURE IN MAINS IN POUNDS; AVERAGES FOR THE UNITED STATES IN 1888.

District.	Population per Mile of Main.	Population per Tap.	Cost per Mile of Main.	Pressure * in Mains. Pounds.
New England	455.4	7.6	$16,800	30–100
Middle	830.0	6.9	21,000	30–110
South Atlantic	713.6	11.1	13,070	30–90
South Central	780.1	13.4	20,650	30–90
North Atlantic	597.7	8.5	13,780	30–100
Northwestern	467.3	12.1	15,660	20–90
Southwestern	563.1	9.4	21,580	30–90
Pacific	276.0	6.1	19,800	20–90
Average	585.4	9.4	17,800†	30–90

* 80 per cent of each district is within the limits given. The actual range is from 10 to 180 pounds.

† Except a few large cities, very few plants cost more than $20,000 per mile, and few less than $10,000.

TABLE No. 11.

NUMBER OF TAPS AND METERS, AND MILES OF PIPES; SEVERAL CITIES; 1897.

City.	Population.	Per Capita Consumption.	No. of Taps.	No. of Meters.	Miles of Distributing Pipes.
New York, N. Y.	1,700,000	112	120,000	32,329	750
Philadelphia, Pa.	1,385,734	187	221,860	1,273	1174
Boston, Mass.	527,630	80	70,879	4,398	596
St. Louis, Mo.	451,770	78	53,354	3,979	462
Detroit, Mich.	205,876	155	48,918	4,000	501
Providence, R. I.	132,146	46	18,695	13,763	294
Indianapolis, Ind.	110,000	81	7,000	500	157
Charleston, S. C.	55,000	28	1.500	25	33
Reading, Pa.	74,410	87.1	14,860	432	88
Yonkers, N. Y.	32,033	100	3.714	3,706	56.7
Augusta, Ga.	33,300	115	....	62	44
Taunton, Mass.	25,448	45	3,843	1,366	71.0
Canton, O.	26,189	110	3,900	20	48
Leavenworth, Kan.	19,768	150	1,440	450	27
Burlington, Vt.	14,590	60	2,969	1,261	36.0
Ogden, Utah	14,889	265	1,800	65	32
El Paso, Tex.	10,338	55	800	450	10
Middleboro, Mass.	6065	35	697	256	14
Charlottesville, Va.	5591	100	871	3	28
Wellesley, Mass.	3600	48	685	656	26
Ocala, Fla.	2904	40	266	3	7.5
Alexandria, La.	2861	5	55	0	7
Elgin, Minn.	885	4.5	25	6	1±
Garfield, Wash.	317	28	30	0	1.2

TABLE NO. 12.

CONSUMPTION PER CAPITA IN VARIOUS CITIES, AS DETERMINED BY METER MEASUREMENTS.

(Brackett in Trans. Am. Soc. C. E., vol. XXXIV. page 189.)

City or Town.	No. of Houses.	No. of Families.	No. of Persons.	Consumption, Gallons per Family.	Consumption, Gallons per Capita.	
Boston, Mass.	31	402	1461	221	59	Highest cost apartment houses in city.
Boston, "	46	628	2524	185	46	First-class apartment houses.
Boston, "	223	2204	8432	123	32	Moderate class apartment houses.
Boston, "	39	413	1844	80	16.6	Poorest class apartment houses.
Boston, "	339	3647	14,261	139	35.6	Average of all apartment houses supplied by meter.
Boston, "	40	....	1699	....	46.1	Boarding-houses.
Brookline, "	....	828	4140	221.5	44.3	Average of all dwellings metered.
Newton, "	490	490	2450	132.5	26.5	All houses with modern plumbing.
Newton, "	....	619	3005	....	6.6	Families having but one faucet each.
Newton, "	....	278	1390	34.5	6.9	" " " " " "
Fall River, "	28	34	170	127.5	25.5	Most expensive houses in the city.
Fall River, "	64	148	740	42.0	8.4	Average class of houses, generally having bath and water-closet.
Fall River, "	....	....	70,000	....	12.3	Total domestic consumption.
Worcester, "	....	20,514	90,942	....	16.8	" " "
Worcester, "	....	81	327	80.2	19.9	Woodland Street, best class of houses.
Worcester, "	....	37	187	118 1	23.4	Cedar Street, " " " "
Worcester, "	....	93	447	95.0	19.8	Elm Street, houses of moderate cost.
Worcester, "	....	245	1104	55.1	12.2	Southbridge Street, cheaper houses.
Worcester, "	....	229	809	55.0	15.6	Austin Street, cheaper houses.
Yonkers, N. Y.	....	....	31,000	....	21.4	Houses renting from $250 to $600; each
London, England	1169	....	8183	....	25.5	have bath and two water-closets.
London, "	727	....	5089	....	18.6	Middle class, average rental $200.

OF CALIFORNIA

TABLE No. 13.

CONSUMPTION; NEW ENGLAND CITIES; 1896.

(Journal N. E. W. W. Ass'n.)

Name of City or Town.	Estimated Population.			Quantity Used through Meters in Gallons.		Percentage of Total Consumption Metered.	Average Daily Consumption. Gallons.	Gallons per Day.		
	Total at Date.	On Line of Pipe.	Supplied at Date.	Domestic.	Manufacturing.			Each Inhabitant.	Each Consumer.	Each Tap.
Attleboro, Mass.....	8,500	7,000	6,500	115,559,540		.....	315,612	37.1	......	48 5
Bay City, Mich.....	33,000	20,000	14,000	53,762,412	52,977,831	10.57	2,758,261	83.0	197.0	1,484.0
Brocton, Mass.......	35,000	32,000	31,000	116,536,470	40,604,662	....	1,087,111	31.06	34.90	239.80
Burlington, Vt.....	17,300	16,900	16,700	107,786,961	18,291,510	. . .	835.565	48.0	50.0	272.0
Fitchburg, Mass.....	26,500	23,000	20,300	56,900,000	279,000,000	....	....	94.0	123.0	670.0
Lynn, Mass.	69,361	.. ...	67,139	249,906,830		. ...	4,551,789	.....	67.8	......
Middleboro, Mass.	*4,200	......	.....	....	....	....	....	.....	.....	....
	7,000	4,000	3,700	.. ... .	....	....	....	50.25	57.0	292.0
New Bedford, Mass.	59,000	49,530	45,570	40,475,088	316,760,610	.....	5,259,317	89.0	108.0	623.0
New London, Conn	15,500	15,000	14,500	....	....	.....	1,385,134	89.0	95.5	505.0
Newton, Mass... ...	29,500	29,100	28,900	275,000,000	55,000,000	.. .	1,812,484	61.4	62 7	289 0
Oberlin, O.	4,600	3,500	2,000	6,000,000	1,900,000	33.0	67,200	14.6	33.6	143.0
Reading, Mass.. ..	4,750	4,160	3,600	5.545,170	1,031,483	.....	199,273	41.95	47 89	245.41
Springfield, Mass....	54,812	48.800	42.500	372,253,326		21.0	4,819,828	88.0	113.0	581.0
Taunton, Mass.	27,093	26,700	26,361	51,276.720	92,735,534	...	1,167,863	43.0	44 0	296.0
Waltham, Mass.. ...	21,500	21,000	20,500	33.441,260		... .	1,521,133	70.0	72.0	490.0
Woonsocket, R. I....	28,500	26,500	26,500	....	....	. .	708,227	24.9	26.7	359.0
Yonkers, N. Y..	40,000	34,000	33,000	248,223,254	365,141,844	50.3	3,331,802	83.0	100.0	910.0

* Fire district.

ART. 16. SUMMARY. QUANTITY TO BE PROVIDED.

For domestic use 30 to 50 gals. per capita per day should be sufficient; and where manufacturers and other similar consumers use large amounts, these will seldom bring the total of water *used* above 60 to 85 per capita. In many cases these limits are not reached; and in none should they be exceeded, with proper management. If, however, a large amount of water is permitted to be wasted there is no limit to the possible consumption, but this may reach 300 or even 1000 per capita. The most abundant supplies are generally those requiring pumping, and the cost of reservoirs, pipes, etc., as well as of pumps and fuel, increases with the consumption. A rate of 100 gals. per capita should be sufficient allowance for *any* ordinary case, and the consumption should be kept within this limit unless the supply can be increased at a less cost—including pumps, reservoirs, pipes, and all features of the system—than that of limiting the waste by inspectors and meters.

The average rate for the year is exceeded by 15 to 30 per cent during certain months, and 50 to 200 per cent during certain days or hours. A large part of this increase is in most cases wasteful, but unless waste is strictly prevented an allowance should be made for variations in monthly rates (see Table No. 6, page 38); and the maximum daily rate may be made 50% greater than the monthly, which maximum will, in residential districts, occur most frequently on wash-days.

For periods of a few minutes or hours still higher rates may be occasioned by fires. Table No. 8, page 40, shows the number of streams which may be required simultaneously, the discharge of each of which will generally be at the rate of 250,000 to 360,000 gals. per day. For a city of 10,000, for instance, this would temporarily increase the rate by 1,500,000 to 4,320,000 gals., or 150 to 432 gals. per capita per day; and the rates thus increased must be in every way provided for if ample fire protection is to be furnished. But the total annual rate is not appreciably affected by this.

QUERIES.

3. If the maximum hourly consumption at Detroit, Mich., given in Table No. 7 was during the largest fire allowed for in Table No. 8, what percentage of the average rate was the domestic consumption at that time?

If the maximum hourly rate given for Boston was *not* during a fire, what would be the total maximum rate to be provided for?

4. If each fixture in Boston leaked at the rate of two drops per second during the year 1892, what would have been the per capita consumption if all leaks had been stopped?

5. Judging from Table No. 10, which part of the country was, in 1888, most liberally provided with water for domestic use?

Allowing one tap to each dwelling, and an extra factory tap to each 75 inhabitants, find from Tables 9 and 11 the rate of consumption per actual consumer in each of the cities mentioned in both tables; using the "persons to a dwelling" in 1880 as still applicable in 1897.

CHAPTER IV.

SOURCES OF SUPPLY.

ART. 17. RAIN.

IN the temperate and frigid zones rain (and snow, which is generally included under this term unless otherwise specified) alone is considered as the source of water. For the use of man, whether for domestic or manufacturing purposes or for irrigation, the rain, since it does not fall continuously, must be caught and stored to tide over periods of longer or shorter duration between rainfalls. Nature performs this to a considerable extent through the agency of porous soil and rocks, underground caverns, lakes and ponds, glaciers, and in other ways; man, by the use of cisterns and larger reservoirs, and to a small extent by the storing of ice. The amount of rain which falls directly into a reservoir is generally but a small part of its capacity; but it is that which flows from some drainage area, whether a roof or a watershed of thousands of acres, which gives the most of the supply. If this drainage area is the surface of the ground, the run-off or yield is called surface-water; but if collected from an artificial surface it is considered solely as rain-water. Water is collected in the latter way and stored in cisterns in many low, flat countries or those with very porous soils, where the hungry soil yields none of the rain as surface-water, or where there are no natural basins for impounding it. Many Southern cities in our own country and the adjacent countries and islands rely almost wholly upon the collection of water from the roofs of dwellings and other buildings.

Art. 18. Surface-water.

By the "yield" or "run-off" of a catchment area, which constitutes surface-water, is meant the total amount of water which flows from a given drainage area, generally in the shape of streams fed by the rainfall upon this area. This is never the whole of such rainfall.

In falling, some rain is intercepted by the foliage and stems of trees and smaller plants, to be later evaporated back again into the air. Of that which reaches the earth, a part flows over the surface and the remainder enters the soil. If the soil be very porous, almost all the rain reaching it may be absorbed; if non-porous, little may enter it. All soil, even the densest rocks, will absorb some water, however. As the unabsorbed water flows over the surface to the lower levels it increases in volume and forms into rivulets, these unite to form larger streams, and the river formed from the union of thousands of such finally enters the ocean. (Exceptions will be referred to later, where streams are wholly evaporated; also others where they enter the ground and emerge into the ocean far from the shore and at a considerable depth.)

After a rain has ceased the streams carry less and less water, but those of any size seldom become entirely dry, even though weeks may elapse between rainfalls and the surface of the ground become dry and dusty. The immediate supply during this time cannot be the rain; but is found to be that portion of previous rainfalls which was absorbed by the earth and which is now being yielded slowly. In general, the more porous the soil the more water it will receive for this purpose during a given time of rainfall; and the finer its grain the more slowly will its supply be yielded and become exhausted. In some instances the ground-flow does not reach the same stream as does the surface-flow, but is carried by the dip of the strata into another valley, as in Fig. 1.

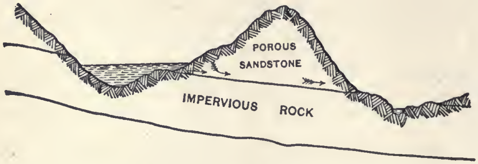

FIG. 1.—GROUND-WATER DIVERSION BY INCLINED STRATA.

The ground-flow frequently emerges as springs; but the larger part of it ordinarily reaches the stream as a general exuding from the banks and in some cases the bottom of the channel.

A study of the material and dip of the strata, and of the surface conditions—slope, vegetation, existence of ponds, etc.,—as well as of the rainfall and other meteorological conditions, is necessary for forming any estimate as to the probable amount and rate of yield of a given watershed, where the actual measurement of this is impossible. Such measurement, to be of value, should be continued for a series of years.

The total amount of rainfall reaching the ground is not yielded by the combined surface- and ground-flow; a large part is evaporated from the surface and from ponds or other bodies of water, large or small; considerable is taken up by the vegetation, which frequently sends its roots several feet into the soil in search of it, the greater part of this being later evaporated from the foliage; and part is held in the soil by capillary attraction. Probably none of the precipitation settles into the lowest strata which have no outlet, since these were filled ages ago. But if man removes water from these strata by deep wells, the amount thus withdrawn must be replenished. From many watersheds in the Atlantic and Gulf States the ground-water travels hundreds of miles to emerge in the ocean-bed far from shore; as off the coast of South Carolina, east of Matanzas Inlet, Fla., and off the shore of Pensacola, Fla.; which water must be withdrawn from the yield of these watersheds.

ART. 19. RIVERS AND LAKES.

The dividing-line between surface- and river-supplies is an indefinite one; but where the supply is taken directly from a river or lake without impounding or storage, this should undoubtedly be called a river- or lake-supply. The conditions are in many respects similar to those affecting surface-waters; but the supply is generally somewhat more constant and of greater volume, owing to the larger drainage area; it is more likely to be polluted, and to lie lower relative to the point of utilization.

Lakes are nature's regulators of flow, and take the place of artificial storage-reservoirs, besides contributing to the self-purification of river-waters. They are generally but enlargements of a river-channel; although some lakes are formed directly from surface-flow or from large springs, and form the sources of rivers; while still others have ground-water as both source and outlet. Lakes can in most cases be relied upon as more constant than rivers in both the quantity and the quality of water available.

ART. 20. UNDERGROUND SUPPLIES.

The water flowing underground towards a surface stream or the ocean may emerge as springs or be reached by a well dug or bored to and through the porous stratum through which it flows. Such water seldom flows in the form of a stream, nor is it collected in caverns as "underground lakes"; but generally fills the porous stratum throughout, and moves slowly through the interstices towards the outlet.

This movement is not always downward, but the outlet must always be lower than the point where the water enters the soil. If, in travelling through such stratum, the ground-water reaches a fault, this may be followed to the surface and the water emerge as one or more springs.

ART. 21. OTHER SOURCES.

In the form of snow and ice, water is stored by nature and man. Many Swiss streams have as their origin thawing glaciers formed from the snowfall of many years ago. Many thousands of tons of ice are stored annually by man, much of which is used in and for drinking-water. But this use is only incidental, and will be considered only in reference to its purity.

Dew and fog have, in a few cases, furnished limited supplies. Construction camps in Mexico have found supplies of fresh water among the sand-dunes; and Laguna Honda, San Francisco, among the sand-dunes, gives more water towards the city supply than the rainfall on its drainage area would furnish.

ART. 22. RELATIVE USE OF DIFFERENT SOURCES.

The table on page 55, compiled from the "Manual of American Water-works" for 1897, shows the percentage, in each of seven divisions in the United States, of works which utilize each of the several sources of supply.

The indefiniteness of the classification used by the different contributors of the data make it difficult to be certain as to the proper placing of each plant; and in some cases two or more sources are used by the same city. But the table gives an approximate statement of some interest and value. We see that over 25% of the plants in the country have ground-water (well) supplies, the largest proportion being in the North-central and Northwest sections; and that New England and the Middle States, with their hilly country, contain the largest number of surface supplies. Also that in all the Southern States rivers are used more generally than any other one source, and ground-water next.

TABLE No. 14.

PERCENTAGE OF CITIES IN THE UNITED STATES USING DIFFERENT SOURCES OF SUPPLY.

	Surface.	Creek.	Brook.	River.	Lake.	Spring.	Well.	Unknown.
New England	14.0	......	16.3	14.0	27.1	17.4	7.7	3.5
Middle States	8.3	14.8	10.7	21.6	4.4	25.1	14.1	1.0
South Atlantic and South-central	3.1	8.6	4.7	29.7	4.7	4.7	21.9	3.9
North-central	3.0	4.5	......	26.1	13.2	9.0	41.5	2.7
North-west	3.0	6.0	0.4	26.6	2.6	4.7	48.5	8.2
South Pacific	4.8	10.9	1.4	44.8	1.4	11.5	23.8	1.4
Pacific	3.0	14.3	6.0	28.6	3.0	20.3	21.8	3.0
Whole United States	6.1	8.3	6.1	25.4	9.1	16.1	25.7	3.2

In the next few chapters the sources above referred to will be considered at length.

CHAPTER V.

RAINFALL.

ART. 23. QUALITY OF RAIN-WATER.

THERE is a popular impression that all rain-water is as pure as any natural water obtainable, but this is not borne out by analyses. In fact, it is not reasonable to expect to find it so when we consider the large amount of impurity contained in the atmosphere, and recall how much purer and clearer this seems after a rain. This impurity consists of dust of mineral matters; of vegetable pollen, dried particles of insects, and other organic matter; and soot, sulphuric acid, ammonia, carbon dioxide, and other acids and gases are found in the atmosphere in and near cities. For instance, Mabery at different times found in the air of Cleveland, O., the following quantities, in milligrams per liter of air:

Soot.	Sulphuric Acid.	Ammonia.
87.5	15.2	.070
45.2	6:3	.010
111.3	21.2	.120
41.8	13.9	.003

Not only are these impurities washed from the air; but, in evaporation, solid matters in solution are carried up from water-surfaces. Thus, salt from the ocean is found in rain; one evidence that this is the source of chlorine in rain-water being the general parallelism of isochlors to the coast. (See Plate I, page 15.)

Bacteria are carried down in small quantities by rain, but probably none of these are pathogenic.

The following analyses are given by the Massachusetts State Board of Health in their Report for 1890.

TABLE No. 15.

ANALYSES OF RAIN-WATER.

Date.	Collected at	Ammonia.		Nitrogen as		Chlorine.
		Free.	Albuminoid.	Nitrates.	Nitrites.	
July 7, 1888..	North Andover ..	.0047	.0038	.0070	.0000	....
Sept. 18, 1888	" " ..	.0016	.0026	.0040	.0000	....
" 21, 1888	Lawrence........	.0298	.0024	.0000	.0000	.007
Oct. 2, 1888...	"	.0414	.0030	.0100	.0000	....
Nov. 27, 1888	"	.0164	.0014	.0050	.0002	.360
May 21, 1889	"	.0086	.0026	.0030	.0001	.070
June 17, 1889	Jamaica Plain....	.0564	.0152	.0180	.0004	.130
Mar. 28, 1890	NewtonHighlands	.0154	.0034	.0050	.0001	.060
Dec. 26, 1887	Boston (snow)....	.0258	.0038	.0030	....	....
	*Troy, N. Y......	.0460	.0225	.0200	.0000	.187
	*Menands Station, N. Y...........	.0150	.0060	.0000	.0000	.060

* From Mason's "Sanitary Water Supply." Both snow.

"When rain, particularly that first falling, is collected in a clean glass bottle, it is seen to be quite dirty. In cities where the air is full of soot this is very marked, but even in the open country the floating matter in the atmosphere is considerable. At the close of a protracted rain the water may be nearly or quite clear, since the air has then been washed clean; and the ammonia is also less than at the beginning of a rainfall. A part of the ammonia in the rain-water has its origin in the direct formation of ammonia from the nitrogen of the air and the hydrogen of the watery vapor, but its main source is undoubtedly in animal exhalations, fine particles of organic dust, gaseous products of decay, etc." (Mass. State Board of Health, Report for 1890.)

A larger proportion of the snow than of the rainfall is frequently contributed to the run-off, but the total amount in any but extreme northern localities is not considerable (snow on melting reduces to about one tenth its bulk of water), although sufficiently so to be of some importance. Snow usually removes more impurity from the atmosphere than does rain, owing to its form. This greater impurity of snow is seen in the above table, the Troy and Boston records being city snow; the Menands Station, country snow.

Rain-water, on account of its softness, is especially adapted to washing; and where the public supply is hard, roof-water collected in cisterns may form a very advantageous auxiliary supply for this purpose. It is also a most wholesome beverage if so collected and stored as to retain its original purity, since the impurities which it contains are not generally injurious unless allowed to accumulate and putrefy. Unfortunately the method of collecting and storing private supplies is often very faulty. Every roof before a rain is foul with excrement of birds, dead insects, leaves, and dust, and in too many cases all of this is washed into the cistern. The first part of each rain should be run to waste, and only after the roof is washed clean should the water enter the cistern. There are many automatic devices for accomplishing this, but few are in common use. A simple two-way valve in a tin or copper breeches-pipe at the bottom of the rain-water leader, to be turned by hand, is a common and efficient contrivance, if properly used and always so left after a storm as to waste the water until turned.

The best tank for storing water is made of slate. Iron properly coated to prevent rust is excellent also. Particularly in the South, cypress wood is much used for cisterns. Masonry walls are ordinarily used for underground cisterns, but should be absolutely tight, and this condition should be tested at intervals of not more than one year. For a time, at least, water stored in these is apt to be hard owing to the lime in the mortar.

Tanks or cisterns should always be covered to keep out dust and other impurities. They should also be so located and so tightly constructed that contamination from outside is absolutely impossible. If underground, their bottoms should be above the top of the sewage in any cesspools located within a radius of 1000 feet; and if privies or other surface deposits of excreta be located within that distance, the tank should be entirely above ground. The air reaching the tank should be pure; and no overflow or other pipe should connect it directly with the sewer. In New Orleans, where a large part of the water-supply is from private cisterns, "the usual capacity of the dwelling-house cistern is about two thousand gallons. They are raised a few feet from the ground, and their contents are protected by a lid or cover. Some are placed under the shade of a balcony; a few have a special roof over them; but the majority have only such protection from the rays of the sun as is afforded by their position against the house-wall. Many, especially in the older parts of the city, are situated in unventilated inclosures which are rank with the emanations from unclean privies." (Dr. Smart, Bulletin National Board of Health, April 17, 1880.) The deposit in New Orleans cisterns Dr. Smart found to collect at an average rate of an inch a year. Sediment collects from all rain-water, and should be removed before reaching any considerable amount, since it is stirred up by the inflow from each rain, and the organic matter therein is continually decomposing.

The following table gives the analyses of a few cistern-waters:

TABLE No. 16.

ANALYSES OF CISTERN-WATERS.

(Nichols' "Water-supply, Chemical and Sanitary.")

Locality.	Total Solids.	Ammonia.		Chlorine.	Authority.
		Free	Albuminoid.		
Boston, Mass...........	5.28	0.013	0.008	0.32	W. R. Nichols
Same, filtered *	6.56	0.012	0.007	0.36	"
Boston, Mass...........	3.24	0.005	0.011	0.10	"
Same, filtered *..........	4.80	0.024	0.016	0.12	"
Boston, Mass...........	3.48	0.021	0.007	0.69	"
Same, filtered *..........	5.20	0.007	0.007	0.70	"
Wilmington, N. C......	5.05	0.002	0.015	0.70	C. W. Dabney
"	6.90	0.016	0.008	0.52	"
"	3.60	0.005	0.008	0.52	"
Cincinnati, O...........	2.68	0.004	0.123	0.55	C. R. Stuntz
"	4.72	0.275	0.055	2.76	"
"	4.48	0.027	0.118	1.97	"
"	7.96	0.004	0.016	trace	"
"	4.10	0.020	0.360	trace	"
West Troy, N. Y........	....	0.1050	0.0175	0.2000	W. P. Mason

* These cisterns were provided with a brick filtering-wall—the inefficiency of which is evident.

ART. 24. QUANTITY OF RAINFALL: ANNUAL.

Rainfall, being the origin of all supplies, is the basis of calculation of the amount available from whatever source, and a consideration of the amount of rainfall is an essential foundation for further discussion.

The rain which will fall at any one place in any day, month, or year cannot be accurately predicted by any known theory or science; but a record of past rainfalls will afford an aid to our judgment in estimating such amount, and in fact forms practically the only basis for such judgment; although in some cases probable changes in certain large features of the country—such as deforestation—may be considered. (Authorities differ as to whether the presence of forests causes increased precipitation; but there seems to be little proof that such increase, if it is so effected, is at all considerable in amount.)

Table No. 17 gives the mean, maximum, and minimum annual precipitation in the United States for each of twenty-one districts into which the Weather Bureau has divided the country; and Table No. 18 the same also arranged according to altitudes, showing the maximum and minimum averages for all places in each district within the indicated range of elevation. For instance, of all stations in New England having an altitude of less than 100 feet above sea-level, the maximum average precipitation found at any one is 45.18 inches, and the minimum 40.73 inches. These tables are based on all records up to Jan. 1, 1899.

TABLE No. 17.

MEAN ANNUAL PRECIPITATION IN THE UNITED STATES.

(Mean, Maximum, and Minimum Averages of Stations in each Meteorological District.)

Districts.	New England.	Middle Atlantic States.	S. Atlantic States.	Florida Peninsula.	East Gulf States.	West Gulf States.	Ohio Valley and Tennessee.	LowerLakeRegion.	UpperLakeRegion.	North Dakota.	Upper Mississippi Valley.
Mean	43.46	43.75	54.09	49.78	54.25	43.15	45.45	35.45	32.61	18.95	34.21
Maximum	47.51	52.34	66.41	57.98	62.61	53.63	54.97	41.28	35.08	23.77	42.83
Minimum	35.74	37.89	47.55	38.46	51.97	29.70	36.68	30.93	29.53	14.70	27.21

Districts.	Missouri Valley.	Northern Slope.	Middle Slope.	Southern Slope.	Southern Plateau.	Middle Plateau.	Northern Plateau	N. Pacific Coast Region.	Middle Pacific Coast Region.	Southern Pacific Coast Region.
Mean	30.25	14.39	22.33	21.61	8.44	12.22	16.36	45.27	29.81	14.59
Maximum	39.93	18.27	33.29	25.02	14.25	16.19	18.25	62.27	45.83	21.52
Minimum	15.77	12.20	12.11	18.19	2.97	8.48	15.15	35.16	20.87	9.00

TABLE No. 18.

NORMAL ANNUAL PRECIPITATION; BY DISTRICTS AND ALTITUDES.
(Mean, Maximum, and Minimum Averages.)

Districts.	New England.	Middle Atlantic States.	S. Atlantic States.	Florida Peninsula.	East Gulf States.	West Gulf States.	Ohio Valley and Tenn.	Lower Lake Region.	Upper Lake Region.	North Dakota.	Upper Mississippi Valley.	Missouri Valley.	Northern Slope.	Middle Slope.	Southern Slope.	Southern Plateau.	Middle Plateau.	Northern Plateau.	N. Pacific Coast Region.	Middle Pacific Coast Region.	Southern Pacific Coast Region.
0–100	45.18 42.96 40.73	52.34 45.10 37.86	66.41 60.26 54.12	57.98 48.22 38.46	62.61 59.60 56.59	48.68 39.44 30.20	......	......	......	......	......	......	......	......	......	......	......	......	......	45.83 33.35 20.87	10.51
100–200	47.91 45.08 42.26	43.95 41.89 39.84	48.32	......	......	......	......	......	......	......	......	......	......	......	......	2.97	......	......	62.27 49.85 37.44	45.83 33.35 20.87	
200–400	......	44.80 44.43 44.06	55.51	......	55.66 54.19 52.72	53.63 51.12 48.60	53.28	35.02	......	......	42.83	......	......	......	......	......	......	......	44.63	23.71	21.52 15.26 9.00
400–600	......	......	......	......	......	46.51 45.62 44.74	50.10 47.93 45.76	34.82	......	......	41.08 36.52 31.96	......	......	......	......	......	......	......	35.16	26.11	
600–800	......	42.85	......	......	......	29.70	54.97 48.47 41.97	41.28 36.11 30.93	35.08 32.30 29.53	......	38.01 36.37 34.72										
800–1000	35.74	37.28	51.92	......	......	......	44.38 40.53 36.68	......	34.76	23.77	33.11 30.29 27.47	36.36									
1000–1500	......	......	......	......	51.96	......	50.99	......	......	......	......	45.72 33.37 21.03	......	33.30 29.40 25.49	......	7.21	......	16.77			
1500–2000	......	......	......	......	......	......	......	......	......	18.38 16.54 14.70	......	15.77	......	......	25.02	......	......	18.25			
2000–2500	......	......	......	......	......	......	......	......	......	......	......	......	14.09 13.44 12.80								
2500–3000	......	......	......	......	......	......	......	......	......	......	......	......	18.27	19.84							
3000–3500	......	......	......	......	......	......	......	......	......	......	......	......	16.71		......	......	......	15.15			
3500–4000	......	......	......	......	......	......	......	......	......	......	......	......	......	......	18.19	9.33					
4000–5000	......	......	......	......	......	......	......	......	......	......	......	......	13.18	12.11	......	......	16.19 12.33 8.48	15.27			
5000–6000	......	......	......	......	......	......	......	......	......	......	......	......	13.45	14.49	......	14.25					
6000–7000	......	......	......	......	......	......	......	......	......	......	......	......	12.20								

The total annual rainfall is seldom the same for any two years at the same place, or at any two places for the same year; and its variations seem to follow no fixed law. During a dry season at one place another scarcely 100 miles away may be visited by a wet one. Table No. 21, page 74, illustrates the latter statement very plainly. For instance, in the Eastern Gulf States in 1898 the precipitation at one station was 15.61 inches greater than the average, and at another 12.97 inches less, the average precipitation for that district being 54.25 inches, or less than twice the range of variation. Plate IV illustrates the variations in annual rainfall, as just stated, Philadelphia, Vineland, and New Brunswick all being within a radius of 35 miles. While there is a general similarity in the curves, the variations are seen to be numerous and pronounced, particularly between 1875 and 1885.

The twenty-one districts into which the Weather Bureau has divided the United States for meteorological purposes are indicated by dotted lines on the map, Plate V, while the precipitation averages for these districts are tabulated in Tables No. 17 and No. 18, and the records have been used for other tables also. A comparison of the district records given in Table No. 17 indicates that many combinations of districts might be made, as far as precipitation is concerned, and the discussion of monthly rainfall in the next paragraph shows additional reason for this. The author has found it practicable, by comparing curves of annual and more particularly of monthly precipitation in the various districts, to so group these as to make only eight divisions of the country, as shown in Plate V by the full lines. These divisions will be hereafter referred to as "precipitation districts," the smaller ones as "meteorological districts."

Although in each meteorological district the general law and average amount of precipitation are practically uniform, in some the variation between the precipitation at different

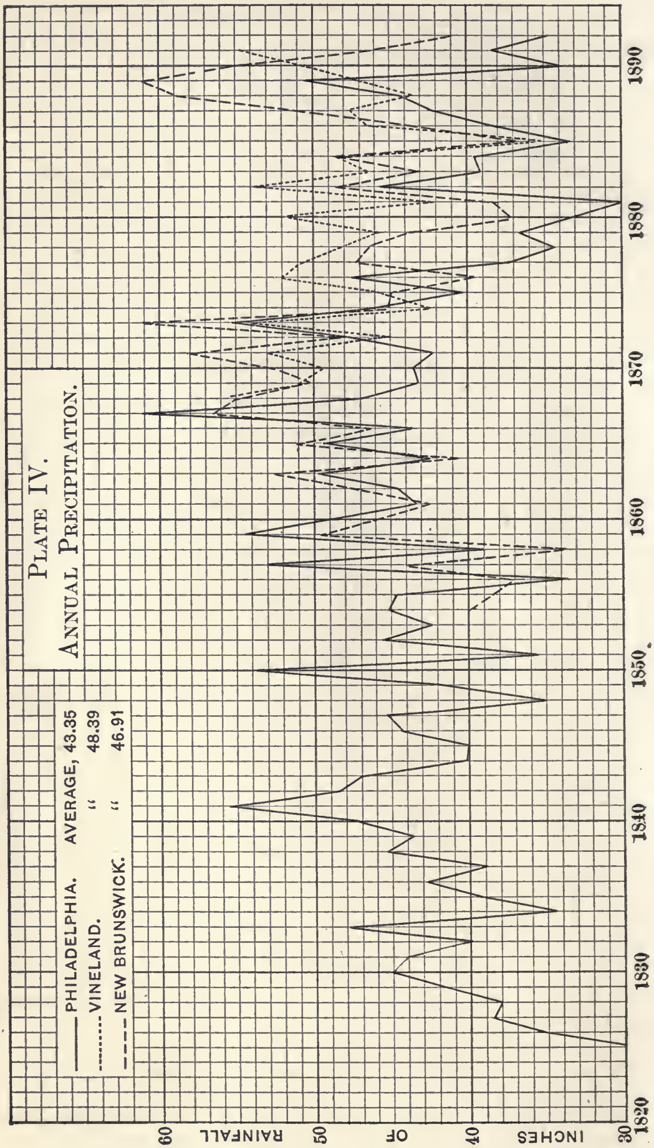
PLATE IV.
ANNUAL PRECIPITATION.
PHILADELPHIA. AVERAGE, 43.35
VINELAND. " 48.39
NEW BRUNSWICK. " 46.91
RAINFALL OF INCHES
60
50
40
30
1820
1830
1840
1850
1860
1870
1880
1890

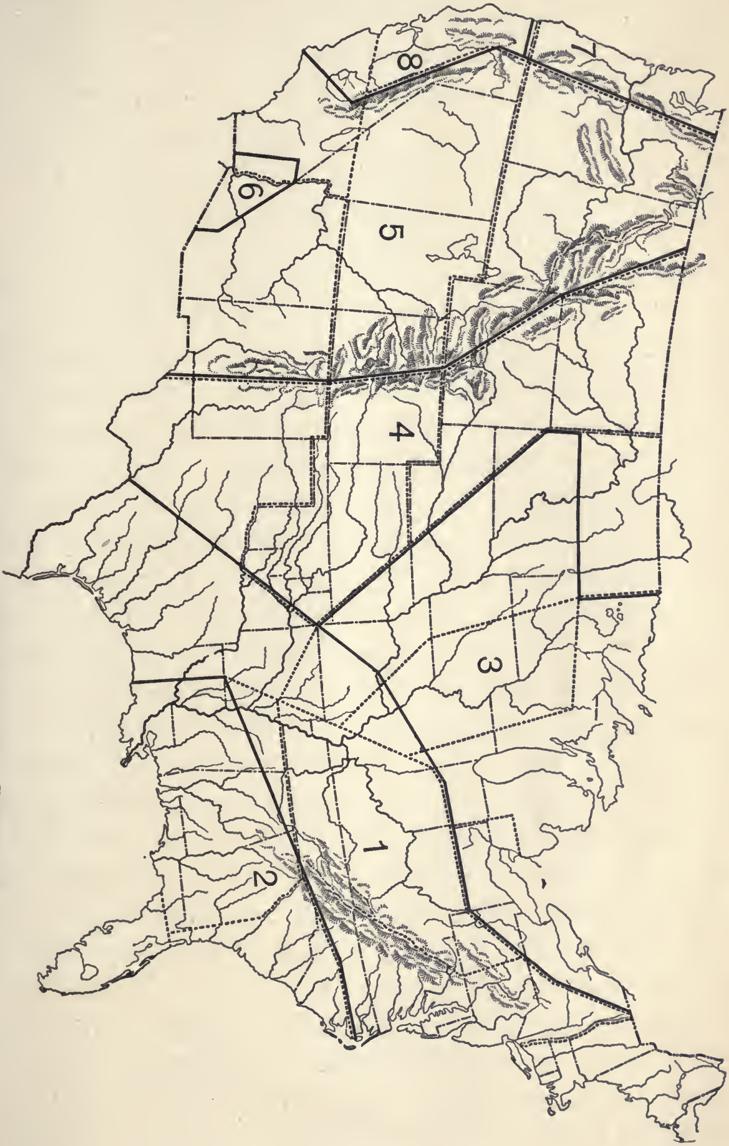

PLATE V.—PRECIPITATION AND METEOROLOGICAL DISTRICTS.

stations is as great as the variation between the extreme district means. For instance, in the Southern Plateau the maximum mean (14.28 inches) is almost five times the minimum (2.97 inches); and in Utah alone the maximum (18.45 inches) is about three times the minimum (6.28 inches). But in most districts the extreme variations from the district average is from 10 to 40 per cent.

The bounds of the precipitation districts are seen to be determined to a large extent by pronounced natural features —as the Pacific Coast Range, the Rocky Mountains, the Mississippi and its western tributaries, its eastern tributaries, and the river-basins of the Gulf of Mexico and South Atlantic. The factor most influential in determining the amount of rainfall in a given district is its position relative to mountain ranges and to the sea or other large body of water. Thus, the warm, moist winds of the North Pacific are robbed of a large part of their moisture by the west slope of the Sierra Nevada and Cascade Range, leaving little for the plateau to their east. The winds blowing over the Gulf Stream into the South Atlantic and Gulf States yield their moisture to them; and as they ascend the valley of the Mississippi and its tributaries their moisture and the consequent rainfall decreases. Above Cape Hatteras the departure of the Gulf Stream from the coast causes its influence to be less felt in precipitation, so that the winds from the Gulf, after passing the low countries of Louisiana, Mississippi, and Alabama, contribute more rain to the Ohio valley than do the winds from the Atlantic to the North Atlantic States.

The statement has been made that precipitation increases with altitude up to about 5000 feet, above which it decreases; but this is true for certain localities only. For example, San Francisco for a number of years had as a minimum annual precipitation 7.4 inches, maximum 49.3 inches, and mean 23 inches; while in the mountains twenty-five miles distant the minimum precipitation was 20 inches, maximum 81 inches, and mean 47 inches. Also the following rates have been found:

	Elevation.	Average Annual Precipitation.
At Sacramento, Cal.........	30 ft.	19.80 in.
" Folsom, "	182 "	23.33 "
" Auburn, "	1360 "	32.55 "
" Colfax, "	2421 "	44.01 "
" Alta, "	3612 "	42.13 "
" Emigrant Camp, Cal.....	5230 "	50.77 "
" Cisco, Cal............	5939 "	57.41 "
" Summit, "	7017 "	47.93 "

F. H. Newell, Hydrographer of the U. S. Geological Survey, judges from data obtained that on the western slope of the Coast Range and Sierra Nevadas "there is a regular increase in the rate of precipitation with altitude, and that this is at the rate of 0.6 in. of rain for each 100 feet increase of elevation." S. A. Hill has shown that in the Northwest Himalayas the precipitation may be represented by the formula

$$1 + 1.92h - 0.40h^2 + 0.02h^3,$$

where h is the number of times less one that 1000 is contained in the elevation, in feet.

Table No. 18, page 62, shows that this increase of precipitation with altitude is not generally true all over the country. Of the twenty-one meteorological districts, in but three do the government records seem to show that this law holds uniformly good, viz., the Upper Lake, Southern Plateau, and Southern Pacific. On the other hand, in nine districts the opposite seems to be true, and the precipitation decreases with the altitude.

It has also been stated that the precipitation decreases in each river-basin with the distance from the river's mouth. This is true of the Connecticut, Hudson, Susquehanna, Mississippi, lower Rio Grande, and Columbia rivers; but is not true of the upper Rio Grande, Gila, Rio Colorado, and others. The last-named rivers lie in the Southern Plateau and Southern Pacific districts, those in which the rainfall generally increases with the altitude. The two statements are apparently one—as we rise above the sea-level and recede from the coast the precipitation becomes less, except in the southwest and possibly along the Great Lakes.

The general statement may also be made that the greatest precipitation is contributed by winds from the Japan Current in the northwest, and from the Gulf Stream in the southeast.

The problem of water-supply is concerned rather with minimum than with maximum rainfall, and the system must be so designed that there shall be no lack during the longest periods of drought. Reference to Plate IV, page 64, shows that the minimum rates do not continue for more than one year, although at times *low* rates continue for three to five consecutive years. These cycles of lowest rates must be provided for according to the engineer's best judgment. When long-period rain-gaugings are available they should be used with judgment; but even those extending over a short period are useful in estimating.

Table No. 19 shows the percentages of the average annual precipitation which represent the minimum average for any two, three, four, and five consecutive years in three cities.

TABLE No. 19.

DRY CYCLES OF FROM TWO TO FIVE YEARS' DURATION.

City.	Length of Record.	Mean Precipitation.	Dryest Year.	Dryest 2 Years.	Dryest 3 Years.	Dryest 4 Years.	Dryest 5 Years.
Boston	74 years	47.00 in.	58%	68%	76¾%	76½%	79½%
Philadelphia.	68 "	43.35 "	68%	73½%	77⅓%	78%	79½%
Denver......	19 "	14.30 "	65%	71%	77%	80%	85½%

There is seen to be a close agreement between the Boston and Philadelphia cycles, and both of these are for much longer periods than the Denver ones. It is of course probable that the longer record will include a dryer season than the shorter one. If we take only the same nineteen years of Boston's and Philadelphia's rainfall covered by the Denver record we obtain the following percentages:

City.	Length of Record.	Mean Precipitation.	Dryest Year.	Dryest 2 Years.	Dryest 3 Years.	Dryest 4 Years.	Dryest 5 Years.
Boston	19 years	49.42 in.	72%	85%	89⅛%	87%	88%
Philadelphia.	19 "	42.40 "	71¼%	75¼%	79%	79⅜%	81⅛%

In each case the nineteen years does not include so dry a period as did the seventy-four and sixty-eight years respectively; an illustration of the fact that allowance should be made, when only short records are available, for dryer periods than those recorded. It is also evident that the percentages in one district cannot be applied to another. Thus, Denver's dry-cycle percentages are lower than those of either Boston's or Philadelphia's for the same period, except for the five-year cycle.

The greater the number of years over which a precipitation record extends the closer will the mean obtained be to the true mean. As an illustration, the mean precipitation at Philadelphia for 15 years, beginning with 1825, was 40.32 inches; for 30 years it was 42.45 inches; for 45 years, 43.80 inches; for 60 years, 43.18 inches; and for 68 years, 43.35 inches. Hence any mean precipitation obtained for any place will only approximate the true mean; more closely, however, the longer the period over which the record extends. In certain small sections of the country one or even two years may pass without any rain whatever falling.

ART. 25. QUANTITY OF RAINFALL: MONTHLY.

For many purposes it is desirable to know the monthly precipitation as well as the annual; and in this a general law is in most cases observable, although the law differs in different localities. Table No. 20 shows the mean precipitation for each month in each of the meteorological districts; and Table No. 21 shows the monthly variations in each of these districts. The former gives the average precipitation of all stations for each month and for the year, and also the maximum average and minimum average found for any single station in each district. The maximum and minimum precipitation for any one month or year would be considerably greater and less, respectively, than these figures. In Table No. 21 are given the monthly variations in each of the meteorological districts for the year 1898, which was believed to be a fair average year as regards rainfall. This table shows, for each district, the maximum variation by which the monthly precipitation at any one station was greater or less than the *average* precipitation at *that* station for that month of the year. + signifies that the precipitation was greater than the average; — that it was less. Where the precipitation at all stations in any one district varied on the same side of their averages, the maximum and minimum variations are given. This table illustrates the impossibility of accurately predicting the precipitation for any month, and the error involved in assuming that all places in the same district are subject to equal variations; since more than 77% of the monthly records show variations in different directions in the same district.

There is, however, a general law for each district which is subject to only occasional exceptions. Thus, in New England June and September are the months of least precipitation;

not only as regards the district mean, but the maximum local means are less for these than for other months, as are also the + variations in Table No. 21. A comparison of these laws for the various districts shows great similarity among certain groups, and these groups have been made the basis of the subdivision into precipitation districts already referred to. These districts are (see Plate V, page 65):

1. The North Atlantic, Ohio, and West Gulf.
2. South Atlantic and Gulf.
3. Upper Mississippi and Great Lakes.
4. Eastern Slope.
5. Northern and Middle Plateau.
6. Southern Plateau.
7. North Pacific.
8. Middle Pacific.

The mean rainfall, monthly and annual, for these districts is shown in Table No. 22, page 76, with the greatest percentage by which any one local average varies from the district average.

Except in the case of the fourth district the variations in these Precipitation Districts are little greater than those in the Meteorological Districts, and the reduction in number makes reference more easy. This division is also in accord with the more important laws of precipitation.

In Table No. 23, page 77, is shown the mean monthly precipitation for four cities, together with the maximum and minimum recorded for each month.

It will be seen from this table that at Philadelphia and Boston the rainfall has, during each month in some year, been less than 1 inch; at Charleston, although the average annual rainfall is less than at these two places, the minimum monthly precipitation is greater for every month but one; while at Denver the minimum monthly precipitation is but $2\frac{1}{2}$ to $22\frac{5}{8}$ per cent of the average for the same month.

TABLE No. 20.

MEAN, MAXIMUM, AND MINIMUM MONTHLY PRECIPITATION AVERAGES.

(Weather Bureau Records to the close of 1898.)

District.	No. of Stations in District.	January.			February.			March.			April.			May.			June.		
		Mean	Max.	Min.	Mean	Max.	Min.	Mean	Max.	Min.	Mean	Max.	Min.	Mean	Max.	Min.	Mean	Max.	Min.
New England	10	4.0	5.3	3.1	3.5	4.5	2.1	3.9	4.7	2.4	3.3	4.0	2.1	3.6	4.0	3.1	2.9	3.5	2.2
Middle Atlantic States	11	3.6	4.3	2.9	3.4	3.9	2.7	3.9	5.2	3.2	3.3	4.5	2.3	3.6	4.7	2.7	3.7	4.3	3.1
South Atlantic States	9	4.3	5.9	3.2	3.8	4.5	3.1	4.5	6.0	3.4	3.4	4.7	2.6	4.0	5.7	2.9	5.6	6.6	4.1
Florida Peninsula	3	2.7	3.5	2.1	2.3	2.8	1.6	2.2	3.0	1.2	1.8	1.2	2.4	4.0	5.9	2.9	6.0	7.6	4.0
East Gulf States	6	5.2	5.6	4.4	4.4	5.3	3.3	5.9	7.6	3.8	4.4	5.8	3.0	4.1	4.9	3.2	5.0	6.6	4.2
West Gulf States	7	3.5	4.9	1.7	3.4	5.3	2.0	3.4	5.2	1.6	3.9	5.2	1.6	4.4	5.8	3.2	3.8	4.8	2.6
Ohio Valley and Tennessee	11	4.2	6.6	2.9	4.2	5.6	2.8	4.3	6.0	2.8	4.0	5.5	2.8	3.9	4.5	3.4	4.2	4.8	3.5
Lower Lake Region	8	2.7	3.3	1.9	2.7	3.4	2.0	2.6	2.8	2.0	2.3	2.5	2.1	3.4	3.9	2.8	3.6	4.0	3.2
Upper Lake Region	9	2.1	2.6	1.1	1.9	2.4	1.1	2.1	2.6	1.2	2.4	3.1	1.6	3.4	3.8	2.2	3.7	4.6	3.1
North Dakota	3	0.6	0.7	0.6	0.6	0.8	0.3	0.8	1.9	0.5	1.9	2.3	1.4	2.3	2.5	2.1	3.8	4.3	3.5
Upper Mississippi Valley	11	1.8	3.9	0.8	2.0	4.0	0.9	2.3	3.8	1.5	3.0	3.9	2.2	4.2	5.0	3.3	4.5	5.2	3.8
Missouri Valley	10	1.1	2.4	0.5	1.4	3.5	0.3	1.8	3.3	0.8	3.0	4.6	1.9	4.3	6.1	2.4	4.4	5.7	3.4
Northern Slope	7	0.7	1.4	0.6	0.5	0.7	0.4	0.8	1.3	0.5	1.6	2.3	1.0	2.4	3.7	1.6	2.6	3.9	1.2
Middle Slope	6	0.9	1.9	0.4	0.8	1.2	0.5	1.5	2.0	0.5	2.0	2.9	1.2	3.6	5.3	1.8	3.1	5.2	1.2
Southern Slope	2	0.8	0.9	0.6	1.4	1.4	1.2	0.9	1.2	0.6	1.8	2.7	0.9	2.8	3.6	2.0	3.2	3.2	3.2
Southern Plateau	6	0.5	0.6	0.4	0.8	0.9	0.4	0.5	0.7	0.3	0.3	0.8	0.1	0.4	1.0	0.0	0.3	0.9	0.0
Middle Plateau	3	1.7	2.5	1.1	1.2	1.5	0.9	1.4	2.0	0.8	1.3	2.2	0.8	1.1	1.7	0.6	0.6	0.8	0.4
Northern Plateau	4	2.3	2.6	1.6	1.7	2.0	1.3	1.8	2.2	1.4	1.4	1.6	1.2	1.6	1.9	1.1	1.4	1.7	1.1
Northern Pacific Coast Region	9	8.1	12.7	5.3	6.0	8.5	3.0	5.4	9.1	2.2	4.3	7.4	2.0	2.7	4.6	1.3	2.3	4.1	1.2
Middle Pacific Coast Region	5	5.5	8.4	3.8	4.1	6.1	3.2	4.0	6.5	2.9	2.4	4.2	1.7	1.6	3.0	0.7	0.5	1.3	0.1
Southern Pacific Coast Region	4	2.8	4.7	1.3	2.6	3.8	1.1	2.2	3.0	1.3	1.3	2.0	0.7	0.4	0.5	0.3	0.1	0.1	0.0
West Indies (one year only)	....	....	....	....	....	....	....	...	...	...	...	...	...	...	...	...	....	...	...

TABLE No. 20.—(*Continued.*)

District.	No. of Stations in District.	July.			August.			September.			October.			November.			December.		
		Mean	Max.	Min.	Mean	Max.	Min.	Mean	Max.	Min.	Mean	Max.	Min.	Mean	Max.	Min.	Mean	Max.	Min.
New England...............	10	3.4	4.9	2.4	4.0	4.6	3.2	3.2	3.8	2.9	4.0	4.8	2.4	4.0	4.8	3.3	3.6	4.2	3.1
Middle Atlantic States.......	11	4.3	5.9	3.2	4.6	6.1	4.0	3.8	4.6	3.3	3.1	3.8	2.6	3.4	5.5	2.3	3.3	4.3	2 7
South Atlantic States........	9	6.4	7.9	5.3	6.8	7.9	5.2	5.3	8.5	3.3	4.0	6.1	2.4	3.1	5.2	2.1	3.5	5.4	2.7
Florida Peninsula...........	3	6.1	9.8	4.7	6.4	9.4	4.7	7.8	9.7	6.2	5.6	9.2	2.4	2.6	3.4	2.0	2.2	2.7	1.6
East Gulf States.............	6	6.0	8.4	4.5	5.9	8.4	3.5	4.5	6.0	2.9	3.0	3.9	2.3	3.8	4.9	3.4	4.3	4.9	3.8
West Gulf States	7	3.0	4.4	1.3	3.7	5.6	2.2	4.0	6.2	3.1	2.8	3.2	1.6	4.0	5.3	2.0	3.3	4.7	1.3
Ohio Valley and Tennessee ..	11	4.0	4.8	3.2	3.6	4.2	3.2	3.0	4.1	2.4	2.6	3.1	2.4	3.7	4.6	2.5	3.5	4.4	2.9
Lower Lake Region..........	8	3.1	3.5	2.7	3.0	3.3	2.7	2.9	3.9	2.4	3.0	4.0	2.4	3.2	4.2	2.7	2.8	3.3	2.4
Upper Lake Region..........	9	3.0	3.7	2.4	2.9	3.4	2.6	3.4	4.4	2.6	3.0	3.8	2.4	2.4	3.0	1.6	2.2	2.6	1.4
North Dakota	3	2.8	3 9	2.1	1.9	2.7	1.1	1.3	2.1	0.8	1.4	1.9	1.0	0.7	0.9	0.5	0.7	0.7	0.6
Upper Mississippi Valley.....	11	3.7	4.2	2.7	3.1	3.6	2.0	3.2	4.2	2.5	2.5	3.1	1.8	2.1	4.2	0.7	2.0	3.4	1.2
Missouri Valley	10	4.1	5.2	2.2	3.2	4.5	1.7	2.6	4.1	1.0	1.9	3.4	0.7	1.5	3.6	0.5	1.2	2.9	0.5
Northern Slope..............	7	1.7	2.2	0.8	1.4	2.4	0.8	1.0	1.3	0.6	0.9	1.0	0.7	0.5	0.7	0.3	0.5	0.9	0.3
Middle Slope	6	2.9	3.9	1.7	2.6	3.6	1.5	1.8	2.9	0.4	1.4	1.8	0.7	0.9	1.9	0.3	0.9	2.4	0.5
Southern Slope	2	1.9	2.2	1.7	2.8	2.9	2.6	2.3	2.4	2.1	1.8	2.3	1.4	0.8	1.4	0.3	1.1	1.4	0.7
Southern Plateau............	6	1.2	2.8	0.1	1.2	1.8	0.2	0.8	1.5	0.0	0.6	1.0	0.3	0.5	0.8	0.2	1.0	2.0	0.5
Middle Plateau..............	3	0.3	0.6	0.1	0.3	0.8	0.1	0.6	1.0	0.3	0.9	1.6	0.4	1.1	1.5	0.1	1.7	2.2	1.2
Northern Plateau............	4	0.5	0.7	0.3	0.3	0.4	0.2	1.3	2.0	0.6	1.3	1.5	1.1	1.3	1 7	0.9	2.0	2.5	1.6
Northern Pacific Coast Region	9	0.8	1.2	0.4	1.0	2.7	0.2	3.2	7.3	1.0	5.3	10.2	2.7	7.5	14.9	3.7	8.9	16.4	5.5
Middle Pacific Coast Region..	5	0.1	0.3	0.0	0.0	0.0	0.0	0.7	1.3	0.3	1.7	2.5	0.9	3.0	4 0	2.1	5.6	8.2	3.7
Southern Pacific Coast Region	4	0.0	0.0	0.0	0.0	0.1	0.0	0.2	0.2	0.1	0.6	1.0	0.4	1.3	2.0	0.8	3.1	4.5	1.5
West Indies (one year only)..	....	....	....	....	...	...	...	10.16	19.7	2.6	5.9	12.1	0.9	7.6	12.6	4.1	3.3	7.7	0.1

Table No. 21.

MONTHLY VARIATIONS IN PRECIPITATION IN EACH METEOROLOGICAL DISTRICT IN 1898.

(Maximum + or − Variation of any Station from its own Average.)

District.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Year.
New England	+ 3.0	+ 4.1	− 2.3	+ 3.7	+ 4.9	+ 0.6	+ 4.7	+ 4.2	+ 0.4	+ 4.4	+ 7.3	− 0.4	+ 17.91
	− 1.0	− 1.0	− 0.1	− 0.6	− 1.6	− 2.7	− 3.2	− 1.3	− 2.8	− 0.3	− 1.3	− 2.0	− 9.30
Mid. Atlantic	+ 0.7	+ 0.9	− 1.6	+ 2.6	+ 5.2	+ 1.9	+ 3.3	+ 4.8	+ 0.7	+ 4.1	+ 4.0	+ 1.3	+ 9.39
	− 2.6	− 2.8	− 0.2	− 1.6	− 0.3	− 3.0	− 2.9	− 2.1	− 2.8	+ 0.5	− 1.3	− 2.3	− 7.49
S. Atlantic	− 3.8	− 3.7	+ 1.1	+ 2.1	+ 1.8	+ 0.2	+ 6.6	+ 15.0	+ 0.7	+ 6.3	+ 4.0	+ 1.9	+ 8.27
	− 1.2	0.0	− 3.6	− 1.1	− 3.4	− 3.3	− 2.1	− 3.7	− 5.1	− 1.9	− 1.1	− 2.1	− 18.21
Florida	− 3.1	− 1.6	+ 1.0	− 0.5	+ 0.1	− 1.6	+ 2.1	+ 8.4	+ 0.4	+ 11.7	0.0	+ 1.6	+ 4.93
	− 1.8	− 1.3	− 2.9	− 1.7	− 2.5	− 6.5	− 1.6	+ 1.0	− 6.3	+ 1.7	− 2.3	− 0.1	− 18.88
E. Gulf	+ 2.2	+ 2.3	− 5.7	+ 1.4	− 2.2	+ 1.9	+ 5.8	+ 10.2	+ 13.1	+ 10.5	+ 3.7	+ 3.1	+ 15.61
	− 3.8	− 4.3	− 1.0	− 3.0	− 4.9	− 3.4	− 1.9	− 0.7	− 1.8	− 1.4	− 0.2	− 2.3	− 12.97
W. Gulf	+ 3.2	+ 1.2	+ 7.3	+ 0.8	+ 2.8	+ 4.4	+ 0.5	+ 2.4	+ 7.1	+ 2.6	+ 1.2	+ 1.4	+ 6.38
	− 2.2	− 4.1	− 2.6	− 3.2	− 3.0	− 2.9	− 2.1	− 3.1	− 2.4	− 3.4	− 2.4	− 2.9	− 10.88
O. Val. and Ten.	+ 5.4	− 4.9	+ 6.3	+ 0.2	+ 2.7	+ 3.7	+ 2.4	+ 3.9	+ 4.7	+ 2.8	− 0.2	− 0.6	+ 16.14
	− 1.2	− 0.8	− 3.3	− 3.2	− 3.5	− 2.9	− 2.2	− 3.4	− 1.5	0.0	− 1.7	− 2.4	− 14.50
Lower Lake	+ 2.2	+ 0.9	+ 3.0	+ 0.2	+ 1.3	+ 1.5	+ 2.6	+ 2.1	+ 1.6	+ 2.3	+ 0.7	+ 0.2	+ 8.26
	+ 0.1	− 1.4	− 0.6	− 1.1	− 1.9	− 2.3	− 2.0	− 1.3	− 1.3	− 1.0	− 0.7	− 0.5	− 7.20
Upper Lake	+ 1.4	+ 1.5	+ 2.2	+ 0.4	+ 1.1	+ 2.0	+ 0.2	+ 2.1	+ 0.5	+ 2.0	+ 0.4	+ 0.8	+ 1.54
	− 1.2	− 1.2	− 0.9	− 2.3	− 1.9	− 1.6	− 2.4	− 1.7	− 2.6	− 1.0	− 1.7	− 1.8	− 11.31
N. Dakota	− 0.4	+ 0.4	+ 0.8	+ 0.2	+ 0.6	+ 1.1	− 0.2	0.0	+ 0.2	+ 1.6	− 0.2	− 0.2	− 4.71
	− 0.6	− 0.3	− 0.1	− 1.2	− 0.9	− 2 3	− 1.4	− 0.9	− 0.5	− 0.2	− 0.5	− 0.6	− 0.26
Upper Mississippi	+ 3.8	+ 0.8	+ 7.0	+ 1.6	+ 4.0	+ 1.5	+ 3.7	+ 4.1	+ 6.4	+ 3.9	+ 0.8	− 0.4	+ 18.27
	− 0.9	− 2.8	− 0.1	− 1.8	− 2.2	− 2.7	− 2.3	− 2.6	− 2.6	+ 0.3	− 2.7	− 2.1	− 7.19

TABLE No. 21.—(*Continued.*)

District.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Year.
Missouri	+ 2.9	+ 0.2	+ 5.7	+ 3.0	+ 3.1	+ 4.5	+ 2.5	+ 1.6	+ 2.7	+ 1.4	+ 0.5	− 0.1	+ 13.91
	− 0.6	− 0.7	− 0.4	− 1.9	− 0.1	− 2.9	− 2.6	− 2.3	− 1.6	− 0.5	− 1.2	− 1.5	− 5.41
North. Slope.......	+ 0.3	− 0.6	+ 1.8	+ 0.5	+ 3.3	+ 1.8	+ 0.6	+ 0.1	+ 1.3	+ 1.5	+ 1.2	+ 0.2	+ 4.54
	− 1.2	− 0.2	− 0.6	− 1.2	− 0.5	− 3.4	− 1.4	− 1.3	− 0.4	− 0.9	− 0.5	− 0.6	− 5.73
Mid. Slop..........	+ 2.5	+ 1.8	0.0	+ 2.9	+ 7.1	+ 1.2	+ 2.2	− 0.3	+ 1.8	+ 0.6	+ 0.5	+ 1.1	+ 11.62
	− 0.4	− 0.6	− 1.0	− 1.9	− 1.6	− 1.4	− 1.6	− 1.9	− 1.2	− 0.7	− 1.0	− 0.2	− 5.16
South. Slope.......	+ 0.3	− 0.4	+ 0.2	+ 0.1	+ 1.5	+ 1.6	+ 1.7	+ 1.1	+ 1.0	− 1.0	0.0	+ 1.3	+ 4.35
	− 0.2	− 0.3	− 0.2	− 0.9	− 1.0	+ 1.3	− 0.2	− 0.7	− 1.6	− 2.3	− 0.4	+ 0.7	− 2.89
S. Plateau.........	+ 1.1	− 0.9	+ 0.2	+ 0.7	0.0	+ 0.6	− 0.1	+ 1.4	+ 0.1	− 0.3	+ 0.6	+ 0.7	− 3.17
	− 0.3	− 0.4	− 0.6	− 0.1	− 0.8	0.0*	− 0.6	− 0.8	− 1.3	− 0.9	− 0.5	− 1.8	− 0.59
Mid. Plateau	− 2.0	− 0.9	− 0.1	− 0.4	+ 2.5	+ 0.7	− 0.1	+ 0.6	− 0.1	+ 0.3	+ 0.7	− 0.2	− 6.08
	− 0.5	− 0.5	− 0.3	− 0.9	− 0.2	− 0.7	− 0.4	0.0	− 0.8	0.0	+ 0.1	− 2.1	− 0.10
N. Plateau.........	− 1.1	− 1.6	− 0.3	+ 0.2	+ 0.6	+ 0.2	+ 0.5	+ 0.7	− 0.3	− 0.4	+ 2.3	− 0.8	− 6.60
	− 0.6	0.0	− 1.5	− 1.3	+ 0.2	− 0.8	− 0.3	− 0.1	− 0.6	− 1.2	0.0	− 1.6	− 0.43
N. Pacific..........	− 5.7	+ 9.0	− 0.8	− 0.7	− 0.1	+ 1.7	+ 0.2	+ 0.7	+ 2.1	+ 1.2	+ 2.5	0.0	− 12.93
	− 1.9	− 3.0	− 5.6	− 2.0	− 2.2	− 0.4	− 0.8	− 2.6	− 2.4	− 2.5	− 1.3	− 3.7	− 2.43
Mid. Pacific........	− 5.2	+ 1.9	− 2.8	− 1.4	+ 1.0	0.0*	0.0*	0.0*	+ 0.8	− 0.3	+ 0.4	− 1.8	− 16.52
	− 2.8	− 1.6	− 4.7	− 2.0	− 0.4	− 0.4	− 0.1	− 0.2	− 0.2	− 1.0	− 2.5	− 5.0	− 10.83
S. Pacific	− 3.3	− 2.8	− 0.6	− 0.5	+ 1.3	0.0*	0.0*	0.0*	+ 0.9	− 0.4	− 0.6	− 1.1	− 14.59
	− 0.3	0.0	− 2.1	− 1.9	+ 0.3	− 0.1	0.0*	− 0.1	− 0.1	− 0.6	− 1.9	− 3.9	− 4.01
United States......	− 5.7	+ 9.0	+ 7.3	+ 3.7	+ 7.1	+ 4.5	+ 6.6	+ 15.0	+ 13.1	+ 11.7	+ 7.3	+ 3.9	+ 18.27
	+ 5.4	− 4.9	− 5.7	− 3.2	− 4.9	− 6.5	− 3.2	− 3.7	− 6.3	− 3.4	− 2.7	− 5.0	− 18.88

* No rain whatever

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Table No. 22.

MEAN MONTHLY PRECIPITATION, BY PRECIPITATION DISTRICTS, AND PERCENTAGES OF MAXIMUM RANGE OF VARIATION.

District Number.	Jan.		Feb.		Mar.		April.		May.		June.		July.	
	Rate.	Variation.	Rate.	Variation.	Rate.	Variation.	Rate.	Variation.	Rate.	Variation.	Rate.	Variation.	Rate.	Variation.
1	3.9	69	3.7	51	3.9	59	3.6	55	3.8	52	3.7	41	3.8	66
2	4.3	51	3.8	56	4.6	74	3.0	93	4.0	47	5.5	38	6.2	58
3	1.9	100	2.0	95	2.2	73	2.7	70	3.9	56	4.1	39	3.5	49
4	0.8	137	0.7	171	1.4	64	1.8	61	2.8	89	3.0	73	2.3	70
5	2.0	45	1.5	40	1.6	50	1.4	57	1.4	57	1.0	80	0.4	75
6	0.5	20	0.8	50	0.5	40	0.3	167	0.4	100*	0.3	200*	1.2	138
7	8.1	57	6.0	50	5.4	68	4.3	72	2.7	70	2.3	77	0.8	50
8	5.5	53	4.1	49	4.0	62	2.4	75	1.6	88	0.5	160	0.1	200
District Number.	**Aug.**		**Sept.**		**Oct.**		**Nov.**		**Dec.**		**Year.**			
	Rate.	Variation.	Rate.	Variation.	Rate.	Variation.	Rate.	Variation.	Rate.	Variation.	Rate.	Variation.		
1	4.0	52	3.5	77	3.1	51	3.7	49	3.4	62	44.0	− 32		
2	6.4	47	5.4	80	3.9	136	3.3	58	3.5	54	53.4	− 28		
3	3.1	45	3.0	67	2.6	73	2.3	83	2.0	75	33.0	− 52		
4	2.0	80	1.5	93	1.3	77	0.7	171	0.7	243	18.6	− 35 + 79		
5	0.3	166	1.0	100	1.1	63	1.2	92	1.9	32	14.6	− 42		
6	1.2	83	0.0	100*	0.6	67	0.5	60	1.0	100	8.4	− 65		
7	1.0	87	3.2	128	5.3	92	7.5	99	8.9	84	45.3	− 22 + 38		
8	0.0	....	0.7	86	1.7	47	3.0	33	5.6	46	29.8	− 38 + 54		

* Never any rain during these months in certain localities.

When several months of low precipitation follow each other consecutively, this fact will have considerable bearing upon the question of storage. In none of the records from which the above were taken did two months of minimum precipitation follow each other; in many cases such months were immediately preceded and followed by months of heavy rainfall; and in the majority of cases the average of any three consecutive months including the minimum was at least two-thirds of the monthly average for that year. There are in the Boston record, however, several instances where the average of six consecutive months, and in the Charleston

TABLE NO. 23.

MEAN, MAXIMUM, AND MINIMUM PRECIPITATION, BY MONTHS, IN VARIOUS CITIES.

		Duration of Record. Years.	Jan.	Feb.	March.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Year.
Boston, Mass......	Mean	74	3.98	3.78	4.36	4.06	3.79	3.27	3.71	4.39	3.55	3.84	4.31	3.96	47.00
	Max.		8.84	9.98	11.75	10.83	10.38	8.01	12.38	12.10	11.95	8.78	11.63	9.04	67.72
	Min.		0.35	0.80	0.96	0.20	0.25	0.30	0.85	0.34	0.23	0.80	0.81	0.26	27.20
Philadelphia, Pa...	Mean	68	3.45	3.08	3.53	3.43	3.78	3.86	4.12	4.50	3.54	3.28	3.39	3.41	43.35
	Max.		7.84	6.64	6.71	9.76	7.42	10.95	11.81	16.84	12.09	10.05	7.96	6.26	61.29
	Min.		0.73	0.82	0.69	0.59	0.19	0.74	0.96	0.62	0.20	0.30	0.80	0.26	29.57
Charleston, W. Va.	Mean	11	3.50	3.88	3.69	3.27	4.17	3.98	3.44	4.78	3.16	2.63	2.84	2.73	42.07
	Max.		5.15	8.10	8.94	4.99	6.94	7.74	6.31	9.05	6.29	6.30	5.37	5.40	59.23
	Min.		1.88	2.08	1.57	1.97	2.08	1.40	0.42	1.56	1.16	0.59	1.16	1.60	32.82
Denver, Col.......	Mean	19	0.58	0.49	0.88	2.07	2.77	1.38	1.67	1.54	0.81	0.81	0.68	0.63	14.30
	Max.		2.35	1.22	2.36	4.94	8.57	4.98	4.32	2.68	2.89	2.15	1.93	2.32	20.12
	Min.		0.10	0.11	0.20	0.05	0.09	0.09	0 33	0.33	0.02	0.12	0.08	0.04	9.33

record several where the average of four consecutive months, is less than one-twelfth the minimum annual rainfall; while in Denver there are, each year, six months of light and six of heavy rainfall. In Massachusetts the mean precipitation is almost exactly the same—11.75 inches—for each of the four seasons, spring, summer, autumn, and winter. These three records fairly well represent three sections of the country.

Table No. 24 (from Vermeule's "Report on Water-supply, Geological Survey of New Jersey") shows the monthly rainfall during two typical dry periods in New York and Philadelphia; while Table No. 25 shows the four dryest periods of from three to twelve months' duration each, in the same cities.

TABLE No. 24.

TYPICAL DRY PERIODS.

PHILADELPHIA RECORD.

	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Year.
1841–1842	5.92	1.36	4.27	2.84	5.31	5.87	3.19	11.81	3.79	1.27	1.72	3.49	50.84
1842–1843	3.66	1.44	2.54	4.42	4.72	2.05	1.69	4.54	9.26	4.86	3.22	4.18	46.58
1879–1880	4.69	1.51	2.43	3.53	2.43	0.54	1.67	7.74	5.09	1.10	1.74	1.75	34.22
1880–1881	4.05	3.66	4.76	3.83	0.61	2.71	3.87	0.96	1.18	0.94	3.04	2.02	31.63

NEW YORK RECORD.

	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Year.
1841–1842	2.70	1.07	2.85	1.25	3.60	3.60	3.30	3.80	2.81	2.10	4.30	1.80	33.18
1842–1843	3.50	1.00	2.31	2.13	2.14	1.00	0.76	1.64	15.26	3.c6	5.91	2.82	41.53
1879–1880	4.94	2.02	2.12	4.66	2.90	0.62	1.14	8.53	5.26	1.85	2.81	2.46	39.31
1880–1881	2.27	4.80	4.93	5.81	0.95	3.20	5.35	1.25	0.86	0.97	1.60	2.36	34.35

For large areas and where storage-reservoirs are used, the monthly and annual precipitation serve all purposes, except the calculation of storm overflow to be allowed for; but for this purpose, and for the immediate collection of rainfall in cisterns and on small watersheds, it is desirable to know the maximum rates of precipitation for short periods. Table

TABLE NO. 25.

DRY PERIODS. NEW YORK AND PHILADELPHIA RAINFALL.

	Date.	Duration, Months.	Total Rainfall.	Rainfall per Month.
New York..............	Aug. 1842–July 1843	12	25.48	2.12
"	Jan.–Dec. 1836	12	27.57	2.30
Philadelphia............	" " 1825	12	29.57	2.46
New York..............	" " 1840	12	29.80	2.48
"	Sept. 1842–July 1843	11	22.67	2.06
"	Jan.–Nov. 1836	11	25.27	2.30
Philadelphia............	" " 1825	11	25.85	2.35
New York..............	Feb.–Dec. 1840	11	27.96	2.54
"	Oct. 1842–July 1843	10	20.57	2.06
Philadelphia............	March–Dec. 1881	10	21.79	2.18
New York..............	Jan.–Oct. 1836	10	23.37	2.34
Philadelphia............	" " 1825	10	24.49	2.45
New York..............	Nov. 1842–July 1843	9	16.27	1.81
Philadelphia............	April–Dec. 1881	9	17.96	1.99
"	Oct. 1879–June 1880	9	18.59	2.07
New York..............	Jan.–Sept. 1849	9	20.22	2.25
"	Dec. 1842–July 1843	8	14.47	1.81
Philadelphia............	April–Nov. 1881	8	15.32	1.91
New York..............	" " "	8	16.54	2.07
Philadelphia............	Oct. 1879–May 1880	8	16.92	2.11
New York..............	Jan.–July 1843	7	11.97	1.71
Philadelphia............	April–Oct. 1881	7	13.30	1.90
New York..............	Jan.–July 1849	7	14.04	2.01
"	April–Oct. 1881	7	14.18	2.03
"	Feb.–July 1843	6	9.97	1.66
Philadelphia............	April–Sept. 1881	6	10.26	1.71
New York..............	July–Dec. 1881	6	11.22	1.87
Philadelphia............	Jan.–July 1880	6	12.11	2.02
New York..............	July–Nov. 1881	5	7.04	1.41
"	March–July 1843	5	7.67	1.53
"	Jan.–May 1836	5	7.70	1.54
Philadelphia............	July–Nov. 1881	5	8.14	1.63
New York..............	July–Oct. 1881	4	4.68	1.17
"	April–July 1843	4	5.54	1.39
Philadelphia............	July–Nov. 1881	4	6.12	1.53
New York..............	April–July 1849	4	6.30	1.58
Philadelphia............	July–Sept. 1881	3	3.08	1.03
New York..............	" " "	3	3.08	1.03
"	May–July 1843	3	3.40	1.13
"	Jan.–March 1836	3	4.41	1.47

No. 26 gives the maximum recorded rates in various localities, which will probably be exceeded only at long intervals, if at all.

Table No. 26.

MAXIMUM AMOUNTS OF RAIN FALLING DURING DIFFERENT PERIODS OF TIME.

Length of Period in Minutes.										Place and Date.
								Over 60.		
5	10	15	20	25	30	45	60	Amt.	Duration.	
....	....	....	....	....	....	....	0.50	5.20	24 h.	Boston, Oct. 12, 1895
....	1.40	....	1.50	....	....	....	....	....	....	" } 1879–1891
....	0.80	....	1.30	....	....	....	....	....	....	"
....	0.70	....	....	....	....	....	....	....	....	" July 18, 1884
....	....	....	....	....	0.50	....	....	....	....	Providence, R. I., May 18, 1877
....	....	....	....	....	1.50	....	....	....	....	" " Aug. 29, 1877
....	....	....	....	....	....	....	2.56	....	....	" " " 6, 1878
....	....	0.75	....	....	....	....	....	....	....	" " " 28, 1882
....	0.90	....	....	....	....	....	1.74	....	....	Ithaca, N. Y., Aug. 4, 1892. Preceded by 5 hours of light rain
....	....	....	....	....	....	....	....	5.03	1½ h.	Mt. Carmel, N. Y., July 2, 1897
....	....	....	....	....	....	....	....	1.73	1½ h.	Morrisania, Oct. 30, 1866
....	....	....	0.50	....	....	....	....	1.18	2 h.	New York City, Sept. 19, 1894
0.35	....	....	....	....	....	....	1.40	....	....	" " " Aug. 19, 1893
....	1.20	....	....	....	....	....	....	....	....	" " " 7 times during 1869–1891
....	0.80	....	1.30	....	1.50	....	....	....	....	" " " 3 " " " "
....	0.60	....	....	....	....	....	....	....	....	" " " 5 " " " "
....	....	....	....	....	....	....	....	3.60	24 h.	" " " May 4, 1893
....	....	....	....	....	....	....	....	6.17	24 h.	" " " 1882 [sewer)
....	....	....	0.80	....	....	....	1.00	....	....	" " " July 6, 1896. (Gorged a
....	....	....	....	2.60	....	....	....	....	....	Brooklyn, N. Y.
....	1.00	....	....	....	....	....	....	....	....	Spring Mount, Pa., June 6, 1893
0.25	1.00	....	1.50	....	0.45	....	....	....	....	" " " during 1893 (4 storms)
....	1.00	1.28	1.60	....	....	....	....	....	....	Philadelphia, Pa., 2 times during 1884–1891
....	0.80	....	1.30	....	....	....	....	....	....	" " 3 " " " "
....	0.60	....	....	....	1.00	....	....	....	....	" " 5 " " " "
....	0.40	....	....	....	0.96	....	....	....	....	" " March, 1890
0.21	0.35	0.46	0.58	0.62	0.64	0.83	0.95	....	....	Baltimore, 1896
0.25	0.60	0.85	0.92	0.95	....	....	....	....	....	" "
0.80	1.60	....	....	....	2.00	....	....	....	....	Washington, D. C., 2 times during 1871–1891
....	0.80	....	1.30	....	1.50	....	....	....	....	" " 5 " " " "
....	0.60	....	....	....	....	....	....	....	....	" " 10 " " " "
0.52	....	0.75	....	1.78	....	2.00	....	....	....	" "
0.50	0.85	1.20	1.48	....	2.05	2.70	3.40	6.00	2 h.	" " mean of many rains
0.40	0.70	1.00	1.32	1.61	1.87	2.62	3.19	3.30	2 h.	New Orleans, June 17, 1895
0.40	0.75	1.20	1.82	1.95	2.15	2.45	2.70	....	....	" " Aug. 13, 1894
0.60	1.20	1.50	1.75	2.00	2.10	2.20	2.32	2.35	1½ h.	" " July 4, "
0.25	0.60	1.00	1.17	1.25	....	....	....	....	....	" " " 14, "
....	....	....	....	....	....	4.12	3.60	....	....	" " Sept., 1889 (2 storms)
0.35	0.65	0.90	1.15	1.40	1.55	1.78	1.68	....	....	" " April 24, 1894. Preceded by 6 hours of light rain
0.14	0.20	0.47	0.85	0.90	1.01	1.71	1.99	....	....	} Jacksonville, Fla., U. S. Weather Bureau, 1896
0.30	0.61	0.70	0.92	1.30	1.57	1.15	1.80	....	....	
0.07	0.26	0.44	0.77	0.82	0.91	1.50	....	....	....	
0.10	0.30	0.37	0.47	0.62	0.75	0.88	1.00	....	....	} Galveston, Texas, 1896. U. S. Weather Bureau
0.30	0.45	0.57	0.65	0.77	0.90	1.10	1.23	....	....	
....	1.00	....	....	....	....	....	....	....	....	Chicago, once during 1889–1891
....	0.80	....	....	....	....	....	....	....	....	" 2 times " " "
....	0.60	....	....	....	....	....	....	....	....	" 2 " " " "
....	....	1.10	....	....	....	....	....	1.75	2 h.	Ohio Valley, July 16, 1896
....	....	....	....	....	....	....	....	3.30	2 h.	St. Louis, May 14, 1891
0.28	0.58	0.88	1.13	1.23	1.38	....	....	....	....	Cleveland, Ohio, 1896 } U. S. Weather Bureau
0.04	0.19	0.49	0.79	1.04	1.31	1.73	....	....	....	" " "
0.15	0.45	0.85	1.00	1.07	1.14	....	....	....	....	Detroit, Mich., "
0.38	0.57	0.63	0.91	1.13	1.30	1.70	1.78	....	....	" " "
0.25	0.50	0.75	0.95	0.99	1.02	....	....	....	....	Little Rock, Ark., "
0.06	0.33	0.46	0.60	0.66	0.70	0.79	0.91	....	....	" " " "
0.35	....	....	....	....	....	....	0.82	....	....	San Diego, Cal., December, 1896
....	....	....	....	....	....	....	11.5+	....	....	Campo, Cal., August, 1891
....	....	....	....	....	....	....	8.8	....	....	Palmetto, Nev., " 1890
....	....	....	....	....	....	....	....	36 +	24 h.	} Island of St. Kitts
....	....	....	....	....	....	....	....	12 +	2 h.	

ART. 26. GAUGING RAINFALL.

Measurements of snow are ordinarily recorded in inches of fall as found upon a level surface free from drifts; but it is very difficult to obtain an average depth in windy weather, although the best judgment must be used to ascertain this. Generally, besides expressing this in inches, a cylinder of snow of this depth is collected and melted in a can or tube of the same diameter as the cylinder of snow, and the depth of water resulting is recorded as precipitation or rainfall.

Measurements of rainfall are taken by rain-gauges; the most common form consisting of a circular cup of thick brass, its top brought to a chisel-edge, the bottom cone-shaped and connected with a deep tube of known diameter into which the rain flows from the cup. (See Fig. 2.) The area of the top of the cup and that of the tube bear a known relation to each other,—10 to 1 is a convenient ratio,—and the depth in the tube is measured by a stick so graduated that, when it is lowered to the bottom of the tube, the scale will give the actual depth of rainfall, allowance being made in the scale for both the relative areas and the displacement of the stick. The depth is customarily expressed in inches and decimals of an inch. The readings are taken daily and at the beginning and ending of each storm.

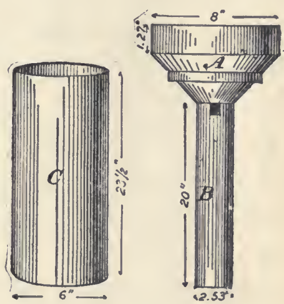

FIG. 2.—RAIN-GAUGE.

For many purposes it is desirable to know the rate of fall for short intervals of five minutes or less, and for ascertaining this self-recording gauges are necessary. Several styles of such gauge have been used. One is the "tipping-tank," which tips and empties itself as soon as it has received .01 inch of rainfall, immediately returning to an upright position, the time of each discharge being automatically recorded. Another gauge consists of a tank suspended by a spring-balance, a pencil attached to the tank continuously recording its vertical position upon a cylinder revolved by clockwork once in twenty-four hours. In using any recording-gauge the total water caught should be retained, and measured or weighed each day as a check upon the record.

The size of the collector-cup seems to have some effect upon the catchment. For example, of four 3-inch cups and one 8-inch one, in use on Mt. Washington, the average total amount collected by the 3-inch cups in one year was 46.26 inches, while that recorded by the 8-inch cup was 58.70 inches. It is probable that the larger the collector-cup the more accurate the result. The position of the gauge relative to the ground-surface also influences the amount of catchment, those placed near the surface generally giving the larger results. It has been found that a gauge 100 feet above the ground will give on an average but 65% as much as one on the surface. Experiments seem to show that the intensity of the wind is the controlling factor in these variations. (See the *Weather Review* for June, 1897.) It is maintained by many that gauges at the surface give less accurate results, since they receive not only the actual precipitation, but also a certain amount of moisture from the surrounding ground which, after falling, again rises by splashing and evaporation and is reprecipitated. A large number of the gauges of the signal service are placed upon the roofs of tall buildings, and in cities this is generally necessary; but in open country a height of 3 to 6 feet from the surface will probably give the most accurate results. The gauge should be at least as far from any building or other obstacle as the top of this is above the gauge. The rim of the collector-cup should be level.

The U. S. Weather Bureau has for years been taking records of precipitation in various parts of the country, and about 150 stations are now operated by them; records being received from hundreds of voluntary observers also. These records are available to any one, and should be consulted in the study of the precipitation at any locality.

Art. 27. Storage of Rain-water.

Since rainfall is not only not continuous, but droughts of considerable duration may be expected any year, users of rain-water must arrange to store a sufficient amount to carry them over the dry seasons. A dwelling 25 by 40 feet, having a roof of 1000 square feet area, would, if in Philadelphia, receive upon this roof an average amount of $\left(1000 \times \frac{43.35}{12}\right)$, or 3613.5 cubic feet per year, or 74 gals. per day. But 43.35 inches is the mean of 68 years, and for five successive years an average of but $79\frac{1}{2}\%$ of this amount fell; for three years $77\frac{1}{3}\%$, for two years but $73\frac{1}{2}\%$, and for one year but 68% of this amount was precipitated. (Table No. 19, page 68.) Moreover we see by Table No. 23, page 77, that a monthly rate of 0.19 inch has occurred in March, and we judge from Table No. 25 that for six months the rate may be only 1.71 inch per month. It is thus apparent that if we need a continuous supply of only about 35 gals., sufficient storage to make up the occasional deficiency during only the six dryest months is required; if about 50 gals. per day is needed, storage for one dry year is required; while if 57 gals. is needed, storage must be provided to tide over three dry years.

What the amount of storage must be may be determined by a calculation similar to that shown on the next page. Taking the year from December, 1879, to November, 1880, inclusive (see Table No. 24, page 78), we find the average precipitation for this year in Philadelphia was 34.22 inches, or an average monthly rate of 2.85 inches. If we assume the cistern as empty at the beginning of this period we will have the following condition from month to month.

Month.	Amount in Reservoir at First of Month.	Amount of Precipitation, cu. ft. per Month.	Consumption. 2.85 in. or 237.64 cu. ft. per. Month.	Surplus Added to Reservoir	Deficiency to be Supplied by Reservoir	Amount in Reservoir at End of Month.
	Cu. Ft.	Cu. Ft.	Cu. Ft.	Cu. Ft.	Cu. Ft.	Cu. Ft.
December	0.000	390.83	237.64	153.19	....	153.19
January	153.19	125.83	237.64	....	111.81	41.38
February	41.38	202.50	237.64	....	35.14	6.24
March	6.24	294.17	237.64	56.53	....	62.77
April	62.77	202.50	237.64	....	35.14	27.63
May	27.63	45.00	237.64	....	192.64	−165.01
June	−165.01	139.17	237.64	....	98.47	−263.48
July	−263.48	645.00	237.64	407.36	....	143.88
August	143.88	424.16	237.64	186.52	....	330.40
September	330.40	91.67	237.64	....	145.97	184.43
October	184.43	145.00	237.64	....	92.64	91.79
November	91.79	145.85	237.64	....	91.79	0.00

It appears from this that there should have been in the cistern on December 1st at least 263.5 cubic feet of water to make up the deficiency during January to June, inclusive; the cistern would then have been empty on July 1st, but during July and August would have received about 600 cubic feet surplus and, if sufficiently large to hold this amount, would on December 1st again contain 263.5 cubic feet to provide for the following year. But this is on the assumption that all the rainfall is collected. Even in a dry year, however, unless the water is carefully filtered, the first of each rain should be wasted, and this will probably require at least $\frac{1}{4}$ inch each month. Making this allowance we find that 410.5 cubic feet must have been on hand December 1st to maintain the supply, and that the capacity of the cistern must have been at least 543 cubic feet; or 552 cubic feet if all the water received was to be stored. The contents on the first of each month would then be as follows:

Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.
410.48	542.67	409.86	353.72	389.25	333.11	119.47	0.00	386.36	551.88	384.91	271.27

Cisterns for private supply have generally a capacity of 500 to 50,000 gals., 5000 gals. (667 cubic feet) being a very good size for average cases. An underground cistern should be absolutely tight, having no openings except for the pump-suction and rain-water leader, and a ventilator or air-escape which cannot be entered by mice or other animals. Where the strength of the building is sufficient, a tank on the top floor is generally a more convenient and better arrangement, as affording inspection and freedom from contamination, and delivering the water under a head; but the water is warmer in summer than if an underground cistern be used. Two circular tanks, each 8.4 feet diameter and 6 feet high, will hold 5000 gals.

The daily consumption of 50 gals. referred to allows, for a family of, say, six persons, but 8 gals. per capita per day,—an amount which does not permit of the use of indoor sanitary appliances; and a much larger supply is desirable for hygienic reasons as well as those of convenience.

Art. 28. Summary. Estimating Future Rainfall.

Estimates of future rainfall must be based upon the records of past precipitation. These should cover as long a period as possible, and estimates based upon the records of two or three years only can be but approximate and tentative. Where there are no long-period records for the locality in question, all available ones of that precipitation district should be consulted, and especially of those places nearest and having the most nearly similar location as regards elevation and surrounding topography.

It will be noticed that of all Weather Bureau stations in the North Atlantic, Ohio, and West Gulf States (District No. 1) none has a precipitation rate more than 32% less than the district average; and in District No. 2, 28% is the greatest amount by which any rate falls below the district average. In District No. 3, excluding the Missouri valley, 18% is the greatest amount; in District No. 4, 35%; in District No. 7, 22%; and in District No. 8, 38% is the greatest amount by which any rate falls below the district average. In the Plateau (Districts Nos. 5 and 6) there is great variation in rates, even within the limits of one State; and still greater variation may be discovered as more stations are added, there being now but thirteen in this district.

In a new country much can be learned by the character of vegetation, size of streams, evidence of past freshets, etc., as to whether the average precipitation is great or small, and what relation it bears to that which is observed.

In figuring upon the sufficiency of a given supply the water-works engineer is most concerned in the *minimum* monthly, yearly, and dry-cycle rates; the maximum having little bearing except upon the methods of controlling and wasting the surplus. Periods of drought seem to occur at intervals of four to ten years throughout the country, but no law of their recurrence has been formulated, nor do the records seem to suggest any such; and the only practicable method seems to be to base the design upon the dryest cycle recorded for that district, or on an even lower rate if the record covers less than thirty or forty years.

QUERIES.

6. Calculate the size of tank required in New York, and the amount necessary to be contained in it on Dec. 1, 1879, if it is to supply up to Dec. 1, 1881, 75 gallons per day, receiving the rainfall on a roof of 1400 sq. ft. area, $\frac{1}{4}$ in. per month of which rainfall is wasted.

7. In case a rainfall of one inch entirely cleared all soot and ammonia from the air of Cleveland, O., when it contained the amounts of these found by Mabery in his first analysis (page 56), what quantities of matter in suspension and of ammonia would be contained in the rain-water, in parts per 100,000? These impurities being considered to permeate the lower 100 feet of the atmosphere.

8. From the Philadelphia records, arrange the months in the order of probable drought; that is, place first the month in which a drought is most likely to occur, and last that in which it is least likely.

CHAPTER VI.

SURFACE-WATERS.

ART. 29. EVAPORATION.

IF all the precipitation upon a given area reached the streams draining it as run-off, the calculation of the amount of the latter would be simply finding the product of the total watershed by the precipitation. But, as has been stated, much of the precipitation returns to the air by evaporation. Since, however, all but a very minute portion of the rainfall on a given watershed which does not leave it as evaporation leaves it ultimately as yield, it would seem as though the latter might be obtained by taking the difference between the total precipitation and the total evaporation; and C. C. Vermeule, as consulting engineer for the New Jersey State Geological Survey, deduces a method of calculating yield in this way. There remains, however, the difficulty of determining the evaporation. The amount of evaporation depends upon the degree of dryness of the air, the temperature of the air and of the soil or water from which the evaporation takes place, upon the amount of moisture in the soil, and upon the force of the wind. Most measurements of evaporation have been made from water-surfaces or by an evaporometer; the Piche being the evaporometer most used in this country. Water-surfaces form but a small proportion of the total areas of most catchment-basins, but the amount of evaporation from these and from the reservoirs themselves can be ascertained much more accurately than that from earth and vegetation. The following tables give the evaporation from water-surfaces at a number of places in this country.

TABLE No. 27.

EVAPORATION FROM WATER-SURFACES, IN INCHES.

Location.	Monthly.			Annual.		
	Max.	Min.	Mean.	Max.	Min.	Mean.
Boston, Mass.	7.50	0.66	3.29	43.63	34.05	39.20
Sweetwater, Cal.	9.02	0.25	4.51	58.65	48.68	53.88
Rochester, N. Y.	6.20	1.51	2.61	34.4	30.0	31.3
Middle Atlantic States	......	......	......	48.1	25.2	39.9
South Atlantic States	......	......	......	51.6	38.4	45.3
East Gulf States	......	......	......	56.6	45.4	50.6
West Gulf States	......	......	......	52.4	45.6	48.9
Ohio Valley and Tennessee	......	......	......	54.8	44.5	49.4
Lower Lake	......	......	......	38.6	32.9	35.8
Upper Lake	......	......	......	36.8	23.0	27.7
Upper Mississippi	......	......	......	52.2	28.1	38.8
Extreme Northwest	......	......	......	31.0	22.1	26.7
Yuma, Ariz.	......	......	......	......	......	95.7
San Diego, Cal.	......	......	......	......	......	37.5

TABLE No. 28.

EVAPORATION FROM WATER-SURFACES, BY MONTHS.

	Duration of Observation.		Jan.	Feb.	Mar.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
Boston	16 years.	Mean	0.96	1.05	1.70	2.97	4.46	5.54	5.98	5.50	4.12	3.16	2.25	1.51
		Maximum	..	....	....	3.12	5.89	7.01	7.50	7.41	5.13	4.13	3.00	....
		Minimum	....	....	....	2.78	3.35	3.94	4.82	4.25	3.08	2.51	0.66	....
Sweetwater	7 years.	Mean	2.69	2.41	2.86	4.46	4.96	5.41	6.43	6.74	5.96	4.77	4.51	2.68
		Maximum	3.61	3.53	3.38	5.82	6.14	7.30	8.81	9.02	7.36	6.56	5.53	6.28
		Minimum	1.59	1.35	1.08	3.63	3.45	3.19	3.16	3.07	4.64	3.00	3.35	0.25

Of the Sweetwater data those for the first four years were gauged in a pan floating in the reservoir (see Fig. 3); for the last three years by a Piche evaporometer. Both this and the Richards evaporation-gauge are thought by some to give inaccurate results, since they are affected by the temperature of the air only, which is seldom the same for any length of time as that of a near body of water. A more accurate method would seem to be to measure the actual loss from a pan filled with water and floating in a lake or other body of water. Such a pan and scale are shown in Fig. 3. Owing to the protection from wind offered by the sides of the pan,

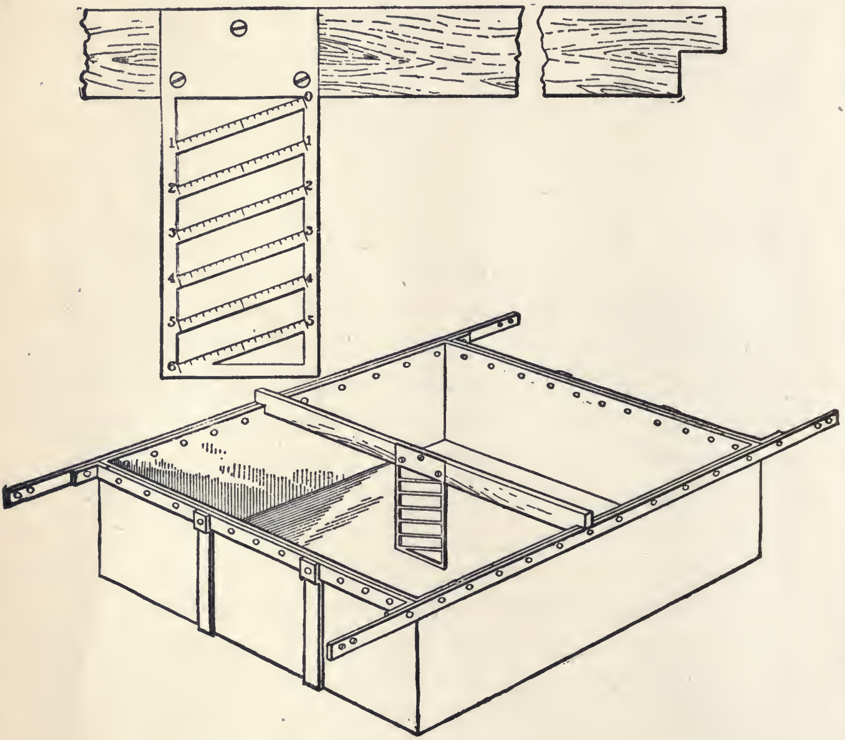

FIG. 3.—EVAPORATING-PAN.

this probably gives results slightly less than the evaporation from lake or pond surfaces.

Evaporation is seen to vary less than does rainfall, the greatest variation from the annual mean at Boston during sixteen years, for instance, being about 13%.

The ratio of the mean monthly to the mean annual evaporation at each place is shown in Table No. 29, as well as the monthly ratios for the year of maximum evaporation at each place.

TABLE NO. 29.

RATIOS OF MONTHLY TO MEAN ANNUAL EVAPORATION FROM WATER-SURFACES AT BOSTON AND SWEETWATER.

	Jan.	Feb.	Mar.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
Boston, mean monthly rate............	.025	.027	.043	.076	.114	.141	.153	.141	.105	.080	.057	.038
Sweetwater, mean monthly rate.......	.050	.045	.053	.083	.092	.100	.120	.125	.111	.088	.083	.050
Boston, monthly rate, year of max. evap.	.022	.024	.039	.069	.086	.161	.162	.170	.117	.064	.051	.035
Sweetwater, " " " " " "	.043	.023	.052	.098	.079	.109	.148	.110	.105	.110	.080	.043

These two places represent almost the extremes of the United States, and the monthly ratios for almost any locality will lie between these as limits. It will be noticed that the ratios are much more uniform on the Sweetwater basin than in Boston, owing largely to the more uniform temperature throughout the year; also that in Boston the excessive evaporation during the year of total maximum occurred between June and September, and on the Sweetwater basin between June and October—that is, during the warmest weather.

The evaporation from snow was found at Boston to average .02 inch per day; and that from ice .06 inch per day. On exposed mountain sides and tops it would probably exceed these averages.

Still more important than the evaporation from water is that from soils of different characters; but very few reliable data on this subject have been obtained. The amount of water taken up by vegetation is important, but on this subject also very little definite information is available.

Experiments at Rothamsted, England, showed the annual evaporation from bare soil to be 17.09 inches when the average rainfall was 31.04 inches; and of the 13.95 inches percolating through the soil 9.44 inches was collected between October and March, and 4.51 inches during the seven warmer months.

TABLE No. 30.

EVAPORATION FROM GRASS AND EARTH.

	Jan.	Feb.	Mar.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Year.
From short grass at Emdrup, Denmark; 8 years	0.7	0.8	1.2	2.6	4.1	5.5	5.2	4.7	2.8	1.3	0.7	0.5	30.1
From water during same time.	0.7	0.5	0.9	2.0	3.7	5.4	5.2	4.4	2.6	1.3	0.7	0.5	27.9
From long grass, same time	0.9	0.6	1.4	2.6	4.7	6.7	9.3	7.9	5.2	2.9	1.3	0.5	44.0
Evaporation from earth at Bolton Le Moor, England; 10 years	0.64	0.95	1.59	2.59	4.38	3.84	4.02	3.06	2.02	1.28	0.81	0.47	25.65
Evaporation from earth at Whitehaven, Eng.; 10 years	0.95	1.01	1.77	2.71	4.11	4.25	4.13	3.29	2.96	1.76	1.25	1.02	29.21

Table No. 31, from the Report of the Kansas State Board of Agriculture for 1889, shows the result of experiments on consumption of water by various crops. From this, grasses and grains would appear to consume about .12 inch or .14 inch per day, vineyards and potatoes about .03 to .04 inch.

TABLE No. 31.

DAILY CONSUMPTION OF WATER BY CROPS. (RISLER.)

	Inches.		Inches.
Lucerne grass	0.134 to 0.267	Wheat	0.106 to 0.110
Meadow grass	0.122 to 0.287	Rye	0.091
Oats	0.140 to 0.193	Potatoes	0.038 to 0.055
Indian corn	0.111 to 1.570	Oak trees	0.038 to 0.030
Clover	0.140	Fir trees	0.020 to 0.043
Vineyard	0.035 to 0.031		

In Western irrigation it is considered necessary to use from 18 to 24 inches of water per year, not all of which, however, is taken up by the crops; but which, on the other hand, is in addition to the rainfall. The crop-consumption of water extends over about six months only in the temperate zone. If from 1 inch (for forests) to 6 inches (for grass) per month is taken up by vegetation, or, say, an average of 4 inches (the amount required for cereals) during six months of the year, it is apparent that during that time much of the rainfall upon areas so cultivated would be thus absorbed. It is probable that there is little evaporation directly from the soil in such areas; the values in Table No. 30 probably including the total loss by evaporation and vegetable absorption. This apparently is further shown by the fact that evaporation from woodland soil is found to be but 35 to 40 per cent as great as that in the open.

The above tables of soil-evaporation and vegetable-absorption should not be considered as being in any great degree accurate. The only method at all accurate which has so far been employed for ascertaining on a large scale the amount of moisture thus withdrawn from the yield is that of measuring the stream-flow and deducting this from the total rainfall. Since the evaporation is, so far as the water-works engineer is concerned, in most cases a means and not an end, the only benefit to him from this method lies in the opportunity offered for studying the evaporation-value thus found in its relation to other known phenomena, and learning the controlling laws. By this method, comparing data from a large number of rivers and their watersheds, Vermeule deduced the formula

$$E = (15.5 + .16R)(.05T - 1.48),$$

in which E is the annual evaporation (including vegetable absorption), R the annual rainfall, and T the mean annual temperature. The monthly evaporation he represents by the following, in which r is the monthly rainfall:

January.	February.	March.	April.	May.	June.
$.27 + .10r$	$.30 + .10r$	$.48 + .10r$	$.87 + .10r$	$1.87 + .20r$	$2.50 + .25r$
July.	**August.**	**September.**	**October.**	**November.**	**December.**
$3.00 + .30r$	$2.62 + .25r$	$1.63 + .20r$	$.88 + .12r$	$.66 + .10r$	$.42 + .10r$

This formula seems to give very close results for New Jersey, eastern New York, and eastern Pennsylvania streams; but is inapplicable to many sections. For instance, applying this to the basin of the Sweetwater River, Cal., would give an annual evaporation 5 to 15 inches greater than the rainfall. It seems probable, nevertheless, that in this direction lies the best method for calculating probable yield of a watershed. But with our present knowledge and data the only accurate one is to make direct measurements of the stream-flow. The safest rule, where this is impossible, is to compare the watershed under consideration with another of known yield and similar in all or most of its characteristics, including precipitation; bearing in mind the effect of variations in elevation, temperature, wind, and other conditions already referred to.

ART. 30. NATURAL STORAGE.

There is another factor of yield which affects more the periodic than the total yield, although it has considerable effect upon the latter also. This is the storage in the ground of water, part of which supports vegetation, and part slowly feeds springs and streams; and in ponds of surface-flow which is gradually, though more quickly, given to the streams. A part of the water so stored in either ground or pond is evaporated; but a considerable proportion of the ground-water, and generally of the pond-water also, reaches the stream. This storage is not only an important factor in maintaining continuous stream-flow and in supporting vegetation, but it has, as stated, some effect upon the total yield, and is valuable also in relieving the storage-reservoir of a considerable portion of its duty. Where there is little ground-storage, as on dense clay land, there can be little vegetation, largely because the ground contains no moisture to support it; and since little water soaks into the ground, most of the rainfall runs immediately to the stream. Where a large amount soaks into the ground considerable of this is taken up by vegetation through its roots and thus abstracted from the yield. We might therefore expect to find a greater total yield from a rocky or clayey soil than from a loamy or sandy one; although it would be more difficult to retain the whole for use, owing to the great quantity flowing off in a short length of time.

Not only does ground-storage develop vegetation, but vegetation, by loosening the soil with its roots and obstructing surface-flow, develops ground-storage; and hence it follows, both as cause and effect, that the yield from a wooded or cultivated soil is more uniform and less "flashy" than from a bare one.

The water which percolates into a soil descends to a more or less fluctuating ground-water level, which is the surface of the stored water. This surface slopes in the direction in which the ground-water is moving, its elevation at any point being governed by that of the outlet and by the amount flowing (the greater the amount the greater being the velocity of flow and hence the slope); the slope increasing with the fineness and density of the soil also. (See also Art. 40.) If a stratum of impervious material lies higher than the elevation at which the ground-water surface would otherwise stand, the ground-water will flow above this, generally with greater velocity and consequently furnishing less storage and more varying yield.

Above the ground-water surface proper, some water is held in the soil by capillary attraction for nourishing plant life, and in very fine-grained soils the amount thus retained may be considerable. As evaporation removes this water from the top soil, capillary attraction draws more from below, to be in turn evaporated; but the amount of water which can thus be raised decreases with the fall of the ground-water surface, and thus, when the interval between rains is long, the upper soil becomes thoroughly dried. The next rain must then renew this upper supply before contributing any water to the run-off storage. "If the rainfall is sufficient to supply the evaporation and plant growth, the flow from ground-water will remain constant, because the head which forces it through the rocks and gravels is constant. When the rain is insufficient, the head will be drawn down and the flow will decrease at a certain fixed rate." Once the draught upon the ground-storage is fairly established and the water drawn down, unless the rainfall is greater than it usually is in summer it is all absorbed by the dried earth and does not reach down far enough to increase the head and consequent flow of ground-water. "Rainfalls which, if occurring in May, or in the autumn after the ground-water has been replenished, would cause violent floods, have no effect at all upon the stream-flow when they occur during dry months. This difference in effect cannot be ascribed to direct evaporation, for in the case of concentrated rainfall evaporation has little time to act. It is due to the drawing down of ground-water, which leaves a great capacity for absorption of rain by the earth." (Vermeule's Report on "Water-supply, Geological Survey of New Jersey.")

The absorption-capacity varies with different soils, and also the amount of water yielded. A coarse gravel will yield almost its entire contents, while fine sand or clay will yield practically none, retaining it all by capillary attraction. The table on page 97, from Schubler, gives results obtained from various soils.

From this table it would appear that, given the same conditions as to vegetation, exposure, climate, etc., the evaporation from all soils would be quite similar.

The subsoil in twenty-three localities in South Carolina and Maryland was found to contain from 37.2 to 65.1 per cent by volume of voids, with a mean of 48.73%.

TABLE No. 32.

CAPACITY FOR ABSORPTION AND YIELDING UP OF WATER POSSESSED BY VARIOUS SOILS. (FROM SCHUBLER BY VERMEULE.)

Soil.	Water absorbed by 100 parts of soil after drying at 40° or 50° Fahr.	Percentage of water evaporated in 4 hours at 56.7° Fahr.	Parts of water in 100 parts of soil evaporated in 4 hours (soil saturated).
Siliceous sand	25	88	22
Gypseous soil	27	....	....
Calcareous sand	29	75.9	22
Barren clay	40	52	21
Fertile clay	50	45.7	23
Loamy clay (or clayey soil)	60	34.9	21
Pure clay	70	....	....
Fine calcareous soil	85	28.6	24
Humus	190	20.2	38
Magnesian soil	156	....	....
Garden soil	89	....	....

The least flow of the Connecticut River is equivalent to about .05 cubic feet per second per square mile of watershed, which is maintained during probably a month at least of no rainfall and three months when the rainfall no more than equals the evaporation and requirements of vegetation. The flow must, during this time, be maintained by the ground-water storage, and is equivalent to about 1.7 inches of water over the entire area. If the soil furnishing ground-storage averages an absorption-capacity of 33%, the ground-storage surface would be lowered about 5 inches on an average. But not all the ground furnishes storage, and probably not more than 50% of that held is yielded to the stream; also the lowering increases with the distance from the river, so that in some places it may amount to 3 feet or more.

In a fairly wet or rainy season the ground-water surface may be raised 5 or 10 feet, even reaching the surface in many places and producing ponds or swamps.

Most storage-reservoirs have not water-tight shores, and as the water rises in the reservoir the ground-water level in the vicinity rises also, and ground-storage is obtained in addition to that in the reservoir. When the reservoir water is drawn down this ground-storage also is drawn upon, and more water is yielded than the capacity of the reservoir.

ART. 31. YIELD OR RUN-OFF.

The yield is the total *rainfall*, less the amount consumed by *vegetation* and *evaporated*, and it is regulated in its final delivery by *ground-storage*. All of these factors and the relations between them are as yet but partly understood, and the data concerning them are scant in many localities. The most common method of estimating yield is to assume it to be a certain percentage of the rainfall—generally about 50% in the eastern part of the country; but this assumes the rainfall to be the only factor, which it is not, and also requires the estimating of the percentage of yield. The amount of yield varies with the climate, the surface and subsurface structure and conditions, the meteorological conditions before and after each rain, and other causes. On the Sudbury watershed, for instance, the amount of rainfall reaching the streams during the seven months from November to May inclusive is large, while from June to October it is small; the greatest stream-flow being in February, March, and April. The annual yield of this shed has varied from 31.9 to 62.2 per cent of the precipitation, the mean for sixteen years being 49.5%. "The percentages depend upon the distribution of rainfall throughout the year. A heavy summer rainfall and a light winter rainfall mean a small percentage of collection; and, conversely, a light summer and a heavy winter rainfall mean a large percentage of collection; so that the total rainfall for the year is but a partial index to the yield of a water-

Table No. 33.

RUN-OFF FROM A NUMBER OF WATERSHEDS.

Watershed.	Area in Sq. Miles.	Average Percentage of Rainfall reaching Streams.													Remarks.
		Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Year.	
Perkiomen Creek, Pa. ..	152	79	85	108	81	40	25	18	33	28	25	46	61	51.6	For 10 yrs. 71% cultivated.
Neshaminy " " ...	139.3	90	96	106	72	31	15	13	20	18	19	40	76	48.7	" " " 92% "
Tohickon " " ...	102.2	105	115	124	82	34	21	18	25	29	24	53	72	59.6	" " " 72% "
Connecticut River	11,083	59.1	65.8	76.3	145.0	132.2	36.5	21.3	21.8	29.3	28.3	44.8	60.7	56.5	For 15 years.
Sudbury River, Mass....	75.199	49.1	78.2	109.6	109.1	62.3	29.1	8.9	13.0	14.2	23.1	39.5	52.5	49.5	For 16 years. Hilly, steep; some swamps.
Cochituate " " ...	18.87	53.1	71.9	84.6	83.8	47.9	27.9	13.1	17.7	23.5	23.6	34.0	49.9	43.8	For 29 years. Flat, sandy.
Potomac River (Maryland and W. Virginia)......	11,043	65.2	98.5	84.9	104.1	44.7	38.2	23.1	21.3	25.0	40.5	54.3	72.3	53.0	For 6 years. Steep valleys; few lakes.
Savannah River........	7,294	57.6	74.6	64.4	95.7	37.3	31.8	22.8	42.7	46.8	41.6	79.1	48.1	48.9	
Arkansas "	3,060	....	....	....	....	....	....	....	....	....	....	....	....	30.9	
Salt "	12,260	....	....	....	....	....	....	....	....	....	....	....	....	29.2	
Upper Snake "	10,100	....	....	....	....	....	....	....	....	....	....	....	....	67.6	
Bear "	6,000	....	....	....	....	....	....	....	....	....	....	....	....	41.5	
Owyhee "	9,875	....	....	....	....	....	....	....	....	....	....	....	....	16.2	

shed." "It may be said that, for systems which depend upon storage, it is not the summer droughts which are to be dreaded, but the winter and spring droughts; for it is the flow in these months upon which we depend to fill the reservoirs." (Desmond FitzGerald, in Transactions of American Society of Civil Engineers, vol. XXVII.)

In California the amount collected annually may vary from nothing when the rainfall is less than 20 inches, to 60% in years of heavy rainfall, averaging not far from 30%. In Southern California the average for seven years was 10.6%. Table No. 33 shows the run-off from a number of watersheds, expressed in percentages of the rainfall. In the case of the Potomac basin the December percentages varied between 18 and 24, and the annual percentages between 30.0 and 58.2. On the Sudbury watershed the December percentage varied during sixteen years between 9.6 and 264.4 per cent, and the annual between 31.9 and 62.2 per cent, and the annual on the Cochituate between 25.7 and 69.1 per cent.

The following table shows the maximum and minimum yield for two watersheds:

TABLE No. 34.

MAXIMUM, MINIMUM, AND MEAN YIELD: CONNECTICUT AND POTOMAC WATERSHEDS.

	Jan.	Feb.	Mar.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
CONNECTICUT RIVER, 13 YRS.												
Rainfall, inches depth......	3.27	3.10	3.94	3.26	3.17	4.00	4.79	4.87	3.04	3.93	3.93	3.39
Max. yield, " "	5.70	4.06	5.64	7.61	6.54	3.11	2.32	2.52	2.24	2.81	3.60	4.93
Min. " " "	0.72	0.77	0.90	2.61	1.90	0.79	0.69	0.70	0.66	0.71	0.67	0.72
Mean " " "	1.93	2.04	3.00	4.73	4.19	1.46	1.02	1.06	0.89	1.11	1.76	2.06
Mean percentage yielded....	59.1	65.8	76.3	145.0	132.2	36.5	21.3	21.8	29.3	28.3	44.8	60.7
POTOMAC RIVER, 6 YEARS.												
Rainfall, inches depth......	3.21	3.35	4.39	3.48	5.11	5.25	4.89	3.81	3.86	2.65	2.88	2.59
Max. yield, sec. ft. per sq. mi.	14.00	15.81	17.10	20.87	18.60	42.88	19.36	5.94	7.63	14.50	15.25	9.48
Min. " " " " " "	0.24	0.22	0.25	0.25	0.23	0.25	0.25	0.27	0.27	0.27	0.27	0.27
Mean " " " " " "	1.81	3.22	3.14	3.15	2.04	1.75	0.87	0.67	0.95	1.07	1.61	1.15
Mean percentage yielded ...	65.2	98.5	84.9	104.1	44.7	38.2	23.1	21.3	25.0	40.5	54.3	72.3

From examples of such wide variation it is evident that the use of a mean percentage of the rainfall for estimating the run-off will give only very approximate results; particularly since the lowest percentages usually occur during the years of minimum precipitation. (See Figs. 4 and 5.)

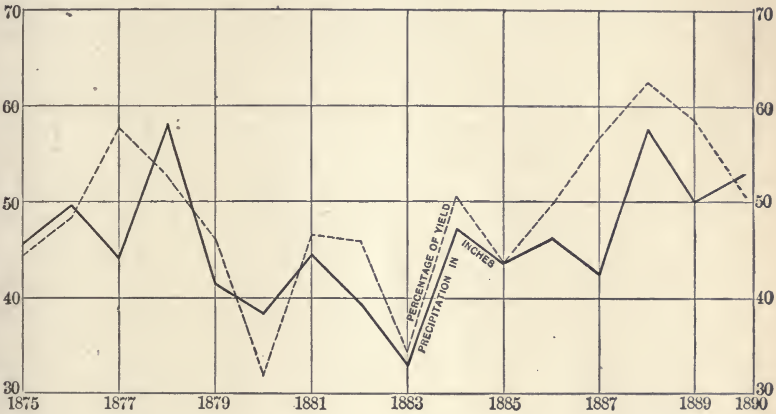

FIG. 4.—PRECIPITATION AND PERCENTAGE COLLECTED; SUDBURY WATERSHED.

FIG. 5.—RAINFALL AND PERCENTAGE COLLECTED; SWEETWATER BASIN.

Run-off is sometimes expressed in cubic feet per second per square mile, as in Tables Nos. 34, 35, and 36. Since run-off is a product of the rainfall and the percentage of this reaching the streams, the records of stream-flow might be expected to show greater variation than either of these, which is seen to be the case.

TABLE No. 35.

RUN-OFF FROM SEVERAL DRAINAGE-BASINS.

Watershed.	Length of Observation, Years.	Area, Sq. Mi.	Run-off, cu. ft. per sec. per sq. mi.		
			Max.	Min.	Mean.
Sudbury	16	75.199	44.26	0.07	1.669
Connecticut	15	10,234	20.00	0.50	1.86
Croton	14	353	75.22	0.21	1.65
Perkiomen	7	152	69.20	0.14	1.90
Raritan	1	879	27.00	0.48	1.72
Passaic	3	773	24.80	0.43	2.58
Potomac	6	11,043	42.60	0.17	1.85
Savannah	8	7,294	41.2	0.27	1.64
Ohio	1	200,500	6.17	0.27	2.10
Missouri	12	526,500	8.52	0.60	2.16
Arkansas	5	3,060	1.55	0.06	0.27
Rio Grande	3	30,000	0.55	0.00	0.06
Gila	1	13,750	0.46	0.00	0.04
West Carson	2	70	18.34	0.50	2.39
Sevier (Utah)	3	5,595	0.42	0.01	0.09

TABLE No. 36.

RATIO OF MONTHLY TO MEAN ANNUAL RUN-OFF.

	Jan.	Feb.	Mar.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Mean.
Mean of 3 New England watersheds for 13 years, cu. ft. per sec. per sq. mile.	1.866	3.042	3.555	2.533	1.518	0.772	0.374	0.569	0.528	0.838	1.278	1.703	1.539
Ratio of monthly to mean	1.20	1.97	2.31	1.65	0.98	0.50	0.24	0.36	0.34	0.54	0.83	1.10	
Boston, for 20 years. ratio of monthly to mean	1.15	1.80	2.68	1.91	1.10	0.46	0.17	0.28	0.23	0.47	0.81	0.94	1.644

That the run-off per square mile from two watersheds should be the same, it would be necessary that they be similar in shape, in ground slope, in soil, in amount and character of wooded and cultivated areas, in area of ponds, and that the wind movements and humidity, as well as the rainfall, be similar in both; since all these factors affect the run-off. Two adjacent watersheds may differ greatly in some of these conditions, and hence in yield. For instance, we find the proportion of rainfall yielded by three Pennsylvania watersheds within a few miles of each other to vary as 51.6, 48.7, and 59.6 (Table No. 32), or a difference of 22% between extremes. Two adjacent Massachusetts watersheds varied in percentage of yield by 12%, or as 49.5 and 43.8. In central California the mean yield is not far from 30% of the precipitation, while in the southern part of the same State the mean for seven years was but 10.6%.

Although these data show a great variation in percentage of rainfall yielded in different parts of the country, and in different years on the same watershed, the average yield in a given section of country is fairly uniform. For instance, in the New England and Central Atlantic States the yield generally averages from 44 to 60 per cent of the precipitation, and the extreme variation in any one place is about 25% of its mean, so that the use of the average rainfall and average percentage yielded would very probably give a result in error not more than 30 or 40 per cent. Again, if we compare the second-feet per square mile, we find in the same locality a mean yield of about 1.6 second-feet, with a maximum variation from this of about 40 to 50 per cent. By using judgment in selecting for comparison a watershed with characteristics similar to the one in question, the probable error may be reduced in many cases to 10 or 15 per cent. Below this it is hardly practicable to go, except by obtaining long-period records of yield of the area in question.

Since the evaporation from various soils is practically the same if the affecting conditions be similar (see Table No. 32), and likewise that from water, it is probable that the relative run-off per square mile from two watersheds similarly located geographically and topographically will be closely related to the proportion of water-surface on each area. Thus, knowing the yield of a given watershed and the proportion of water-surface to the entire area, the probable yield of the ground-surface alone can be found by adding to the total yield of the entire area the evaporation from all water-surfaces and deducting the rainfall upon the same. The quotient found by dividing the result by the area of the watershed in square miles will be the approximate yield per square mile of the *ground-surface*. This can then be corrected for any given watershed having similar characteristics, by adding to the yield of the entire area, considered as a ground-surface, the net yield of all water-surfaces on this area. (See the table, page 117.)

Where no watershed with similar conditions can be found, the decision must be a more or less arbitrary one, based upon information obtained from the oldest inhabitants, observed stream-channels and ponds, the location of the place with reference to the coast, sea-level, mountain-ranges, etc., the character of soil and topography, and the vegetation.

The measured yield of the area in question for a long series of years forms of course the most reliable basis of estimate; the next is the known yield of similar areas in the same district; and from precipitation, either gauged at the locality in question or estimated from the district rate, the yield may be estimated either as a certain percentage of this, or by deducting the probable evaporation according to Vermeule's rule (page 93) for the Eastern States, or a similar one formulated for the locality in question. In fact, it is desirable to estimate by each of these methods and compare results, giving them relative weight in the above general order.

The mean yield thus obtained can be made the basis of estimating the volume of water-supply, when sufficient storage is provided to tide over a number of dry years; but when this is not possible, and for calculating the storage necessary, the minimum and maximum yield must be known approximately; and these may be estimated by comparison of known data in the same way as the mean yield, but the length of time covered by the records plays an even more important part.

With reference to yield throughout the United States the general statement is made in the U. S. Weather Bureau Reports that "for the area of the United States east of the 95th meridian (Omaha and Galveston) the run-off is from 35 to 50 per cent of the total rainfall. It appears to be largest in the vicinity of the Great Lakes, and diminish from this region slowly to south and east, and rapidly towards the west. In the lower peninsula of Michigan, for instance, the run-off is 50% of the total rainfall. Along the Gulf coast it appears to be only from 30 to 40 per cent, and along the Atlantic coast it probably varies from 30 to 50 per cent. In general, for the interior States east of the 95th meridian the run-off is between 40 and 50 per cent of the total rainfall.

"As soon as we cross the 95th meridian westward we find a very sharp fall in the percentage of run-off to the total rainfall. For the band extending north and south between the 95th and 105th meridians this percentage varies from 10 to 25 per cent, and over Iowa is about 33%. The percentage is highest at the northern end of the band indicated, and lowest at the southern end. Going still farther westward we come to another very marked area, that of the Continental Divide; here the percentage of run-off suddenly increases, reaching the highest figure to be found in the United States. From Montana to Colorado it varies from 60 to 70 per cent of the total rainfall. In New Mexico it falls to about 33%. This is evidently on account of the easy flow of water from the mountain ranges in the area in question. West of the Divide the run-off is again small, being only 15 or 20 per cent in Arizona and Nevada, about 30% in Idaho, and nearly 50% in

Utah. Utah, it seems from its topography, partakes of the character of the band lying just to the east of it. Along the Pacific coast the run-off is about 25% in Oregon, 30% in Washington, and between 45 and 50 per cent in California" (the northern part only).

The above is a very general statement, and, particularly in the far West, is subject to many variations and exceptions, some of which are recorded in the preceding tables.

(It may be convenient to remember as an approximation that the average yield of New England watersheds in general ranges between 300,000 and 600,000 gals. per day per square mile.)

The apparent non-uniformity of the data relating to yield has been emphasized, not because such data or the methods of using them should not be made a basis for estimating, but to call attention to the fact that the estimates cannot be expected to give more than general and approximate results. When the data are meagre or unreliable, any works based upon them should be more or less tentative in design and construction, and capable of being modified as the developing conditions require.

The data required for estimating yield will be found in the annual Reports on Hydrography of the U. S. Geological Survey, the Meteorological Reports of the Weather Bureau, Reports of the Geological Surveys of New Jersey and of several other States, of the Departments of Agriculture of several Western States, and the Reports of the water-works departments of various cities, particularly in the New England States.

Art. 32. Run-off from Storms.

If a storage-reservoir is to retain all the yield of a given area it must be sufficiently large to hold the entire run-off

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from the heaviest downpours, over and above what may be already stored in it during the wettest years. Probably no reservoir has been constructed to do this, and the overflow must therefore be capable of passing the maximum storm run-off. It is hence necessary to know what the maximum rate of run-off from the heaviest storms will be. The maximum run-off from large drainage areas in the north frequently occurs, not from the maximum precipitation, but from a heavy warm rain falling upon a ground covered with considerable snow—the spring freshets. At this time about 8 inches of snow is equivalent to one inch of water. Hence two feet of snow carried off in 24 hours by a rain would add to the run-off the equivalent of 3 inches of additional rainfall running off in that time, or of $\frac{1}{8}$ inch per hour additional to the actual precipitation, if all of the latter be considered to run off. Probably the addition for snow of $\frac{1}{8}$ to $\frac{1}{4}$ inch per hour to the run-off from the rainfall will be sufficient for most cases. If the surface under the snow be frozen 90% or more of the rainfall may be yielded.

The maximum rates of rainfall generally last for five to fifteen minutes only. Assuming a velocity of flow over the surface of 2 feet per second (probably the maximum ordinarily obtained), and in the stream of 4 feet (although this may reach 8 feet per second), with 1000 feet of watershed above the head of the creek, at the end of 15 minutes of maximum rainfall the first rain falling at the head of the basin would have travelled 2600 feet, and at all points above its position at that time the run-off would be that due to the maximum rate of rainfall. But at all points more than 2600 feet from the basin-head, the run-off from only the nearer part of the basin would be at that rate, the upper part contributing to the flow at a rate due to the rainfall previous to the beginning of the maximum. Thus, at 3800 feet from the basin-head the flow would be that due to a rainfall for a certain five minutes on the upper 600 feet of basin, for the next five minutes on the next lower 800 feet of basin, for the next five minutes on the next lower 1200 feet, and for the last five minutes on the nearest 1200 feet of basin. At any point in the run-off channel the maximum flow will be approximately the product of the area of shed above such point, the maximum average rate of precipitation for the time consumed by the run-off in flowing to this point from the most distant point of the basin, and the proportion of rainfall running off. The determination requires a knowledge or assumption of the proportion of rain yielded as surface flow, and the velocity of flow of the maximum run-off. No close approximation to this latter can be made except from actual obervation on each watershed. The run-off from heavy downpours of short duration may be 60 or 70 per cent on steep clay or stony hillsides, and even 90% or more on rocky or frozen ground; while for flat slopes of loose soil 30% may be the maximum amount.

It is evident that the larger the drainage area the less will be the rate of precipitation used and hence the rate of run-off per square mile. Distance is as large a factor of this rate as is area. From a small drainage-basin 10,000 feet long, with a maximum velocity of run-off of 2 feet per second, the length of time for which the maximum rate of rainfall is to be considered is 5000 seconds, or 1 hour 23⅓ minutes. At Mt. Carmel on July 2, 1897, 5.03 inches fell in 1½ hours. Assuming this as a maximum rate and 50% running off, we have a run-off of 1.68 cubic feet per second per acre, or 1075 per square mile. This rate of precipitation—3.36 inches per hour—can probably be considered a maximum for the New England and North Atlantic States.

Several empirical formulas have been devised to express the maximum rate of run-off from a given area. A few of these are:

Fanning's formula.	$Q = 200M^{\frac{5}{6}}$;
Dredge's "	$Q = 1300\frac{M}{L^{\frac{2}{3}}}$;
Col. Dickens' "	$Q = CM^{\frac{3}{4}}$;

in which Q = cubic feet per second yielded from the whole area;

M = area of watershed in square miles;

L = length of watershed in miles;

C = 200 in flat country, 250 in mixed country, 300 in hilly country, for a rainfall of 3.5 to 4 inches; or 300 to 350 for a 6-inch rainfall.

In the above example, if the watershed contain 3 square miles, being 1.9 miles long, the maximum rate of run-off as calculated by the above formulas would be:

by Fanning,	$Q =$ 500	cu. ft. per second;
" Dredge,	$Q =$ 2550	" " " "
" Dickens,	$Q =$ 684	" " " "
" above solution,	$Q =$ 3225	" " " "

As the size of the drainage area increases and the rate of precipitation used consequently decreases, these formulas will give quantities more nearly approaching those obtained by the above analytical method, and when an area 20 or 25 miles in length is involved, that of Fanning and the analytical method would give approximately similar results. But for small areas the above formulas will generally give unsafe results, and the method outlined is recommended for designing waste-weirs.

ART. 33. STORAGE.

In Art. 27 was given an illustration of the reason for storage and the amount required for private use. The reason is the same for storing public supplies, but the amount stored is of course vastly larger, and the length of drought provided for is usually longer. The storage-reservoirs for the San Francisco water-supply have a united capacity equal to the total consumption for three years. In the East two thirds this capacity or less would be considered sufficient, however, the annual precipitation being more uniform.

A measurement of the drainage area having been obtained, and a decision formed as to the probable average, minimum and maximum yield, by both year and cycle, and the consumption to be provided for being determined, a calculation of the storage required can be made. If the minimum annual yield is equal to or greater than the desired consumption, storage for only the dry season of one year of drought is required; if the minimum *daily* yield equals the maximum daily consumption, no storage is required; but if the assumed consumption is nearly or quite equal to the mean yield, all the surplus from the years of greatest rainfall must be stored and carried until times of drought. In many cases it may be advisable to construct a reservoir in such a location or of such capacity that it is capable of tiding over one dry season only, if this be ample for the consumption for a few years to come; and when the capacity of the reservoir is almost reached by the consumption, a reservoir of larger capacity may be built, and better adapted to the watershed in question because based on data meantime collected; the interest on the additional sum which a larger original reservoir would have cost being saved during this period.

In making the calculation for storage, evaporation from the reservoir must be considered, and may be added to the consumption. It is of course proportional to the area of water-surface in the reservoir, and for the preliminary calculation this area must be assumed; usually as a certain proportion of the drainage area, which relation will vary with the average depth of the reservoir, and this with the nature of the ground at the reservoir site—whether the side slopes are more or less abrupt. In the New England and Middle Atlantic States the maximum reservoir required would have a capacity 150 to 175 per cent of the mean annual yield. An inspection of the yield for a series of years at any location will give the approximate capacity required to permit the consumption to equal the average yield. This capacity divided by the average depth of reservoir will give its area. About one tenth that of the drainage area may be considered a maximum which will rarely be exceeded; and one twentieth may be taken as an average maximum. The amount of evaporation in cubic feet will be this area times the rate of evaporation, both expressed in feet. The annual rate of evaporation east of the Mississippi probably never exceeds the mean rate by more than 10 or 15 per cent, although in the Western States it may by 30 or 35 per cent. (See Table No. 27, page 89.) To be on the safe side the maximum annual rate may be used, and apportioned to the months if monthly yield and consumption are to be used in the estimate. Table No. 28, page 89, gives the monthly rates for Boston and Sweetwater, and Table No. 29, page 91, the proportion of the annual total evaporation occurring each month.

The maximum evaporation, annual and monthly, area of reservoir, monthly yield and consumption (see Table No. 6, page 38) having been decided upon, a table can be prepared similar to that in Art. 27, the monthly consumption and evaporation being combined for the third column, and yield substituted for precipitation in the second. In estimating yield it should be borne in mind that *all* the rainfall falling upon the *reservoir* is stored.

On the Sudbury basin the mean evaporation from water was 171% of the mean yield. Assuming the reservoir area as $\frac{1}{20}$ the watershed, we would have the loss by evaporation about 8.5% of the yield; and since the size of the reservoir will vary with the consumption, which cannot exceed the mean yield, we may assume with little error that, on similarly located reservoirs, the evaporation loss from the reservoir will not exceed 8 to 10 per cent of the consumption.

There will be some loss from a reservoir due to seepage through the dam. That into the ground may be considered as additional storage; for, although a part of this may be absorbed by vegetation, the proportion will probably be little if any greater than the loss by evaporation from the reservoir. Through a masonry dam there will be a little loss, but it should be inappreciable. Through an earthen embankment, however, the loss may be considerable. The amount so lost will depend upon the character of this embankment, which should be so constructed that the daily seepage shall not be more than 10 gals. per square foot of vertical longitudinal section of embankment. If the reservoir be ten times as long as the length of the dam, and this length be 100 times the average height of the dam, this would give a daily loss by seepage of .01 gal. per square foot of reservoir area, or 3.65 gals. per square foot yearly, or say 6 inches; or about 1.3% of the yield. (These figures are for New England only.) With good materials and workmanship the seepage may be reduced to 5 or even 3 gals. per vertical square foot of embankment.

Evaporation and seepage combined may be assumed at 10% of the consumption for the New England and Central Atlantic States, which will be a safe figure for use when accurate data are not obtainable. The consumption should be increased or the yield decreased by this percentage, or one similarly obtained in estimating storage and *available* yield.

For example, to estimate the maximum consumption available from the Sudbury watershed: we have a mean yield of 30,003,580,000 gals. per annum or 82,200,000 gals. per day. Allowing 10% for loss as above, we have 27,276,000,000 gals. per annum or 74,727,000 gals. per day as the maximum available supply.

A convenient method of calculating the storage required, on a given watershed of known or assumed yield, to meet different rates of consumption, is the graphical or "mass diagram" one, the cumulative yield from the beginning of a dry period to the end of each month in succession being plotted on the ordinate of that month. Such a method is shown in Plate VI, using a cycle of the dryest eleven years on the Sudbury watershed, viz., 1878–88. (See Fig. 4, page 101.) The wavy line is the curve of cumulative yield, plotted from the records. (From this curve the greater yield in each winter and spring is very apparent.) A straight line FF' is drawn at an angle representing 30,000,000,000 gals. per year (the mean yield, also the assumed consumption plus loss by evaporation and seepage), and so located as to be tangent to the curve and nowhere intersecting it. The vertical distance between FF' and the curve at any point represents the amount which must be in the storage reservoir at that time if the assumed amount of consumption is to be continuously furnished. Thus it appears that at the beginning of this eleven-year cycle the reservoir must contain about 37,000,000,000 gals., or more than one year's consumption; and that the capacity of the reservoir must be at least 56,500,000,000 gals., being full during the spring of 1879. After this the reservoir becomes less and less full until November 1885, when it becomes empty, but after which time the precipitation is sufficient for the consumption. An inspection of the 16-year curve, Fig. 4, shows that the supply required in the reservoir on January 1, 1878, would not have been provided by the yield of the three previous years, but might have been accumulating for several years back.

If we assume the reservoir empty on January 1, 1878, we draw a straight line from C tangent to the curve and cutting

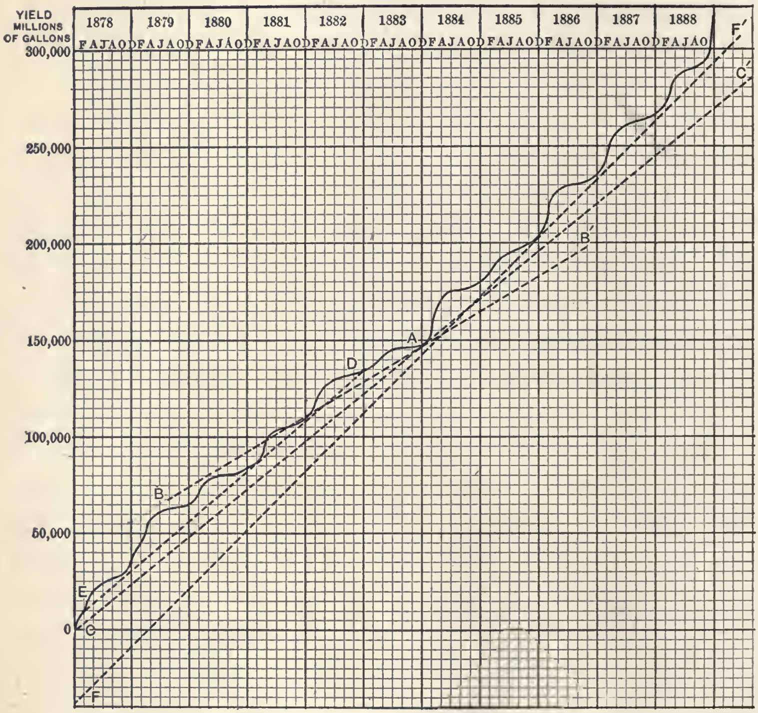

PLATE VI.—YIELD AND STORAGE DIAGRAM, SUDBURY WATERSHED.

it nowhere. This line is shown by CAC', representing by its angle of slope 24,500,000,000 gals. annually, or about 67,000,000 gals. per day; less 10% loss by evaporation and seepage gives 61,000,000 gals. per day available. We see by AC' that this rate is more than provided by the yield after January 1, 1884. We also see that the maximum storage required is 27,000,000,000 gals., and the reservoir would again be full and wasting water in April of 1887 and all through the spring of 1888. It is of course not advisable ever to permit the reservoir to become entirely empty, and a storage of at least 30,000,000,000 gals. should be provided in this case; and double this amount in the former one.

If but 10,000,000,000 gals. storage is provided, what will be the maximum uniform rate of consumption made possible? Place one end of a thread at the deepest loop in the curve, A, and swing it towards the left until the greatest vertical distance between the thread and any point of the curve above it is 10,000,000,000 gals.—as the line AB. As a check, continue this line towards the right end of the curve, making BAB' a straight line. No part of AB' should come above the right half of the curve; and if it does, either A is not at the period of greatest drought, or the storage is unnecessarily large. The rate represented by AB is about 17,600,000,000 gals. per year, or a consumption of 43,800,000 gals. per day.

This curve may also be used for finding the total, and mean rate of, yield for any length of time. Thus, from March 1878 (E) to December 1882 (D), inclusive, the total yield was 138,700 — 14,700, or 124,000,000,000 gals.; and the mean rate, represented by the angle of the line DE, was 25,700,000,000 gals. per annum.

This method is sufficiently accurate for all practical purposes, except that allowance is not made for the variations in monthly consumption and evaporation. An approximate determination may, however, be made by diagram, and more accurate figures obtained by calculation, as follows:

Taking the case of a reservoir with 10,000,000,000 gals. capacity, we see that the first storage begins in the middle of January 1882, and we may begin our table there. Quantities are in millions of gallons.

Month.	Amount of Yield, Mil. Gals.	Consumption and Loss.	Surplus added to Reservoir.	Deficiency supplied from Reservoir.	Amount in Reservoir at End of Month.
January, 1882	1446	607.3	838.7	....	838.7
February	5060	1257.7	3802.3	....	4641.0
March	6618	1239.3	5378.7	....	10019.7
April	1957	1324.9	632.1	....	10000
May	3011	1483.6	1527.4	....	10000
June	1193	1748 0	....	555.0	9445.0
July	201	1844.2	....	1643.2	7801.8
August	129	1748.0	....	1619.0	6182.8
September	691	1631.9	....	940.9	5241.9
October	697.7	1477.3	....	779.6	4462.3
November	472.3	1350.3	....	878.0	3584.3
December	733.6	1299.5	....	565.9	3018.4
January, 1883	780.5	1214.7	....	434.2	2584.2
February	2174.4	1257.7	916.7	....	3500.9
March	3755.0	1239.3	2515.7	....	6016.6
April	3044.5	1324.9	1719.6	....	7736.2
May	2185.3	1483.6	701.7	....	8437.9
June	676.9	1748.0	....	1071.1	7366.8
July	268.9	1844.2	....	1575.3	5791.5
August	183.1	1748.0	....	1564.9	4226.6
September	205.9	1631.9	....	1426.0	2800.6
October	433.2	1477.3	....	1044.1	1756.5
November	461.7	1350.3	....	888.6	867.9
December	451.1	1299.5	....	848.4	19.5
January, 1884	2319.9	1214.7	1105.2	....	1124.7
February	6197.3	1257.7	4939.6	....	6064.3
March	8824.3	1239.3	7585.0	....	10000
April	6437.3	1324.9	5112.4	....	10000

The monthly yield in this was taken from the record (Trans. Am. Soc. C. E., vol. XXVII. p. 276); the consumption is found by dividing 17,600 million gallons by 1.10 (10% for loss from reservoir), and the quotient by 12 for the monthly mean consumption; the rate for each month being the product of this by the percentage given in Table No. 6, page 38. The loss by percolation is taken as 1½% of the mean consumption. And the evaporation is found by multiplying 8½% of the mean consumption by the factors in Table No. 29, page 91; the sum of these losses and the consumption being given in the third column.

It is seen that by December 31, 1883, the reserve is reduced to 19.5 million gallons, but is quickly brought up to the limit of the reservoir by the spring run-off.

Another method of making this calculation is to correct the monthly yield of the watershed for the greater or less yield of the ponds or other water-surfaces thereon, including the reservoir, and to consider only the consumption as being deducted from the reservoir. For this purpose the area of water-surfaces relative to that of the entire watershed must be known, and also the yield of land-surfaces. (See Art. 31, page 98.) To the land-surface yield over the entire area (assuming seepage from ponds to contribute as much as percolation from rainfall on an equal area) is added the net yield or loss of each month from the water-areas.

The ponds and other bodies of water on the drainage area evaporate more water than do the earth. In New England the monthly rainfall minus monthly evaporation on a water-surface averages as follows, in inches of depth:

Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Year.
2.79	2.87	2.50	0.31	− 1.09	− 2.29	− 1.91	− 1.10	−0.79	1.09	1.67	1.91	5.96

It appears from this that during May to September the evaporation from a pond is greater than the rainfall upon it; and that during an average year the excess of rain, or yield, is about 6 inches only. In some sections of the country the evaporation is ten times the rainfall; and at Yuma, Ariz., it is thirty times; but these sections are limited in area.

ART. 34. QUALITY OF SURFACE-WATERS.

Fallen rain, whether it flows over the surface or through the ground, is changing its character continually. The surface flow takes into solution and suspension mineral and organic matters. The mineral is mostly in suspension, in the form of sand, clay, etc.; little being dissolved on account of the brief duration of contact. In the ground-water flow the impurities are mostly mineral because, the water passing slowly through the soil and over and through rocks, time for solution to take place is given; and because there is little or no organic matter in the soil below a depth of 12 to 18 inches. There are exceptions to each of these statements; water flowing over an alkaline soil will dissolve many of the salts present; and that flowing through underground beds of phosphate and fish deposits and of vegetable matter of prehistoric origin will often be high in ammonia and organic matter.

Surface flow may take into suspension large amounts of both mineral and organic matter, which it will later deposit on less steep surfaces where the velocity of flow is less. Underground flow, also, will generally purify a water of all or most matters in suspension, and often of some of those in solution.

With the exception of common salt and other alkaline salts, iron, and occasionally sulphur and alum, water seldom contains in solution sufficient of any mineral to make it injurious or unpleasant to the taste. Ground-waters in particular, however, frequently contain in abundance mineral matters which form a food for algæ and other vegetable organisms, which latter attain such numbers as to become obnoxious if not injurious.

The organic matter present is injurious when in a putrescent condition. In some cases, such as of water from peaty land, an objectionable color may be given by the vegetable matter although the water may have no injurious properties.

The most dangerous impurities are those due to pathogenic bacteria, which are ordinarily, if not invariably, derived from human excreta; and a watershed should be carefully examined for and guarded against such contamination. No surface privies or overflowing cesspools should be permitted. Deep, tight cesspools at a distance from any stream, and from any well (since the health of occupants of the watershed is very important to the consumers), should be compulsory for any scattered occupants; and the use of night-soil as fertilizer should not be permitted within such drainage area. If there is a village or any considerable congregation of houses on the watershed, these should be provided with sewers, and the sewage either so treated that no germs can reach the reservoir, or else discharged beyond the drainage area or below all impounding reservoirs. This is very important, since many epidemics of typhoid fever have been traced to single cases upon a watershed. One such epidemic of considerable violence was due to the depositing upon the snow of the excreta of a typhoid patient, which were washed into the reservoir with the spring rains.

Probably the greatest amount of impurity is found in surface-water in the spring, when that absorbed by snow from the air and ground (which absorption is continued while the snow lies) is added to that of the rainfall, and all passes over the frozen ground without any of the purification effected by underground flow.

Surface water, when stored in reservoirs, is subject to certain changes, most of them advantageous but some otherwise. Much of the matter in suspension is here deposited, if the reservoir be of such size that there are no perceptible currents. Together with the coarser matter many bacteria may be carried down, probably not entirely on account of their weight, but also because their food-matter is settling to the bottom. At Oberlin, O., for example, the number of bacteria was found to be reduced from 2000 per cubic centimeter in a 15,000,000-gallon reservoir to 426 in the effluent. In the Chestnut Hill Reservoir (Boston) in 1894 the average number of bacteria found at the surface, middle, and bottom were 77, 246, and 319 respectively; the surface-water at no time containing more than one half the number found at the bottom. The benefit of sedimentation to a water-supply is illustrated by the typhoid-fever epidemics at Philadelphia in 1891 to 1899, where the highest mortality was almost invariably found where water was pumped directly to the consumers, and the lowest where the capacity of the reservoir relative to the consumption was greatest.

Waters entering a reservoir from soils of different character, and as both surface- and ground-flow, will possess different characteristics. These waters largely intermingle, the more polluted being diluted by the purer, and to a certain degree chemical combinations resulting. For instance, the ammonia of a polluted water may be oxidized into nitrates by the free oxygen in a purer water; or ferrous oxide, by a similar addition of oxygen, may become insoluble ferric oxide and settle to the bottom.

In addition to these processes continual changes are being effected by the living organisms in the water, both vegetable and animal. The former consume only the mineral matters in the water, both those originally so and those resulting from the decomposition of organic matter (except that bacteria decompose organic matter also); the animal organisms subsist upon the organized matter, including other living animal and vegetable organisms. The lower organisms have by far the greater power of multiplication, and may increase more rapidly than the higher organisms for which they serve as food, and their death and decomposition result in a pollution of the water. As a familiar illustration, a considerable increase in mineral matter suitable for plant-food, or in the nitrogen resulting from the decomposition of organic matter. may suddenly cause the presence of vast numbers of algæ, which, not being accompanied by a similarly rapid increase in animal organisms which will consume them, cause gross pollution of the water.

If a reservoir is used without the flooded portion being first cleaned of all organic matter—leaves, bushes, stumps, roots, etc.—these furnish food for great numbers of organisms, but sufficient increase in the number of higher animals to keep the water clear is prevented by natural limitations both of propagation and of existence, especially by the small amount of oxygen in such water; and pollution thus caused will continue for many years. For this reason reservoirs should be carefully cleared before use, not only of the surface vegetation, but of the vegetable soil-matter or humus also. An investigation made by the city of Boston of the site of the Nashua reservoir indicated that below a depth of 9 to 12 inches there was little organic matter—in few cases more than 2%, but above this the amount was considerable; and consequently the top soil was removed to this depth at a cost of about $3,000,000. In cases where the expense of this seems prohibitive the reservoir should at least be thoroughly cleared of all vegetation, and all stumps and roots grubbed out. Clean sand or gravel spread over the soil forms an excellent and sightly bottom for a reservoir, and will to some extent prevent the evil effects due to organic top-soil; but the removal of all organic matter is decidedly preferable.

While water stands in a reservoir the top surface becomes heated in summer and cooled in winter more than do the lower strata. Wind stirs the water to a depth of 5 to 20 feet, and causes it to be warmed somewhat to this depth in summer, although the warmer water, being lighter than the cold, remains always near the top. In winter the cooler surface-water settles to the bottom, and the temperature thus becomes more nearly uniform, the bottom being generally somewhat warmer.

The following table gives the average temperature of the surface of several ponds and reservoirs in Massachusetts, and of the air at the same time, by months.

TABLE No. 37.

TEMPERATURE OF PONDS AND RESERVOIRS IN MASSACHUSETTS (DEGREES FAHR.).

	Jan.	Feb.	Mar.	Apr.	May.	June.	July	Aug.	Sept.	Oct.	Nov.	Dec.
Surface of water.....	35.3	35.3	36.7	44.3	57.7	67.9	73.7	72.9	66.9	55.2	44.1	36.1
Air...........	24.1	25.8	32.2	43.9	56.3	66.0	71.4	68.6	60.8	50.1	39.0	28.5

The higher temperature of the water is due to the sun (the air temperature being taken in the shade) and the longer retention of heat by water than by air. There were few variations from these average temperatures of more than 1° to 5°, the shallow ponds being generally the warmer.

Table No. 38 gives the temperature of Jamaica Pond (Boston) and Lake Cochituate at different depths, showing the variations referred to above.

TABLE No. 38.

TEMPERATURE OF LAKES AT DIFFERENT DEPTHS.
(Mass. State Bd. of Health.)

JAMAICA POND, July 14, 1891.

Depth............	Surface.	10′	20′	30′	35′	40′	47′
Temperature of Water.	75.4	75	54	42.4	42.	42	41.3
Percentage of Dissolved Oxygen.	100 (Saturated).	100	49	29.47	4.18	0	0

LAKE COCHITUATE, August 17, 1891.

Depth.............	Surface.	10′	20′	30′	40′	45′	50′	57′
Temperature of Water.........	74.7	66.4	53.6	49.3	48.2	48.2	45.7	44.8
Percentage of Dissolved Oxygen..	79.15	83.69	35.86	21.33	20.93	1.65	0	0

In summer the only motion in the water of lakes and reservoirs, aside from an inappreciable current, is that due to the wind. If the bodies of water are large and exposed this agitation may extend to a depth of 10 or even 20 feet, where the water attains this depth; but if the body of water be small or shut in by woods there may be little of such effect felt. It is found that all water below that which is so stirred up forms a comparatively stagnant layer. Since much of the organic matter in the water settles to the bottom, this often becomes very foul and all of the free oxygen here is utilized in nitrifying such matter. In Table No. 38 this is illustrated by data from two reservoirs; the upper 10 or 15 feet being stirred up by wind continually absorbs fresh oxygen, while retaining little organic matter to consume it; but the bottom 15 or 20 feet, containing much organic matter, is very low in oxygen. This fact is also illustrated in Table No. 39, by the difference in amount of organic matter, as represented by free ammonia, in the surface and bottom waters of several reservoirs and lakes.

TABLE No. 39.

DEPOSITS OF ORGANIC MATTER, AS FREE AMMONIA, AT THE SURFACE AND BOTTOM OF DIFFERENT BODIES OF WATER.

Location.	Date.	Depth of Water.	Depth of Deepest Sample.	Free Ammonia.	
				Surface.	Near Bottom.
Jamaica Pond, Mass.......	Aug. 14, 1890	57	50	0.0000	0.4720
Waban Lake, Mass.........	Aug. 27, 1889	36	35	0.0012	0.1760
Lake Cochituate, Mass.....	Sept. 18, 1890	65	60	0.0004	0.0680
Wenham Lake, Mass.......	July 24, 1889	46	45	0.0000	0.0560
Boston Reservoir No. 4....	Aug. 31, 1887	46	40	0.0000	0.0021
Lake Winnepesaukee, N. H.	Aug. 28, 1893	......	110	0.0000	0.0000

This table also shows, by the last two illustrations, that the stagnant layer is not necessarily foul, but only when organic matter is present in the water.

"As the surface-water cools in the autumn and becomes heavier than the water below the surface, vertical currents are produced which extend down to and somewhat beyond the depth where the water is at the same temperature as at the surface. These currents are nearly continuous and extend deeper and deeper as the season advances, until some time in November, when they extend to the bottom of the pond. After they have reached the bottom they continue to keep the water in motion for several weeks until the whole of the water in the pond has reached the temperature of maximum density." "In a lake with any considerable amount of organic matter in it and also in deep artificial storage-reservoirs, where the surface has not been stripped, the lower layers, which are quiescent during the stagnation period, gradually collect all the organic matter from the upper layers, and decay goes on until the oxygen is used up. The water becomes darker and darker, until by October it is very yellow, and generally has a disagreeable smell. Of course, when the great overturning comes, in November, all this bad water is brought to the surface, and the infusoria and diatoms begin to grow in enormous numbers, because the organic matter and oxygen are brought together and provide food for organic life. The same phenomenon takes place in the spring period of circulation, although on a smaller scale." (Fitz-Gerald on the "Temperature of Lakes," Trans. Am. Soc. C. E., vol. XXXIV.)

In Plate VII are shown the typical winter and summer temperatures of a lake that freezes.

The above explains the sudden presence in water-supplies of the "fishy" taste which is so unpleasant, it being caused by some matter—probably an oil—given up by the decomposition of many species of algæ.

If the water-supply be drawn from the surface of a reservoir from May to September, the purest water will thus be

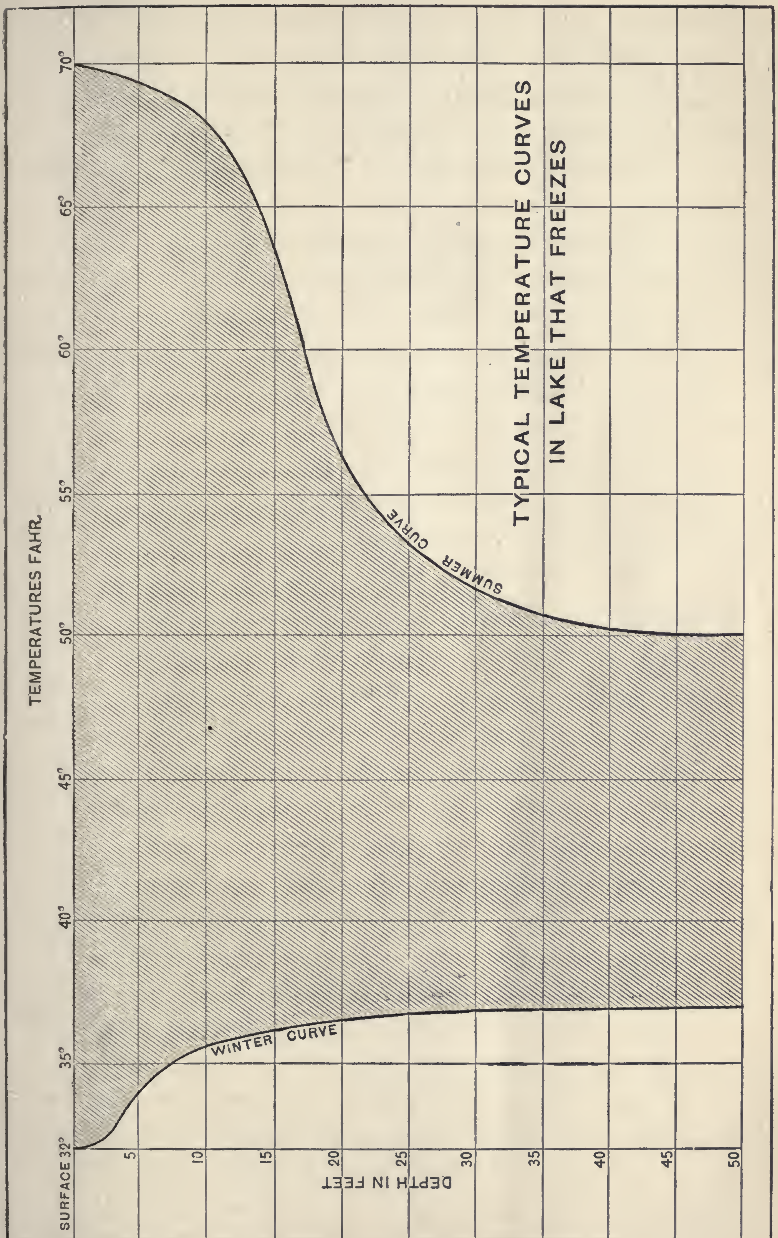

PLATE VII.—TEMPERATURE OF A LAKE THAT FREEZES.
(From Trans. Am. Soc. C. E., Vol. XXXIV.)

obtained. If now, just before the overturn, the bottom layer of impure water be drawn off through a waste-pipe, much of the fouling of the reservoir will be avoided. If neither the water nor the bottom of the lake or reservoir contain organic matter or nitrogen, all this trouble is of course avoided.

If much unoxidized organic matter remain in a reservoir when this is frozen over, and the access of additional oxygen to the water from the air is thus shut off, putrefaction may take place with its resulting gases; but this can happen only when the water is more impure than any supply should be.

Ice is not an important source of supply in this country, although it is in some extreme northern ones, and in certain localities in the Alps. A comparatively small amount is used in ice-water, however, and for this reason its purity is of importance. Impure ice is as dangerous as impure water, and is more commonly found in use as a beverage; some families using melted ice in summer as their principal drinking-water, which ice may have been obtained from a highly polluted pond. The popular idea that water is purified in freezing is but partially true, much of both organic and inorganic impurity frequently remaining in the ice. When water is frozen slowly, however, much of the impurity is excluded and is taken up by the remaining water, which thus becomes more impure. Hence the most impure ice is that frozen last, and is generally found at the bottom, if from a shallow pond, or at the centre of artificially frozen cakes. From deep ponds, however, the under side of the ice is purest, because the slight increase in impurity of the lower water caused by the freezing of the surface is more than offset by the greater percentage of purification effected by the slower freezing of the under ice. When ice is flooded, all the impurities in the flooding water must be contained in the ice. This refers to bacteria as well as to other impurities.

The following data (selected from Mason's "Sanitary Water-supply") shows the amount of purification by freezing in several instances.

	Total Residue from 100 c.c.	Loss on Ignition, Volatile and Organic Matter.	Inorganic Residue.	Per cent of the Mineral Matter originally in the water, yet remaining in the ice.	Per cent of Organic and Volatile Matter of the water, yet remaining in the ice.
Troy City supply...........	.0092	.0035	.0057	....	....
Ice from same...............	.0010	.0010	trace	0.	28.5
Very hard spring water......	.0540	.00	.0540	....	....
Ice from same...............	.0045	.00	.0045	8.3	....
Water from Erie Canal where public ice supply is taken..	.0112	.0033	.0079	....	....
Ice from above locality, used for public supply..........	.0067	.0025	.0042	53.2	75.7

Dr. Prudden found in ice which had been frozen for eleven days 1,019,403 bacteria per cubic centimeter; in that frozen seventy-seven days, 72,930; and in that frozen one hundred and three days, 7348 per cubic centimeter. He also found from 50 to 200 times as many bacteria in snow- or bubbly-ice as in clear, transparent ice free from air.

QUERIES.

9. The Sweetwater reservoir has an area of 530 acres when half full. Assuming this area as an average, correct the calculation in Art. 11 for evaporation from this, and find the area that can be irrigated.

10. If the Sudbury basin contain 2% of water-surfaces on its area, find the yield in 1883 of a neighboring watershed similar in all respects except that 10% of its area was water-surfaces.

11. How account for the percentage yielded being greater than 100 during certain months, as given in Table 33, page 99? Why is the May percentage lower and the fall percentage higher in the Potomac than in the Connecticut River basin?

CHAPTER VII.

RIVERS AND LAKES.

ART. 35. RIVERS.

SURFACE-WATER is generally collected as well up towards the head of a stream as possible, both because the greatest fall is thus obtained to the point of utilization, and because here are more frequently found the best locations for reservoirs and dams. This source of supply is most applicable to hilly or mountainous country, whose watersheds are sparsely occupied,—conditions generally associated with a poor and thin soil.

In the lower lands there are few locations for reservoirs, and pumping must generally be resorted to; moreover the streams here are usually of sufficient size to furnish an ample supply even in dry times. The water is therefore taken directly from the stream, and no storage is required, although it may be desirable for permitting sedimentation.

The quantity of water flowing in a river is that reaching it by surface or underground flow from all the drainage areas above on all its branches, less what may have been removed by evaporation, by seepage, and by man for irrigation and other purposes. A part of the flow may in some cases be underground, beneath and near the bed of the river.

The yield to the river will be the total yield of the drainage areas of all its tributaries, the estimating of which has already been discussed under the head of surface-water.

The evaporation will be that from its entire surface and from the surfaces of all tributary streams, ponds, lakes, and other bodies of water. (For the rates see Tables Nos. 27, 28, and 29.) The seepage will vary from almost nothing to the entire volume of flow, depending upon the character of soil, amount and character of sediment carried by the stream, and height of ground-water. In a clay or rock channel the seepage will be very small. In a sandy soil it may be great; but if loamy or clayey matter is carried to the stream by heavy rains, this will gradually be deposited as sediment upon the bottom and form an impervious channel.

If the ground-water stands level with or above the river-surface, there will be no loss by seepage, but rather a gain. Such ground-water, however, must be derived from the drainage area of the stream, and hence be included in the general calculation of yield. This ground-water will usually flow slowly both towards the river and down its valley.

In many Western rivers the flow is in places altogether beneath the surface during a large part of the year, the river-bed being dry except during rainy seasons. At certain points in the courses of many of these rivers rock or clay outcrops through the porous soil, and here the water is forced to the surface and flows in the channel, to disappear again further on where the impervious stratum again dips beneath the surface. Several of these rivers have no visible outlet, but simply disappear into the ground, from which the water is absorbed by vegetation and evaporated.

A few rivers flow for a part of their course through underground caverns, generally in the limestone; but these are so exceptional as to require no special consideration.

The underground flow of a river can either be utilized as ground-water (see Chapter VIII), or in some cases can be intercepted by a dam carried down to the impervious stratum and across the channel of pervious soil which affords it passage. Such a dam, which causes the porous soil above to act as a storage-reservoir, has been built across Pacoima Creek, Cal., the channel in bed-rock being at this place but 550 feet wide and filled with gravel to a depth of 40 feet.

An estimate of the quantity of water flowing in a river at various seasons can be made by deducting from the rainfall the evaporation, both from earth (including plant-consumption) and from water-surfaces; or by other methods of estimating yield referred to in Art. 31. But the only accurate method is by direct measurement, and this is of greater value the longer the series of years it covers. Variations in river-flow are illustrated in Tables Nos. 34 and 35, pages 100 and 102, and it is here seen that the maximum may be 500 times the minimum, and the latter but 5 to 10 per cent of the mean annual flow. If this minimum amount is not more than the consumption, an additional source must be obtained; or means provided for storing the river-water, either by dam in the river itself, or by a storage-reservoir into which the water is pumped. The latter is generally preferable, but frequently the more expensive of the two.

ART. 36. QUALITY OF RIVER-WATER.

The quality of rain-water when it reaches a stream has already been considered, but there are many changes in it continually taking place after this. As in the case of reservoirs, mineral and other impurities carried in suspension are deposited as the current velocity becomes less, but more slowly in rivers because of the greater motion in the water. The stratification found in reservoirs and lakes does not exist in rivers, the current keeping the water in constant circulation. For the same reason the temperature of rivers is more uniform at different depths, although varying more from month to month; the variation being between 32° and 80° in seven Massachusetts rivers. Also more oxygen is generally available for nitrification in a river than in a lake or reservoir, since all parts of the water are in turn brought in contact with the air.

In spite of these means of purification, however, the water of a river is generally less pure than that of the run-off contributing to it, owing to the impurities reaching it from the various farms and communities past which it flows. The most dangerous of these is sewage contamination, although that from slaughter-houses and rendering establishments is fully as offensive and is far from being harmless. Waste waters from dye-works and numerous other manufacturing industries may render a water totally unfit to drink. A minor source of impurity, although it may become an important one, is the waste from passenger-steamers and other boats.

Since each person excretes every day an average of .015 lbs. of free ammonia, .003 of albuminoid ammonia, .218 of solids, and .042 lbs. of chlorine, the pollution added by the sewage of a large community is seen to be considerable; but more serious still are the bacteria, millions of which are found in each thimbleful of sewage, and some of which may at any time be pathogenic.

The pollution from manufacturing establishments may consist of almost any acids, alkalis, or organic matters. A carpet, blanket, and cloth mill on the Schuylkill River used daily, a few years ago, 48,700 lbs. of organic matter in 18 different forms; 2520 lbs. of 21 different acids; and 950 lbs. of 6 different alkalis. Brass-works discharge considerable sulphate of copper, cyanide of potash, and oils. The principal waste from iron-works is sulphate of iron; from paper-mills come filaments of jute, cotton, and other organic matters, caustic soda, chloride of lime and sulphite; from woollen-factories the washing of the wool produces large amounts of organic wastes, and soda, alkalis, logwood, fustic, madder, copperas, potash, alum, blue vitriol, muriate of tin, and other dye-wastes are found in the waste-waters. This list might be continued indefinitely; but the appearance of most rivers receiving such wastes is evidence of the seriousness of the contamination.

The following table gives an analysis of the Passaic River, showing gross pollution due partly to manufacturing wastes, but even more to sewage pollution; and also an analysis of the relatively pure Hudson, although this receives the sewage of several cities and towns above Albany.

TABLE No. 40.

ANALYSIS OF PASSAIC AND HUDSON RIVER WATER.

	Ammonia.		Chlorine.	Nitrogen as		Total Solids.	Loss on Ignition.	Mineral Matter.	Oxygen required.	Bacteria per c.c.
	Free.	Albumin.		Nitrates.	Nitrites.					
Passaic, below Passaic Falls	.02947	.04220	.8116	.0342	.00296	10.856	3.831	7.025	.5694	747,000
Hudson, above Albany	.0030	.0087	.35	trace	.000	7.3	3.5	3.8	.376	

Although there is considerable movement of river-water, both vertically and across the stream, still the greater part of the suspended material transported, together with the bacteria, is found near the bottom; also an impure stream entering on one side of a river may travel for miles before being equally commingled with the purer water. These facts should be taken advantage of in locating an intake.

A great objection to many river-waters is the large amount of mineral matter in suspension carried in time of flood. The use of such water is in some cases avoided by providing storage-reservoirs holding sufficient clear water to permit the discontinuance of pumping when the river is muddiest. This is always after a rain, and may last for but a day or two at a time; the duration varying directly with the size of the drainage area above. The extreme variation in the amount of silt present in some rivers is illustrated by the Ohio, in which, in 1895, the maximum amount of suspended matter found was 531.1 parts per 100,000, the minimum was 0.1 part, and the mean 22.5 parts.

It is probable that manufacturing wastes and sewage are in most cases quite constant in amount, and hence the polluted water is most impure when the river is low. The quantity of organic matter washed from the banks, which may include considerable human excreta, will be greatest after a rain. There will in most rivers be a wide variation and sudden changes in the impurities found, both mineral and organic. Table No. 41 shows such variation for the Hudson River above any direct sewage-inflow.

TABLE NO. 41.

VARYING AMOUNTS OF IMPURITIES IN RIVER WATER (MASON).
(In parts per 1,000,000.)

Date.	Ammonia.		Nitrogen as		Chlorine.	Required Oxygen.	Total Residue.	Loss on Ignition.	Suspended Matter (Silt).	Temperature, Degrees Fahr.
	Free.	Albuminoid.	Nitrites.	Nitrates.						
Nov. 3, 1894..	.030	.087	.000	trace	3.5	3.76	73	35	88.4	.....
Dec. 15, " ..	.045	.150	trace	.15	4.5	13.00	107	42	68.8	36
Jan. 12, 1895..	.025	.080	.000	.10	3.5	7.65	43	39	0.0	.....
Feb. 5. " ..	.055	.100	.000	.15	.....	8.85	88	45	0.0	34.6
Mar. 4, " ..	.085	.150	trace	.10	.....	10.00	93	51	0.0	33.
April 5, " ..	.042	.235	"	.30	2.4	5.90	388	88	11.0	46.4
" 10, " ..	.058	.660	"	trace	.....	15.50	583	74	495.0	41.
May 8, " ..	.030	.205	"	.10	2.5	7.30	67	31	0.0	68.
June 5, " ..	.045	.120	.000	trace	3.5	8.70	78	50	0.0	71.
Sept. 20, " ..	.280	.320	.0015	.30	5.0	2.65	.....	.....	......	.....
Oct. 30, " ..	.055	.155	trace	.....	3.5	14.85	101	57	0.0	44

The record of April 10th shows the largest amount of albuminoid ammonia coincident with that of suspended matter, and hence probably caused by rain.

The increase of bacteria caused by rain washing them into a stream is illustrated by the Croton (New York City) water, which ordinarily contains about 35 bacteria per cubic centimeter, but after a hard rain as many as 7200 have been found. River-water generally contains more bacteria in winter than in summer.

ART. 37. LAKES.

A lake is generally but the broadening of the channel of a river or other stream, and the water entering a lake is but that of such stream. The quantity of water passing through a lake is no more than that flowing in its river; but when this latter becomes temporarily small in times of drought the lake acts as a storage-reservoir, and hence is generally preferable to a river as a source of supply, if the quality is equally as good.

In passing through a lake water often undergoes changes in quality which would not occur in the stream. Lakes are ordinarily found in a hilly country, where the currents of the streams are more or less rapid, while that through the lake is very slow. Suspended matter which was carried by the stream is therefore permitted in a lake to settle to the bottom, and the water is thus clarified; many bacteria being carried down during the sedimentation, or dying off from lack of food-matter. The water at the lower end of a lake is hence in most cases purer than that at the upper end, provided no pollution finds its way into the lake from the shores. There being no more water flowing through a lake than flows in the river in whose course it lies, there will be in the long run no more dilution of sewage or other impure water discharged into a lake than if the same were discharged into the river; but the water may become more pure in passing over a given distance, because of the greater time occupied and the greater opportunity for sedimentation thus afforded.

Most lakes are deeper than their rivers, and such effect as depth may have upon the quality of water is found in many or most lakes. Owing to this depth, to the size of a lake as compared with a river channel, and to the slight current movement, lakes offer better opportunities for locating water-works inlets than do rivers or smaller streams.

Like river-water, lake-water must ordinarily be pumped; except in the case of lakes on mountain streams, which act practically as natural reservoirs of surface-water. In fact, lakes and reservoirs have in most respects similar effects upon the quality of water; and most of the statements made in Art. 34 concerning reservoirs are applicable likewise to lakes.

QUERIES.

12. If the Passaic were flowing 1000 cu. ft. per second when the analysis in Table 40 was made, how many people were apparently contributing sewage to it, assuming that all the free ammonia was from sewage? Assuming the same of albuminoid ammonia? Of chlorine?

CHAPTER VIII.

GROUND-WATER.

ART. 38. WATER-BEARING STRATA.

THE rainfall absorbed by the soil of each catchment area, after percolating downward, continues to travel in some generally horizontal direction toward a stream, lake, or sea. Its underground passage is subject to many of the laws affecting surface-flow—its surface must fall in the direction of flow, and the velocity of flow is proportional to this fall; the hydraulic gradient of this flow cannot be, at any point, lower than the body of water into which the flow discharges; the water will seek the lowest accessible channels, but ordinarily fills the soil over large areas. There are the additional influences of friction in passing through the soil; the capillary attraction of this; and confined flow caused by super-strata of impervious material, the conditions then approximating those found in water-pipes or other conduits under pressure.

The amount of flow is dependent upon the size and perviousness of the catchment areas which contribute to it, and upon the precipitation upon those areas. It is in many cases, however, increased by seepage from rivers crossing the pervious stratum, the drainage areas of which rivers are not strictly parts of the catchment-basin in which this pervious stratum lies.

If AB is a stratum of sand or pervious sandstone between two strata of impervious clay or rock, the percolation from the catchment-basin or valley B will travel toward A and

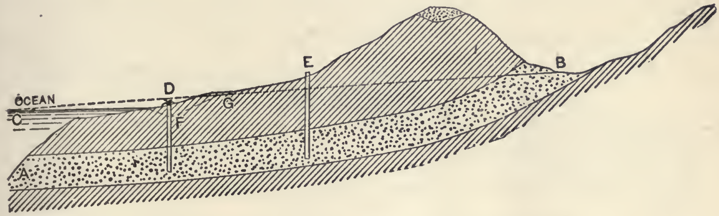

FIG. 6.—UNDERGROUND FLOW IN STRATIFIED ROCK.

emerge there. The hydraulic gradient will be the line BC, which will be straight if the material and thickness of the stratum AB be constant. A well at D would then overflow at any point below BC; while in that at E the water would rise to this line only and would need to be pumped. If water were drawn from D, the hydraulic gradient would be lowered at that point. If lowered to the point F, it is probable that some salt water would reach the well; as happened at Galveston, Texas. The rise and fall of the tide, causing a corresponding change in the hydraulic gradient, will cause a fluctuation in the height to which water will rise in a well situated similarly to E.

The amount of water emerging at A is in some places so great as to occasion fresh-water springs in the ocean; as off the coast of South Carolina, and the Gulf coast of Florida, where such springs boil up through a depth of 100 to 300 feet of water in such volume as to make it difficult to row a boat above them.

Fig. 7 shows a section through the St. Peter and Potsdam sandstones along a section passing through Streator, Ill., and Madison, Wis. These two strata furnish water to a large number of cities in the north-central part of the United States, as well as to numberless private wells; the water rising to the surface in a large number of wells in the Potsdam, and a few in the St. Peter sandstone.

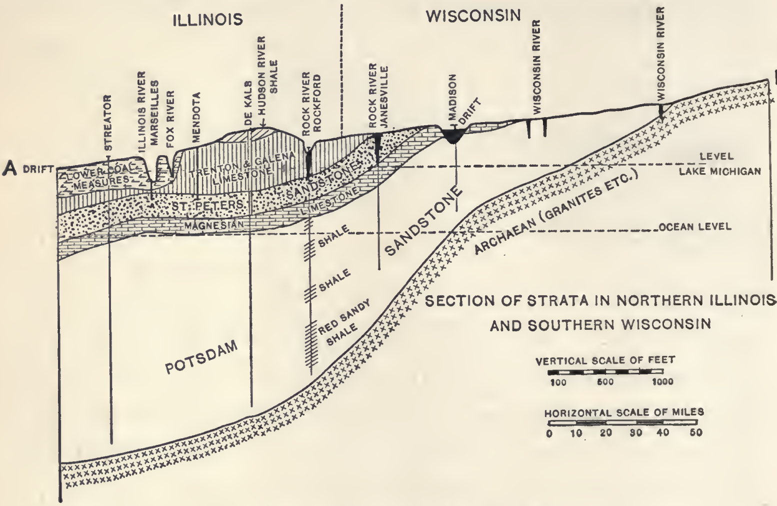

FIG. 7.—UNDERGROUND FLOW IN STRATIFIED ROCK.

In Fig. 8 is shown the condition in most river valleys, where the ground-water from the hillside flows towards and

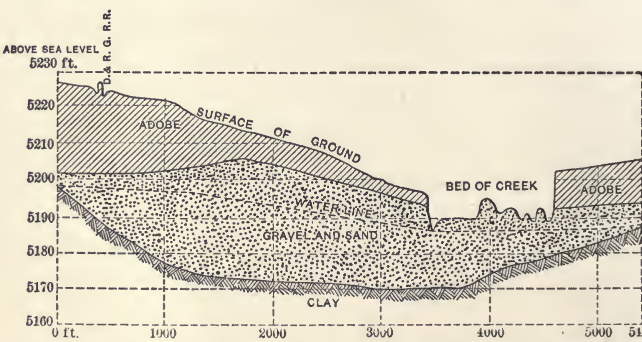

FIG. 8.—CROSS-SECTION OF VALLEY OF THE FOUNTAIN QUI BOUILLE, PUEBLO, COL.

into the river, ordinarily as a general seepage. The flow is in most cases approximately at right angles to the trend of the valley. But if the soil along the river be very porous the ground-water may flow parallel with the river, being in fact but a part of its flow; as the Platte River in Colorado and Nebraska; and in such cases the ground-water is more constant in its volume than the visible flow, and in many Western rivers is being more relied upon as a source of supply. The dam across the Pacoima Creek, already referred to, is an instance of the utilization of such flow.

The well in Fig. 9 would receive largely local drainage, although a considerable territory in some instances might

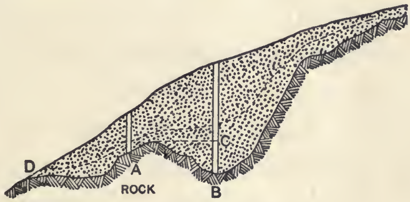

FIG. 9.—SHALLOW WELLS.

drain to and by such shallow wells. That at *A* would probably run practically dry in times of drought, while in *B* the water, when there was any underground flow, would stand at *C*, filling up to this point each time after the well was drawn upon and lowered; unless the draught exceeded the underground flow reaching this point, when the level at *C* would continually fall, but the depression at *B* would act as a reservoir, supplying water after *A* was entirely dry.

In New Jersey, south of the Raritan River, the strata for several hundred feet down are alternately blue and yellow

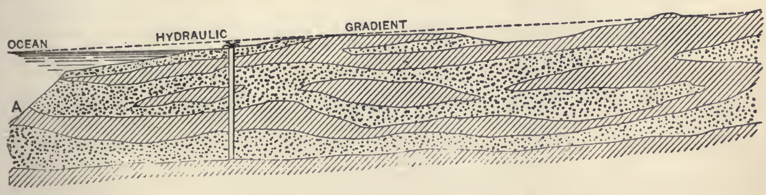

FIG. 10.—TYPICAL SECTION IN CENTRAL NEW JERSEY.

clays and marls, and sand of varying thickness. Most of the sand strata are water-bearing. The flow from a depth of 425

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feet in one well half a mile from shore rose to $6\frac{1}{2}$ feet above mean high tide, indicating an outlet into the ocean at least 8 or 10 miles distant (as at *A*, Fig. 10). Similar conditions are found along the Gulf of Mexico and over a considerable part of the South Atlantic coast. (See Fig. 11, page 142.)

Art. 39. Classification of Ground-waters.

Ground-waters may be generally classified as being derived from—

I. Underlying stratified rock deposits.
II. Drift.
III. Alluvial and marine deposits.

No. I may be subdivided into—

1. Water flowing through the pores of rock (Figs. 6 and 7).
2. Streams in caves and subterranean passages.

Nos. II and III may be further classified as—

3. Water derived from direct percolation, with little lateral transmission or hydrostatic pressure (Fig. 9).
4. Water in valley bottoms with considerable lateral transmission and small hydrostatic head (Fig. 8).
5. Water flowing in porous strata of alluvial or marine deposit or glacial drift beneath impervious strata (Figs. 10 and 11).

The water in class No. 1 is received from rainfall and flow of streams at and over the outcrop of the rock, and generally flows for long distances before finding an outlet. The whole rock, being saturated with water and giving it up slowly, acts as a reservoir, and supplies from this are but little affected by droughts lasting for a few months or even for a year or two. The outcrop often covers large catchment areas,—as the St. Peter sandstone, from which a large number of wells in

Illinois, Iowa, and Wisconsin draw their supply; and underlying this the Potsdam sandstone, which has an outcrop of about 14,000 square miles in the upper Mississippi valley, and which furnishes a large supply in the States just named and in Minnesota and Michigan. In Ohio and Indiana the Niagara and Trenton limestones furnish abundant supplies; and in the Dakotas and northern Nebraska the Dakota sandstone.

Class No. 2 derives its waters largely from surface streams and direct percolation, which find sink-holes or open seams in the rock, and dissolve out passages for themselves. This generally occurs in limestone, and as the water is not under pressure and is in narrow and scattered streams, it is seldom used for supply except where it emerges as springs or rivers.

Wells of the third class are generally found in pockets of drift and have only local sources of supply. They are shallow and feel quickly the effect of dry or wet seasons. They are not generally suitable for public supplies; and if used, the contributing surface must be carefully guarded from pollution.

Wells of the fourth class furnish the supply to a large number of cities in this country. The line between this and class No. 3 cannot be sharply drawn, but this class has a much more extensive drainage area. The alluvial deposits in the southern and central valleys and the drift in the northern third of the United States afford many abundant supplies of this class; most of which, however, are being or should be abandoned on account of the danger of pollution. Shallow wells and galleries are constructed for utilizing this source of supply; and the interception of subsurface river-flow, as at Pacoima, also belongs to this class.

The waters of the fifth class are probably used more extensively than those of any other. The Atlantic coast States from New Jersey to Georgia, and the entire Gulf coast, offer this source of supply in the alluvial and marine deposits at depths of 50 to 1000 feet. Pensacola, Fla., obtains from this source 2,000,000 gals. daily; Memphis, Tenn., 10,000,000 gals.; Brooklyn, N. Y., 25,000,000 gals.; Fort Wayne, Ind., 6,000,000 gals. In western Florida, Mississippi, and Louisiana water is found in abundance in sand and fine gravel interspersed with strata of vari-colored clays. West of this around the Mississippi River silt predominates and the wells yield scantily. In eastern Florida the wells penetrate cavernous limestone, spurs of the Georgia mountains, and come under the first and second classes. North of this are again water-bearing sand strata. A typical Gulf section is shown in Fig. 11, running north and south through Pensacola. Here the hydraulic gradient falls about 1 in 4500

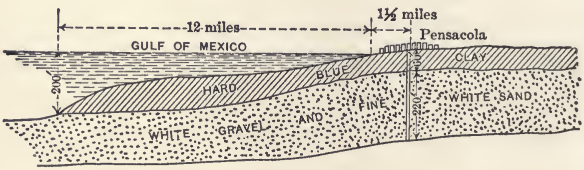

FIG. 11.—SECTION OF WATER-BEARING STRATUM; PENSACOLA, FLA

($s = .0002 +$), the water rising 16 or 17 feet above sea-level. At Natchez, Miss., the water-bearing sand stratum is but 40 feet thick.

The impervious strata are generally clays and marls, with occasionally hard-pan, and vary in thickness from 5 to 50 feet or more. The pervious material, generally sand, sometimes runs in streaks in irregular courses, and again spreads out into thick strata miles in width (Fig. 10). There can therefore be no certainty of finding a particular stratum by boring at any given point, but this is largely a matter of chance. The thicker and more important water-bearing strata generally extend over large areas, however, and may be found with considerable certainty at any point within a district whose outer limits it is known to underlie.

ART. 40. FLOW OF GROUND-WATER.

The general direction of flow of waters of the fourth and fifth classes is, in the majority of cases, towards the sea, along the line of glacial motion if in drift, or diagonally across and down a valley if in alluvial deposits.

When a well is pumped, it draws water from the sides and below as well as above, and to an extent depending upon the amount the water-surface is lowered below the hydraulic gradient. Waters of the first class follow in their flow a course from their outcrop to the ocean or other point of discharge, which is ordinarily the general direction of pitch of their pervious strata, but which may or may not be that of the ground-surface. Those of the second class follow the faults or seams and the general pitch of the strata, occasionally sinking from one stratum to the next lower.

The velocity of flow depends upon the slope of the water-surface; the size and uniformity of the grains of sand or gravel, or of the pores of the rock through which it flows; and upon the temperature, although this ordinarily varies but little. In finer gravel and sand the velocity is found to be directly proportional to the slope, while in coarse gravel it is more nearly proportional to the square root of this. The velocity in sand may be represented by the formula

$$V = cd^2s,^*$$

in which $V =$ velocity in feet per second;

$c =$ a coefficient, about 0.29 as determined by a few experiments;

$d =$ the effective size of the sand-grains in millimeters. (Effective size of sand "is such that 10% of the material is of smaller grains and 90% is of larger grains than the size given.")

$s = \frac{h}{l} =$ sine of the slope of the hydraulic gradient.

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* Allen Hazen, in Report of Mass. Bd. of Health.

This formula is not considered applicable when d exceeds 3.

The relative area of open spaces in sandy soil through which the water flows determines the quantity of flow. In a given cross-section this area will generally range between .35 and .60 of the total area. The quantity of flow, Q, per unit area of vertical section would then be $.35V$ to $.60V$. On Long Island, where the Brooklyn water-supply is obtained, s is about .0002 in dry weather to .002 in wet. This would give a velocity of flow in dry seasons, assuming $d = 0.5$, of $V = 0.29 \times .25 \times .0002 = .0000145$ feet per second, or 1.25 feet per day; and $Q = .40 \times 1.25 = 0.5$ cubic feet per day per square foot of vertical section. In wet seasons these values might be ten times as great; s being much greater.

Slopes in sand of 30 to 50 feet per mile are found ($s = .0057$ to $.0091$), the slope generally increasing with the fineness of the material. In valleys having gravelly soils the cross-slope is generally very flat, while the longitudinal slope is practically that of the river.

Through rock the velocity of flow is less than through sand, owing to the presence of the interstitial cementing material, but practically nothing definite is known upon this subject. In the Dakota sandstone the distance from the outlet in the Missouri River to the catchment outcrop in the Rocky Mountains is about 500 miles, and the difference in elevation about 5000 feet, giving an average value of s of .002; but it is thought that the hydraulic gradient is steeper than this near the outcrop, since it is flatter in Nebraska; probably because of great irregularities and faults in the strata in and near the mountains.

If the quantity of flow through a given material due to different water-slopes is known, we can approximate the amount of water available from such material in a given locality by sinking two wells in the line of flow, but some distance apart, and noting the water-level in each; or, if the direction of flow is not known, both this and s can be determined by sinking three wells at approximately the corners of an equilateral triangle. The velocity of flow can be found approximately by noting the time elapsing after placing salt in one well before it makes its presence known in another directly below it in the line of flow. If the upper well is artesian, or flowing, rock salt may be inserted in a bag and lowered into this well, the flow from it being then immediately stopped.

If a well be pumped, the water-surface is lowered below the original hydraulic gradient, the plane of the gradient being drawn down in the manner shown in Fig. 12; the extent and depth of the depression increasing with the amount of water pumped.

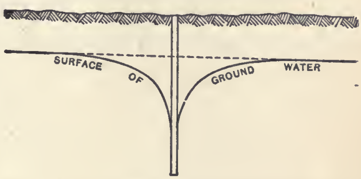

FIG. 12.—EFFECT OF PUMPING ON GROUND-WATER.

Since the velocity of flow must increase as the well is approached, the slope of the water-surface increases correspondingly, and a curved surface is formed, the shape of which depends upon the form of the equation for V. In Brooklyn, in 1886, the water-level 4300 feet from a well which was being pumped fell 6 inches, at 2300 feet the fall was 26 inches, and at 300 feet it was 56 inches. In another Brooklyn well the base of the depression was about 4000 feet in diameter when the water was lowered 8 feet at the well. In another the diameter was 5000 feet when the water was lowered 15 feet.

Some time must elapse after pumping begins before the depression assumes a final shape, since the water formerly filling it must first be exhausted. Similarly, the depression will fill gradually when pumping ceases. Thus, a set of wells in Brooklyn was pumped for twenty days, at a rate of 5,000,000 gals. per day, the ground-water being lowered 14 feet; and the depression was not filled until the twelfth day after pumping ceased. If the diameter of the depression was 5000 feet, and the soil was 33% voids, and 60% of the contained water was yielded, the amount of water to be replaced was probably about 95,000,000 gals.—approximately the total amount pumped. If this was the case, the depression might not yet have assumed its permanent form.

If wells be located too near each other, their cones of depression will intersect, and the flow of each or some will be reduced. They should be placed across the direction of flow, otherwise the upper wells will leave little water for the lower ones, which thus become almost useless.

Art. 41. Wells.

The method ordinarily adopted for intercepting ground-water of the first and fifth classes, and frequently of the third and fourth classes, is that of sinking wells into and through the water-bearing strata. For waters of the third and fourth classes large dug wells are frequently employed, walled in with brick or stone (see Fig. 66, page 398); but for deep wells, and in many cases for shallow ones, small pipe-wells 2½ to 12 inches diameter are sunk (see Plate XVII, page 397). These latter are frequently carried to great depths and through all kinds of material. At St. Augustine, Fla., is a 12-inch well 1400 feet deep through shale and limestone; at Charleston, S. C., is a 5½-inch well 1900 feet deep; at Atlanta, Ga., is one 2044 feet deep; at Paris, France, is one 2359 feet deep; at St. Louis is one 3850 feet deep; in West Virginia is one 4500 feet deep (dry); and at Pesth is one 8140 feet deep. Table No. 42 gives a list of some of the larger cities and towns of the United States using ground-water as a supply.

TABLE No. 42.

SOME OF THE LARGER CITIES AND TOWNS OF THE UNITED STATES HAVING SUBTERRANEAN SOURCES OF WATER-SUPPLY.

(D. W. Mead in Trans. Am. Soc., C.E., vol. xxx.)

City.	State.	Population, 1890.	Source.	Geological source.
Anniston	Alabama	9,998	Springs	
Appleton	Wisconsin	11,869	3 artesian wells	St. Peter sandstone
Akron	Ohio	27,601	Artesian wells	
Atlantic City	New Jersey	13,055	Large well	
Aurora	Illinois	19,688	2 artesian wells	St. Peter and Potsdam sandstone
Beloit	Wisconsin	6,315	Large well	Trenton rock
Big Rapids	Michigan	5,303	Drive wells	Drift
Brooklyn	New York	806,343	" "	"
Brunswick	Georgia	8,459	Artesian wells	
Cedar Rapids	Iowa	18,020	" "	Potsdam sandstone
Charlestown	S. Carolina	60,000	3 artesian wells	
Clinton	Iowa	13,619	4 " "	Potsdam and St. Peter sandstone
Columbus	Ohio	88,150	Filter-gallery	Drift
Dayton	"	61,220	65 8-in. tube-wells	"
Des Moines	Iowa	50,093	Large well	
Dixon	Illinois	5,161	2 artesian wells	Potsdam sandstone
Dubuque	Iowa	30,311	Artesian wells	" "
Fond du Lac	Wisconsin	12,024	5 artesian wells	St. Peter sandstone and drift
Freeport	Illinois	10,189	18 drive wells	Drift
Fresno	California	10,818	10 artesian wells	200 to 600 feet deep
Galena	Illinois	5,635	Artesian wells	Potsdam sandstone
Galesburg	"	15,264	Drive wells	Drift
Grand Island	Nebraska	7,536	" "	Platte River Valley
Jamestown	New York	16,038	Artesian wells	
Jackson	Michigan	20,793	" "	
Joliet	Illinois	23,264	Drive and artesian wells	Drift and St. Peter sandstone
Jacksonville	Florida	17,201	4 artesian wells	Cretaceous strata
Kearney	Nebraska	8,074	Drive wells	Platte River Valley
Kalamazoo	Michigan	17,853	2 large wells	Drift
Lincoln	Nebraska	58,000	2 " "	"
Macon	Georgia	22,746	Springs	
Memphis	Tennessee	64,495	Artesian wells	Cretaceous strata
Mansfield	Ohio	13,473	16 wells, 2 springs	Drift
Montgomery	Alabama	21,883	Artesian wells	
Michigan City	Indiana	10,776	Springs	Drift
Muncie	"	11,345	Artesian wells	
Manistee	Michigan	12,812	50 drive wells, 1 large well	Drift
New Brighton	New York	16,423	76 drive wells	"
Nashville	Tennessee	76,168	Underflow	Cumberland River, filter-gallery
Oshkosh	Wisconsin	22,836	Artesian well	Lake Winnebago for fire protection
Peoria	Illinois	41,024	Large well	Gravel in ancient bed of Illinois River
Pine Bluff	Arkansas	9,952	Drive wells	
Pensacola	Florida	11,750	16 drive wells	
Pittsburg	Kansas	6,697	Artesian wells	
Rockford	Illinois	23,584	9 artesian wells	Potsdam and St. Peter sandstone
Savannah	Georgia	43,186	Artesian wells	
Stockton	California	14,424	Artesian well	
Selma	Alabama	7,622	3 artesian wells	
South Bend	Indiana	21,819	32 " "	Drift
Springfield	Illinois	24,963	Well and filter-gallery	"
Sioux City	Iowa	45,000	Drive wells	
Winona	Minnesota	18,208	2 large wells	Mississippi underflow

Wells are used for irrigation to a small extent in the West. In the fourteen Western States and Territories, in June 1890, 3930 deep wells furnished irrigation for 51,896 acres, or 1.43% of the total area irrigated. The total number of deep wells in those States at that time was 8097, their average depth 210 feet, and average discharge 54.43 gals. per minute; giving a duty of 109 acres per second-foot.

In order that all surface-water may be excluded, the well-casing should always be tight from the surface down to a thick, impervious stratum of clay or rock, with which it should make a tight joint; or, if there are no such strata, the entire casing should be tight except at and near the bottom, to exclude local percolation which has not been filtered through considerable depth of soil. For the same reason the top should be tightly closed, or should be carried above the reach of surface-water.

Wells sunk to ground-water whose hydraulic gradient lies above the surface at that point, and which consequently overflow, are generally called artesian wells. But by many this term is used for either class of wells; the difference after all lying solely in the elevation of the ground-surface at that point.

It is almost always necessary or desirable to pump from a deep well, whether artesian or not; since it is generally cheaper to increase the supply from a few wells by pumping than to sink more wells. There may be exceptions in the cases of small supplies which one or two artesian wells can furnish by gravity.

The amount of water obtainable from underground has already been referred to. This cannot all be intercepted by wells in any practicable way, 10 or 25 per cent probably being the maximum amount obtainable under any conditions. The amount obtainable by a given well can never be predicted with any accuracy except by comparison with other wells in the same stratum.

For a short time after beginning to pump a well the delivery may increase, owing to the opening of channels in the water-bearing material. But if the pumping increase the temporary yield beyond the natural local ground-flow, and especially if this be true for a number of wells in one neighborhood, the ground-storage may be drawn upon, the hydraulic gradient lowered, and the natural yield diminished. Thus, at Rockford, Ill., where a number of wells are sunk into the Potsdam sandstone, the gradient fell 10 feet in six years.

A 10-inch well in Brooklyn 110 feet deep has yielded 150,000 gals. per day; a 5-inch well at Rockford, Ill., 1800 feet deep, 259,000 gals., and a 6-inch one at the same place 364,000 gals. per day. The Ponce de Leon 12-inch well at St. Augustine yields 10,000,000 gals. per day; and at Charleston, S. C., a $5\frac{1}{2}$-inch well 1900 feet deep yields 1,250,000 gals. daily. The amount of yield depends to a certain extent upon the freedom with which water can enter the well, and has often been diminished or altogether stopped by a choking of the inlet openings. Increasing the size of the well-casing increases the flow chiefly by decreasing the velocity and friction within itself. If a deep-well pump is to be used, the size of the casing must be adapted to this. (See page 397.)

Art. 42. Infiltration-galleries.

When the ground-flow near the surface, as in Fig. 6, page 137, is to be used for a supply a number of wells, driven or dug, may be employed; but a larger quantity can ordinarily be obtained by use of a long crib placed at right angles to the direction of flow and below the ground-water surface (see Fig. 67, page 399). This is generally placed near a river, not to utilize the river-water, which in most cases is impossible, but because the ground-flow increases in volume as its outlet is approached. Infiltration-galleries are made of wood, brick, or stone in the shape of a small, long gallery, with openings in the sides and bottom through which the water enters. A well-located gallery will intercept almost all of the ground-flow in its locality, which is pumped from it direct or led from it by pipes or channel to a pump-well.

An infiltration-gallery may be considered as a large, oblong well, and the statements above relative to ground-flow apply to these as well as to wells.

At Newton, Mass., and some other cities wells are sunk along the line of the filter-gallery and discharge into this, thus uniting the supplies.

An infiltration-gallery is sometimes placed across the channel of a river which has a large underground flow, to intercept this. This method is particularly applicable to some of our Western rivers, where the underground flow is at most times greater than the visible. The underflow of the Platte River was thus used for the Denver water-works. In many or most of such cribs considerable water is drawn from the visible supply, when there is any.

The great probability that water so near the surface will be polluted has led to the abandoning of many infiltration-galleries in the Eastern States; and they are recommended for such localities only as are beyond any sources of pollution. The probability of such water being polluted in and near cities is shown by the following analysis of water from the subdrains of the Framingham sewers before the latter were put into use, the pollution probably being from cesspools.

Ammonia.			Nitrates.	Nitrites.	Total Nitrogen.	Chlorine.
Free.	In Solution.	In Suspension.				
.0648	.0058	.0000	.6000	.0036	.6665	3.62

Art. 43. Springs.

When the stratum through which any underground water flows comes to the surface at a point lower than the catch-

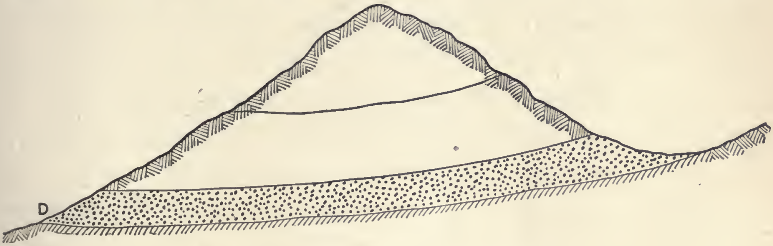

Fig. 13.—Spring at Outcrop.

ment area, the water emerges as a spring, as at *D*, Fig. 13, or as seepage. Also where faults occur the water may rise vertically to the surface, as at *A*, Fig. 14 (but is more likely

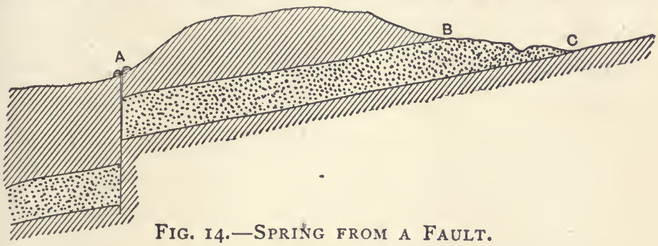

Fig. 14.—Spring from a Fault.

to fall to a lower pervious stratum). Such a spring at San Antonio, Tex., yields 50,000,000 gals. daily. Underground water may also rise through a stratum of clay or hardpan, as at *D*, Fig. 15.

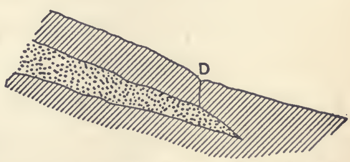

Fig. 15.—Spring in Hardpan.

It is evident that spring-water is the same in origin and character as water obtained by wells from the same stratum; those shown in figures 14 and 15 being practically natural wells.

Springs may be simply walled in without further development; but in many cases the supply may be increased. A spring is often but one of a number of points of emergence of a given stratum of ground-flow, although the others may be only a general seepage, and may be some distance away. A trench or infiltration-gallery placed at right angles to the direction of flow will then intercept much more than the yield of the spring; or a lowering of the outlet may draw from other channels draining the same catchment area. If there are one or more porous strata near and parallel to the surface of a hillside, horizontal tunnels may be driven into the hill to intercept water from these; as is done at Oakland, Cal.

The deepening and enlargement of a spring and drawing down of the water-level will often cause a considerable increase in the flow at the expense of other springs in the neighborhood. A spring yielding 75,000 gals. per day was so developed in this manner by the author as to yield more than double this amount and furnish the supply for a small community.

Art. 44. Amount of Ground-water Available.

The amount of water flowing in a given stratum may, as already stated, be estimated by the formula $Q = aV$, a being the area of open spaces in a vertical section of the stratum across the line of flow, and V being obtained by test-wells, or by the formula given on page 143. But the total flow in a given stratum cannot continuously exceed the amount entering it by percolation from the catchment area; although it may, temporarily, by drawing upon the ground-storage.

The percolation may vary within very wide limits, but will probably be about 60 to 70 per cent of the annual rainfall in sand, 25% in sandstone, 15% in limestone, and much less in clay, granites, etc. English experimenters have found about 35% to percolate through gravelly loam and chalk. The ground-flow may receive water from not only its own outcrop, but also from that of any porous strata above it. In such a case all such contiguous porous strata may be considered together in figuring the catchment area. A well into the lowest of these will draw from the others as well as from its own flow.

The total percolation into a stratum must generally fill and flow through all parts of such stratum which are below the hydraulic gradient. It is therefore possible that the volume per square foot of section flowing at any point may be either less or greater than the average percolation per square foot of vertical section at the outcrop; and this latter section may be but a small percentage of the actual exposed area, as at *BC*, Fig. 13.

Waters of the third and fourth classes are generally derived from the rainfall over the entire area of the basin or valley in which they are found, augmented by a considerable ground- and surface-flow from the adjacent hillsides. In dry weather the flow is in some instances reversed, and water enters the ground from the river or lake. This is generally true when the soil is of gravel or coarse clean sand, and when the river carries little silt. In any soils the river channel will probably be impervious if the river at times carries much clay or loam in suspension.

The exceeding slowness of flow through rock results in this flow being almost constant through all seasons and years; the more extensive the stratum the greater being the uniformity. In sand and gravel the flow is more subject to variation from this cause, but here also the more extensive the stratum the less the variation. When the stratum feels the effects of droughts the storage capacity of the soil may be called upon. If in Fig. 16, for instance, the ground-water stand at the upper dotted line during the average season, it might during a dry one be gradually lowered to the lower line, the pumps having withdrawn not only the contemporary seepage, but in addition an amount represented by the fall in water-level. It is evident that the ability of a stratum to tide over a drought is measured by its area, depth, proportion of voids, and percentage of contained water which can be abstracted. The volume of voids will vary probably from 20 to 45 per cent of the total volume, depending little on the size of grain but much on the uniformity of size. The

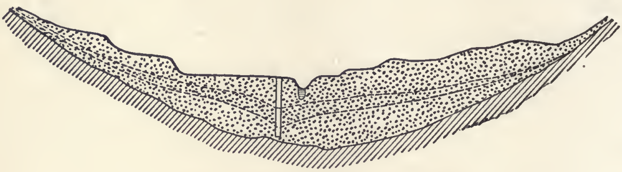

FIG. 16.—GROUND-STORAGE.

amount of the contained water which the soil will yield depends upon the capillarity and hence the fineness of grain. Thus, gravel will give up practically all and clay almost none of the contained water. Ordinary sand will surrender 60 to 70 per cent, and fairly permeable soils 50 to 60 per cent, of their water. An average sandy loam will therefore yield about 20% of its total volume. A stratum of 10 square miles area would thus yield about 418,000,000 gals. per vertical foot of saturated soil, i.e., soil below the ordinary ground-water level. If the daily supply for six months was 10,000,000 gals., while 12,000,000 was required, the additional might be obtained by lowering the ground-water an average of ten inches. The area, however, must be that of the ground-water surface and not of the ground; and as the water surface is lowered its area contracts, and hence the water-level falls more quickly the longer the ground-storage is called upon. This ground-storage may be an important consideration with waters of the third and fourth classes.

The amount of ground-water considered in this article is the total flow; but it must be remembered that in almost no case is all of this available.

ART. 45. QUALITY OF GROUND-WATER.

Ground-water being but the intercepted ground-flow of the yield of a drainage area, the remarks in Art. 34 applying to such flow are applicable to ground-water. But the latter is unmixed with surface-water, and if from deep wells ordinarily furnishes a supply free from organic pollution and colorless.

The analysis on page 150 shows an extreme case of the pollution of the upper ground-waters; but there is always great danger of such pollution when the seepage water is from an inhabited area.

With deep wells, particularly when the water-bearing stratum is overlaid with an impervious one, there is little danger of such pollution; although disease-germs have been known to travel for a considerable distance through such strata when very porous, living for a week or more after entering the ground.

Springs are subject to such contamination when from shallow surface strata or from deep ones which are very porous. The best-known illustration of the latter is the case of Lausen, Switzerland, where, in 1872, typhoid germs were found to pass through a hill and transfer an epidemic from one side of this to the other. The porous stratum here filtered out flour placed in the water, but was comparatively coarse, since the water passed through the hill in a few hours. A spring which shows little effect from droughts will generally be from a deep and extensive stratum, and free from all organic impurities.

Deep wells, and springs from deep and extensive strata, usually give a water containing little free oxygen and much mineral matter, the oxygen formerly present having united with the latter. The Grenelle well at Paris, 1780 feet deep, contains no oxygen. The Ponce de Leon well contains 319 parts per 100,000 of mineral matter, nine minerals being recognized, 195.8 parts being sodium chloride. "Old Faithful" geyser, Yellowstone Park, contains 139 parts of mineral matter, 63.9 being sodium chloride. These waters are not potable. The following analyses of Brooklyn City water from deep and shallow wells show an average amount of mineral matter for such waters.

Table No. 43.

ANALYSES OF BROOKLYN, N. Y., DRIVEN-WELL WATER.

	Number of Analyses.	Residue on Evaporation.			Ammonia.	
		Total Solids.	Loss on Ignition.	Mineral Matter.	Free.	Albumin.
Spring Creek; old driven-well plant	7	19.44	4.04	15.40	.0005	.0015
Jameco Station; driven wells (shallow)	2	17.45	4.45	13.00	.0483	.0140
Jameco Station; driven wells (deep)	1	12.50	2.00	10.50	.0784	.0156
Hempstead stream	20	5.58	1.77	3.81	.2122	.0209

	Number of Analyses.	Chlorine as		Nitrogen as.		Hardness, Equivalent to Carbonate of Lime.	
		Chlorides.	Equivalent to Sodium Chloride.	Nitrates.	Nitrites.	Before Boiling.	After Boiling.
Spring Creek; old driven-well plant	7	1.2857	2.1186	.4516	None	11.0214	8.0929
Jameco Station; driven wells (shallow)	2	3.2500	5.3560	.0426	"	5.0750	5.0750
Jameco Station; driven wells (deep)	1	0.4500	0.7420	.0716	"	9.2000	8.7000
Hempstead stream	20	0.8000	1.3183	.1875	.0006	1.7375	1.7025

The high ammonia and chlorine in the old Jameco Station well would seem to imply pollution by sewage; although the chlorine may be due to the fact that the ocean is but a short distance away. The mineral matter in the wells is seen to be high as compared with the Hempstead Stream surface-water.

Ground water is generally cooler than surface-waters in summer, and warmer in winter; that of wells ranging in Massachusetts between 49° and 53°, and of filter-galleries and shallow wells between 48° and 67°. Some ground-waters are very warm, however; as the Ponce de Leon well, which has a temperature of 86°; and the "boiling springs" found in the Yellowstone Park and several other parts of the world.

QUERIES.

13. Find an equation which will approximately represent the curve of ground-water depression at the Brooklyn well, data for which are given in Art. 40, page 146.

14. If a crib 100 feet long intercept all the ground-flow behind and over it of a stratum 10 feet thick, the slope of the ground-water being 20 feet per mile, $d = 0.5$, $c = .29$, and the area of interstices in a cross-section being .4, what is the maximum amount of water such crib could intercept? What area of catchment-basin at Denver is necessary to supply this amount, assuming 40% of the rainfall to pass off as ground-flow?

CHAPTER IX.

GRAVITY SYSTEMS.

ART. 46. DEFINITIONS.

A SUPPLY of water of the requisite quality and quantity having been decided upon, and the amount of storage necessary, if any, having been calculated, there remains the problem of conducting this to the consumer and distributing it in such quantities as shall be necessary, and under the necessary head.

The methods of bringing the water from its source to the point of utilization may be generally divided into two classes —Gravity and Pumping Systems. In the former the elevation of the source above the point of utilization is so great that, if proper conduits be provided, the water will flow by gravity from the former to the latter; supplying also the pressure-head necessary in the case of city supplies. In pumping systems the source has not sufficient elevation to provide this flow and pressure, and the water must be raised or given sufficient pressure by some form of pump. The source may be higher than any point of utilization, and pumping still be necessary to overcome friction or to raise the water over an intervening ridge or other elevation.

ART. 47. HEAD-WORKS OF GRAVITY SYSTEMS.

A gravity system can be divided into three parts: the distribution system, or the various main and lateral pipes or channels through which the water is distributed; the main conduit, which carries the water from the source to the distribution system; and the head-works, by which the water is intercepted and introduced into the main conduit. As a part of the head-works may be considered all which is necessary to provide the water at the proper rate and head, such as impounding- or distributing-reservoirs, and of the proper quality, such as filters or sedimentation-basins.

The essential part of the head-works of any gravity supply are: a dam (in a very few cases unnecessary), and an inlet to the conduit, with valves for regulating the flow into the same. In the great majority of gravity supplies some storage is necessary, for which an impounding- or storage-reservoir must be supplied. If this reservoir is at a considerable distance from or above the point of utilization, a distributing-reservoir is frequently interposed in the conduit near such point, to relieve the distribution system of excessive pressure, lessen the liability of interruption of service, and permit the discharge of large amounts of water during short periods. The effecting of the first-named result by a lower distributing-reservoir is apparent. The second result is usually obtained because, should there be a break in, or other interruption in the service of, the conduit between the impounding- and distributing-reservoirs, the latter would continue the supply for some part at least of the time required to repair the conduit. A short conduit under pressure from a service-reservoir will deliver water at an unusually high rate with less loss of head than will a long one, since the total loss of head varies with the length; and, moreover, a size of conduit adapted to a given (temporary) high rate of discharge may be carried from the distributing-reservoir only, which can be fed by the continuous flow through a much smaller one from the intake, thus saving in the cost of the latter line. If, however, the utilization is continuous and constant in volume, the last reason for the use of a distributing-reservoir is not applicable.

For example, in fig. 17 a city, *A*, is to be supplied with water from a storage-reservoir, *C*, there being a hill, *B*, but one fifth the distance from *A* that *C* is, which hill is 200 feet above *A* and 25 feet below *C*. A supply for fire purposes or other heavy draught under a 100-foot pressure, to pass which would require a 20-inch pipe from *B* to *A*, would require a 27-inch pipe from *C* to *A*; while an average supply of one fourth this amount, which would keep *B* full

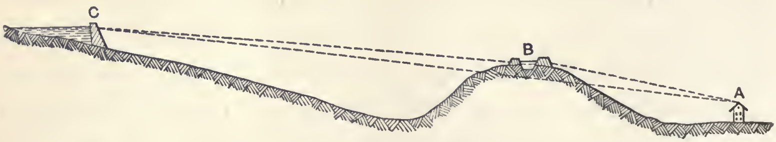

FIG. 17.—DISTRIBUTING-RESERVOIR.

from day to day, would require between *B* and *C* but a 16-inch pipe.

In addition to the above reasons, there is convenience and safety in having complete control of the supply to the conduit at a point near the city.

When it is not necessary to store water in order to obtain a continuous supply, a storage-reservoir is not required; but even in this case it is generally necessary to place a dam across the stream (which is, in most gravity supplies, a small one) to furnish sufficient depth and decreased velocity for the proper intaking of the water, and to facilitate excluding sand and gravel from the conduit.

ART. 48. STORAGE-RESERVOIRS: LOCATION.

A storage-reservoir for a gravity supply is generally placed on the course of the stream or streams furnishing this supply; and in most cases is formed by placing a dam across a valley. If the valley above this point be long and narrow, the reservoir will be of this shape; and in many cases two or more valleys of contributing streams are united in one reservoir; but if a natural basin can be found at a convenient elevation and location this is to be preferred. It is desirable that the enclosing hills be steep, and that the valley be narrow where the dam is to be located, to avoid shallow water and expensive construction. A basin or valley with little slope longitudinally will provide a given amount of storage with less height of dam than one with a steep channel, and is for this reason preferable. The geological formation should be such that there may be no loss of water by leakage under or around the dam, or into another watershed.

The larger the drainage area above the reservoir the greater the quantity of yield, and for this reason the distance of the dam from the head of this area is important. But the reservoir must be sufficiently high up the valley to enable the water to flow to the point of utilization, or even to furnish the desired head of water at this point. The distance from this point, also, and hence the cost of the conduit, it is desirable to make a minimum.

All of these conditions may not exist at any one point, but each should be given due weight in choosing the location of the reservoir. It may be necessary to construct two or more reservoirs to obtain the desired supply, or to carry the water long distances. The first consideration should be the quality of the supply, the next the quantity. A sufficient head to avoid pumping should be aimed at; and this it may sometimes be desirable to obtain by constructing two or more reservoirs rather than by pumping from one at a lower elevation. The location of the dam with reference to its stability is a matter of great importance; and the distance away of materials for constructing the dam and convenience of transportation are important financial considerations. Sound bedrock at or near the surface is desirable; or a thick bed of hardpan or clay, if an earth embankment is to be used.

In deciding the exact location of the dam, cost will generally be the controlling consideration. The elevation of the crest having been decided upon, that location is then best which requires the least expense for excavation and construction; and this is generally when the least quantity of material is required for construction. To decide this point an accurate contour-map should be made of the surface of the ground and of the rock or hardpan to which the dam is to extend, and the approximate quantities required for several trial locations calculated; unless one location appears by inspection to be undoubtedly the best. In many cases a straight dam at the narrowest point is the best location; but conditions of topography are frequently met with which make more economcial a dam whose centre line is curved up-stream, or contains an angle. A detour may sometimes be desirable, also, to avoid a fault in the rock bottom; and a curve adds to the stability of a masonry-dam.

ART. 49. STORAGE-RESERVOIRS: GENERAL CONSTRUCTION.

The amount of storage required, and consequently the capacity of the storage-reservoir, has been treated of in Art. 33. The relation between area and depth, and the general shape of the reservoir, must ordinarily depend upon the topography of the country; but the more regular the shore-line the better, since small depressions in this are apt to cause stagnation of the water, and since in general shallow water and consequent danger from organic growths increase with the length of shore.

Shallow water not only encourages the growth of algæ and other vegetable organisms by admitting light and heat to the bottom and more polluted layers of water, but it also, in summer, causes the average temperature of the water to be higher. On the other hand the deeper layers are apt to become stagnant in summer (see Art. 34) below a depth of 10 to 20 feet. The depth of non-stagnation can be increased by increasing the exposure to winds, as by clearing the shores of timber for some distance back from the reservoir; but this would also increase the evaporation, which is largely affected by wind, and the amount of sediment washed into the reservoir by storms, and is not to be recommended. Decrease in depth also means increased surface area and consequent loss by evaporation. A deep reservoir is hence advisable for all reasons except the formation of a stagnant layer, which may pollute the whole reservoir when the water "turns over" in October or November. If the reservoir contains little organic matter or unoxidized nitrogen, this lower layer is not likely to become polluted, however; and this condition of water should be obtained when possible.

It is difficult to prevent the growth of large amounts of plants and other vegetable organisms in water which is less than four or five feet deep; and it is therefore advisable that as little as possible of the reservoir water have less than this depth for any length of time. This requires that the shores should all be steep down to a depth of five or more feet below the ordinary water-surface, and that all elevations in the bottom which would cause shallows be removed. An ideal reservoir would be approximately oval in shape, with vertical retaining-walls along the shores reaching a depth of ten feet or more, the bottom rapidly reaching the maximum depth of twenty or more feet, at which depth it would have a uniform flat surface. Paved slopes may be substituted for vertical walls to save expense; and the oval form can be only distantly approximated in practice.

The prevention of pollution of water by surface impurities has already been alluded to. But it is also necessary to prevent pollution of the water while in the reservoir. This pollution may come from the reservoir itself or reach the reservoir from the outside. The first is generally due to the improper cleaning of the reservoir before filling. Any organic matter in the bottom of the reservoir is slowly decomposed and the resulting nitrogen often supports vast quantities of algæ. Many reservoir sites have been cleared merely by cutting down the trees and bushes, leaving stumps, roots, grass, and other vegetable matter; but the majority if not all of such reservoirs for years afterward give trouble by the pollution of the water due to the decomposition of this matter. The "fishy" taste so often found in impounded waters is generally due to this false economy in but partially clearing the reservoir site. *All* organic matter should be removed from a reservoir bottom. Investigations made by Prof. Thos. M. Drown in 1893 for the Massachusetts State Board of Health seemed to show that in ordinary uncleared land the proportion of organic matter in the soil was greatest near the surface, and below 9 to 11 inches decreased rapidly, being seldom more than $1\frac{1}{2}$ to 2 per cent at a depth of one foot, although amounting to 15% in some surface soil. Swamp land or muck showed much larger percentages of organic matter. The conclusion reached was that all soil containing more than $1\frac{1}{2}$ to 2 per cent of organic matter should be removed, which generally involves taking off the top 12 inches. All stumps should of course be removed. Pockets of muck should be cleared out; but if these are very deep only the top 8 or 10 feet need be removed, and the holes should then be refilled with clean sand or gravel.

All buildings should of course be removed from a reservoir site, and all organic wastes deposited there by former residents; the privies in particular being cleaned out and the soil for some distance around them being removed, the excavation being then disinfected and refilled with clean gravel and sand, or earth free from organic matter. Care should also be taken that the soil is not polluted by the workmen upon the reservoir; to prevent which, closets should be provided below the dam site and the workmen compelled to use them. The Sodom Reservoir included within its boundaries twenty-one dwellings and barns, three mills and two factories, besides six miles of roads; and the Vyrnwy Reservoir (Liverpool waterworks) embraced the village of Llanwddyn, consisting of about forty dwellings, with barns, etc., and a cemetery.

It would be quite desirable to provide a concrete or similar artificial bottom for a storage-reservoir, but the cost involved renders this impracticable.

Pollution from outside the reservoir may be from human beings defecating upon the banks or swimming in the waters; from organic matter deposited therein through malice or ignorance; and from leaves and organic dust blown into the water. (It is of course assumed that no stables, piggeries, or out-houses will be permitted around the reservoir.) No picnics, bathing, or loitering around the banks of the reservoir should be permitted; to insure which a watchman should be constantly on hand. (It may be desirable, however, to permit driving around the reservoir.) To better permit watching the reservoir banks, and also to prevent leaves from falling into the water, it is well to clear all trees and other vegetation from a space 25 to 100 feet wide all around the reservoir banks.

The water entering the reservoir should bring as little matter in suspension as possible. For this reason, if the stream have considerable volume and velocity it may be desirable to provide a settling-basin at its entrance in the reservoir; generally by constructing a submerged weir across the reservoir from bank to bank near the mouth of the stream.

The conduit must receive water from the reservoir in such a way that there is no loss by leakage, that the mouth of the conduit can be tightly closed if desired, that no gravel, sand, leaves, fish, ice, or other matters can enter it, and that the water can be drawn from different elevations above the reservoir bottom at pleasure.

It must also be possible to draw off and waste the water from the bottom of the reservoir when this is to be cleaned or repaired; or, as is often desirable, to remove the bottom layers of stagnant water just before the "turn-over."

ART. 50. SPILLWAYS.

If the reservoir should be approximately full at the time of a rain-storm, the run-off from this would need to overflow and be wasted. Provision for this is one of the most important details of reservoir designing, and insufficient allowance for it has caused more damage and loss of life than all other reservoir details combined. This waste water can of course be allowed to flow over the whole length of the dam creating the reservoir, but this is not permissible in the case of an earthen dam, and requires a most substantial construction along the whole foundation and front of a masonry or timber one. For these reasons the waste-water is usually provided for by a spillway, waste-way, or waste-weir.

A waste-weir in the centre or side of a dam is frequently used, being practically but a part of the dam whose top is lower than that of the remainder and whose construction is more substantial. The waste-water flows from this to the bed of the original stream. A spillway is frequently provided in the bed-rock at one end of the dam, the rock being so cut down as to permit the water to overflow at the desired level. Where applicable, this method is generally preferable to a weir. A side spillway is practically a continuation of the dam along one side of the reservoir by a low wall whose top serves as the weir, the waste-water flowing along a channel between this and the ground outside the reservoir. This construction is practicable only when rock is found near the spillway level along one side of the reservoir, to serve as foundation for the wall and bed for the waste-water channel.

In some instances the spillway is entirely separate and at some distance from the dam, being placed in a depression or "saddle" in the surrounding hills, to which the water is raised by the dam. A low masonry waste-weir will then suffice, and all danger from wash at the toe of the dam be avoided, the water being discharged into another valley. This plan cannot often be adopted, but where practicable is an admirable one.

The waste-weir or spillway should be constructed in the most substantial manner to withstand the shock of the overflow from the greatest floods. Its top must be designed to receive the blows from, and to pass over its crest, ice, logs, or any other matter brought down by the flood. It must, without any possibility of failure or of choking up, so provide for the passing of all water, ice, and floating matter that the water in the reservoir can never under any condition reach the top of the dam.

The length of a spillway and the depth of water to flow over it demand careful consideration. The elevation of the top of the spillway, and not that of the dam, decides the elevation of the water-surface in the reservoir and hence the amount of storage provided. This elevation then is the starting-point. The water should never reach such a level that its waves can rise above the top of the dam, which must therefore be higher than the spillway by an amount equal to the greatest depth of water on the spillway plus the greatest height of waves possible. If the dam be long, this additional height will add considerably to the expense; and to keep it at a minimum, the depth of water on the spillway must be decreased by increasing its length. But this may mean an increased cost due to the spillway, which is often much more expensive per lineal foot than the rest of the dam. The least expensive construction can ordinarily be ascertained only by comparing two or more plans. The depth of flow over the spillway during heavy floods should not be so shallow as to permit of ice, logs, etc., stranding there and forming an obstruction. For the same reason piers, posts, or other obstructions which would be likely to catch ice, brush, or other floating matter should not be placed in the spillway.

The depth to be allowed for waves will vary with the length of the reservoir and consequent sweep of wind possible. Stevenson gives the formula

$$H = 1.5\sqrt{L} + (2.5 - \sqrt[4]{L}),$$

in which H is the maximum height of wave, in feet, and L is the length of the reservoir, in miles. If the reservoir be $\frac{1}{2}$ mile long, this formula would give $2\frac{3}{4}$ feet as the maximum height of waves; and if 2 miles long, $3\frac{1}{3}$ feet. Two feet is the least which should ever be assumed for wave height.

In calculating the capacity of a spillway—that is, the maximum rate of run-off from any storm—the method outlined in Art. 32 is recommended, rather than any of the formulas. The Melzingah (N. Y.) dam, which failed by overflowing in July 1897, had a spillway which, calculating by the formulas, was sufficient for its drainage area of 1.1 square miles, but which proved its practical insufficiency.

It will generally be desirable to calculate the run-off from maximum rates of precipitation for 10, 20, 40, and 60 minutes on the drainage area, if this be small; and for 1, 2, 6, 12, and 24 hours if it be large; and for the whole area in either case; since the run-off from a part of the area due to a short-period rate of rainfall *may* be greater than that from the whole due to the lower rate of rainfall involved. By using these few calculations to plot a curve representing the run-off due to maximum precipitations for different intervals of time, the maximum rate of run-off may be determined.

ART. 51. DISTRIBUTING-RESERVOIRS.

The reasons for the use of distributing-reservoirs have been given in Art. 47. Where these are not used the storage-reservoir acts as a distributing-reservoir also.

The damming of a valley, when one admitting of this is favorably located, is the least expensive method of forming a distributing-reservoir. In most cases, however, such a reservoir is constructed on the top or side of a hill above and near the point of utilization. In such a situation part or all of the sides of the reservoir are in most cases partly in embankment. Stability and economy are generally best obtained by locating a reservoir on comparatively level ground, thus avoiding high embankments; and in no case should any part of the bottom of a reservoir be above the original ground-surface. The location should generally be as nearly as possible on the direct conduit-line from the storage-reservoir to the point of utilization, and near the latter. Many distibuting-reservoirs have been located within the limits of a city, as is the case in New York and Philadelphia.

The reservoir capacity should be at least equal to the maximum consumption for three or four days. It is desirable to have two reservoirs, or one divided by a partition-wall, that each may separately be emptied and cleaned without interrupting the service. This also permits the water to stand for three or more days and deposit any sediment which it may contain, the other reservoir being used meantime.

In plan the reservoir is frequently a quadrilateral, with rounded corners if earth embankments be used. But this is determined by economical considerations, the greatest capacity being obtained at the least expense.

Since it is so much smaller, a distributing-reservoir can generally be constructed more as theory dictates than can a storage-reservoir. For example, the banks can all be given a steep slope and paved throughout; the reservoir should be of a considerable depth, there being no "turn-over" to avoid; it can be fenced in and all pollution from outside sources avoided, being covered in some cases to insure this and to preserve a low temperature as well as to prevent the growth of algæ.

A distributing-reservoir is provided with a conduit from the storage-reservoir and one to the distribution system, and with a waste-pipe to permit emptying it, as well as gates for controlling these. It should be perfectly tight and stable, more particularly when in the midst of or near an inhabited section.

Art. 52. Gravity Supplies from Large Streams.

The above articles have considered the supply as being from surface-water and small streams only; but in some cases, particularly in irrigation-works, a gravity supply is obtained from a stream of such size that no storage is necessary, a part only of the ordinary and flood discharges being diverted. A canal or flume leading from one bank of the river will intercept a part of the flow; but it is also necessary that provision be made for intercepting a large part or all of it in time of low water, for excluding gravel and the heavier sediment during floods, and for drawing off a constant supply at all times. This ordinarily requires a dam which will retain all the flow when necessary, but pass most of it during flood; and head-gates by which water can be taken into the conduit from the bed of the river during low water, but from near the surface during floods when the bottom flow is full of heavy sediment. There should also be provision for flushing out the deposit which will collect behind the head-gates and dam, for which purpose sluices at the bottom of the same are desirable. It is also necessary to provide that the channel, particularly at low water, shall pass by the end of the conduit, and this may require spur-dams, and a sluice near this point.

It is particularly necessary in works thus situated that the foundations and all portions of every structure be of the greatest strength and solidity.

ART. 53. OPEN CONDUITS.

Conduits between storage- and distributing-reservoirs, and all conduits in irrigation systems, may be open and follow the hydraulic gradient; or may be closed and rise and fall with the surface, being under internal pressure due to their distance below this gradient. Conduits from distributing-reservoirs, or from storage-reservoirs on city supply systems where there are no distributing-reservoirs, must, for the last part of their length at least, be under pressure.

The simplest form of open conduit is a canal excavated in the earth. To avoid loss by seepage this is frequently lined with concrete or other material. Conduits are also constructed of timber, or of sheet iron or steel, supported on the ground or on trestles, or of masonry resting on the natural soil or on solid embankments. Open conduits are carried across valleys and streams by means of aqueducts, and through mountains by tunnels. In some Western works conduits of concrete, stone, wood, and iron, aqueducts, tunnels, and pressure conduits are all found on the same line.

A canal must ordinarily follow quite closely the surface contour of the country traversed, having only such fall as will give the water the desired velocity. This may, in a mountainous country, lead to such detours as to enormously increase the length, cost, and head lost. Where a straight course can be obtained, however, and the ground is fairly level laterally as well as longitudinally, a dug canal is generally the cheapest. If the general longitudinal grade is steeper than that permissible for the canal, an occasional drop can be made in the latter, either as a falls or as a rapids, or the head can be consumed by gratings or contractions in the channel, wooden or masonry construction being used at these points.

The chief objection to canals is the great loss by percolation, which has been found in Utah to amount to 20 inches per day; and on the Erie Canal to from 35 to 100 cubic feet per minute per mile of canal 40 feet wide, or about 3 to 10 inches per day. The following table, compiled by Prof. L. G. Carpenter of the Colorado Agricultural Experiment Station, shows the daily loss by seepage on various canals.

Pleasant Valley and Lake Canal....	c.66 to 5	feet
North Farm Lateral...............	0.80	"
Fort Morgan Canal...............	1.00 to 2.60	"
Hoover Ditch.....................	1.00	"
Greely No. 3, special case........	30.00	"
" " July 20, 1898......	18.00	"
North Poudre Lateral............	0.6 to 1.00	"
Muzza Canal, Italy...............	1.70	"
Naviglio Grande..................	0.80	"
Martesana Canal.................	1.50	"
Centreville and Kingsbury Canals..	6.00	"
Kings River and Fresno Canal.....	0.6 to 1.70	"
Fresno Laterals..................	1.2 to 6.40	"
Kern County Canals..............	0.39 to 2.60	"
" " " sandy soil......	1.00 to 2.00	"
" " " " loam....	0.39 to 1.30	"
Campine Canal, Belgium, sandy....	2.00 to 10.00	"
Erie Canal.......................	0.25 to 0.80	"
Carpentras Canal, France.........	1.20	"
Marseilles " "	0.40	"

A long canal in earth may lose by seepage more water than it delivers. To remedy this, much can be done by admitting water heavily charged with clay in suspension and permitting it to pass slowly through the canal. If the water intercepted does not carry clay or loam, or if the canal must be tight from the beginning, the sides may be puddled if materials for this are at hand. If they are not, or if still greater tightness is desired, a cement lining may be given the canal. This method was adopted in the case of one of the Riverside Irrigation Canals. Or a heavier lining of concrete or stone masonry may be used. (See Plate VIII.)

Where an excavated canal is not constructed because of seepage, or of unfitness of the soil, or because the transverse slopes are so steep as to require a dangerous amount of embankment, but where the contour can be followed, one or both walls of the canal are sometimes formed of masonry, the bottom of the canal being either bed-rock or concrete.

In place of a canal a conduit or flume of wood, iron, or steel is often used, resting upon the levelled ground where possible, but often upon trestles or embankments. These can be made practically tight, thus permitting no loss of water except that due to evaporation. For crossing valleys at the hydraulic gradient trestles or aqueducts are most frequently used. (See Plate IX.) When the flume rests upon a level bench cut into a hillside it is called a bench-flume. (See Plate X.) A flume should never rest upon an embankment, which is sure to settle somewhat; and a bench-flume must be water-tight if resting upon earth, as any erosion of this caused by leakage would be fatal.

The name aqueduct is generally given to a valley crossing of some magnitude and of substantial construction, as of masonry. High Bridge, New York, is an excellent example of this.

The longer water remains in an open conduit the greater the loss by evaporation and seepage; for this reason, and to prevent the growth of weeds in earth channels, considerable velocity is desirable; and is necessary also if the depositing of silt in the canal is to be prevented. But if the velocity become too great the earth is eroded, or the metal, wood, or masonry abraded by the sand or gravel carried. Also, since velocity is obtained at the expense of head, a high velocity may lower too much the level of the canal or the pressure-head at the point of utilization.

The growth of weeds may ordinarily be prevented by a velocity of 2 to 3 feet per second; and this velocity will also prevent the deposit of such matter in suspension as should properly be let into even an irrigation-canal. Light or sandy soil is likely to be eroded by a velocity of 2 feet or more; in firm loam or clay a velocity of 3 feet is permissible; in brickwork, wood, or sheet-metal flumes a velocity of 5 or 6 feet may be allowed; but if the velocity exceed this a substantial construction of hard stone masonry should be provided.

The grades which will give these velocities, in either an open or a closed conduit, depend upon the size, form of channel, and character of the wetted surface, and may be calculated by Kutter's or some other acceptable formula. (See Chapter XI.) For open conduits particularly, Kutter's formula seems to be the most satisfactory.

The area of cross-section of the canal must be greater than the quotient obtained by dividing the maximum amount to be passed per second, in cubic feet, by the velocity of flow in feet per second; the banks being at least 12 to 18 inches higher than the highest water-surface. The irrigating season lasts but about 100 days in most of our Western country, and in that time practically all of the water used must pass through the irrigating canal. This is the basis of the method of expressing duty by "acres per second-foot"; that is, the number of acres which can be irrigated by a continuous flow

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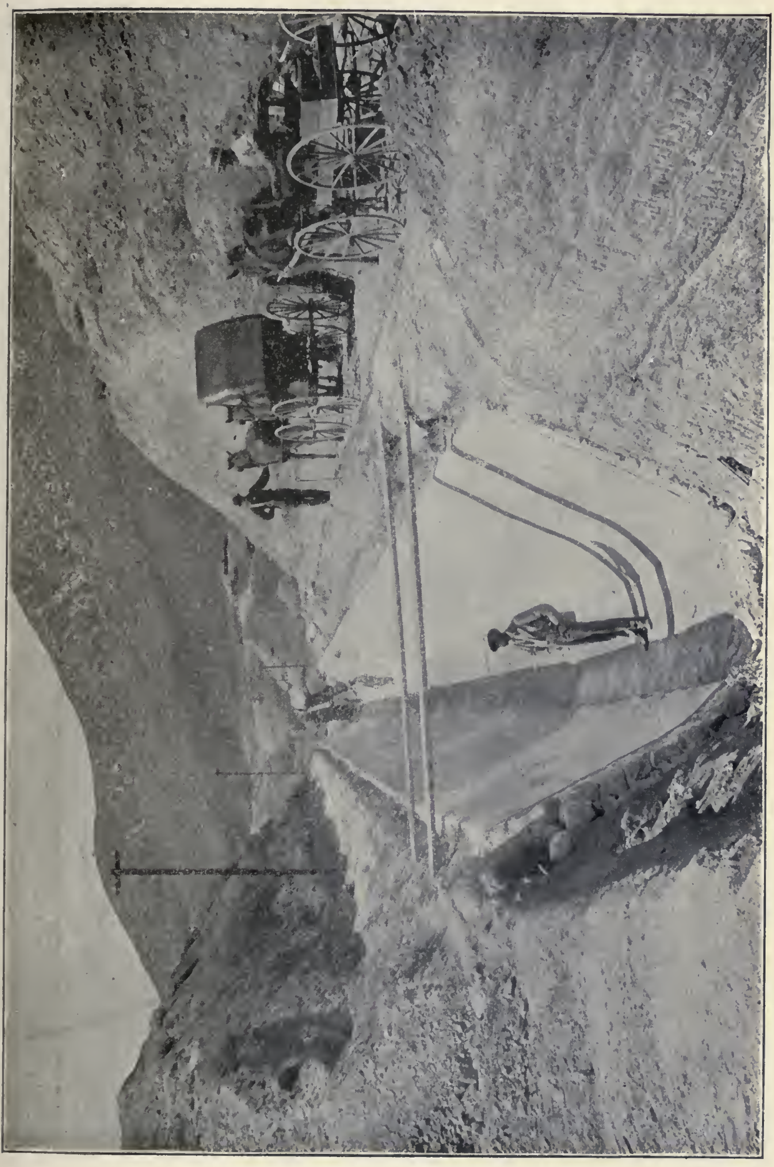

PLATE VIII.—PAVING OF SANTA ANA CANAL, CALIFORNIA.
(From Trans. Am. Soc. C. E., Vol. XXXIII.)

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PLATE IX.—CONDUIT ON TRESTLE.
(From Trans. Am. Soc. C. E., Vol. XXXIII.)

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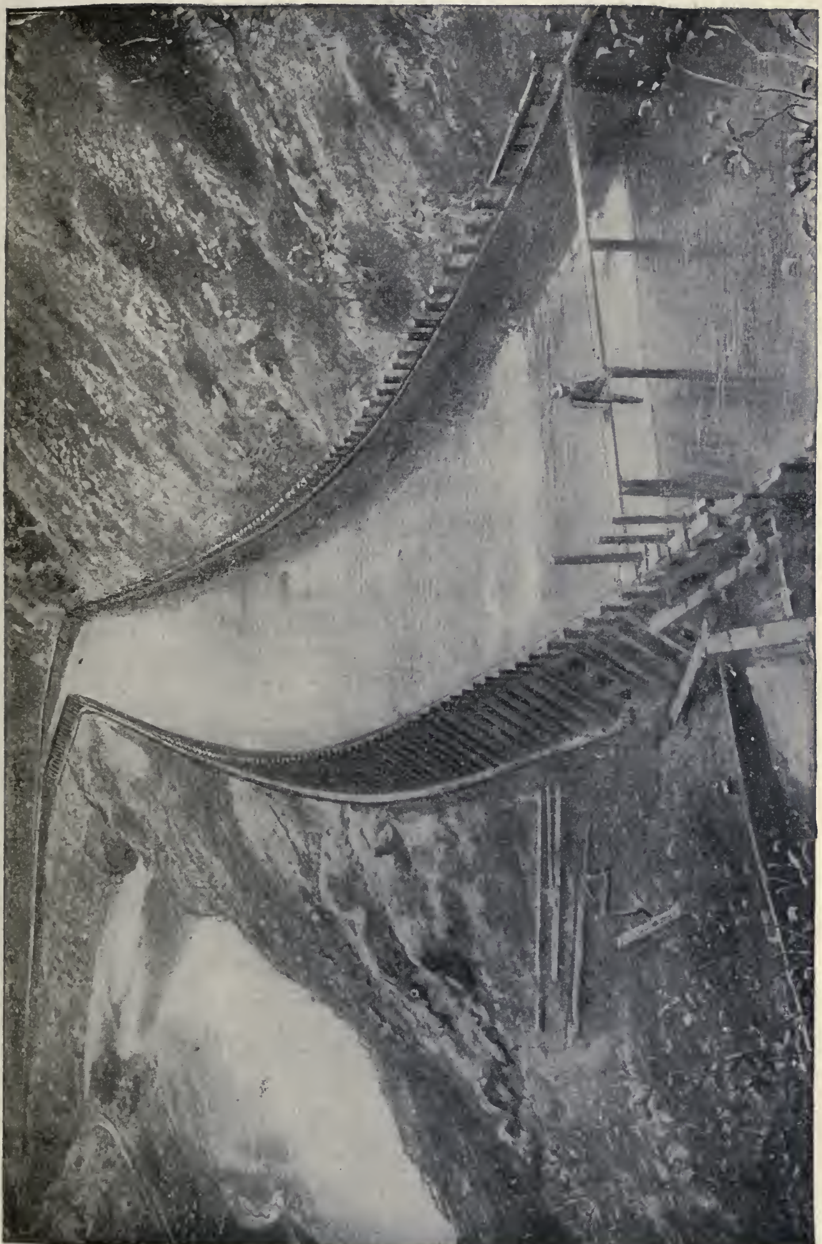

PLATE X.—BENCH-FLUME, COLORADO.

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during the irrigation period of one cubic foot per second. Sixty to one thousand acres is about the range of duty, and one hundred may be taken as a general average where no exact data are available. The higher rates are found where subsurface irrigation is practised; being 250 to 500 in the San Bernardino (Cal.) district.

For example, if 30,000 acres are to be irrigated, duty 100 acres, there must be a flow of 300 cubic feet per second plus the loss by evaporation and seepage; and the area of cross-section of the canal, if the velocity be 3 feet per second, must be $\frac{\text{300 cu. ft.} + \text{per cent of loss}}{\text{3 ft.}}$, or 100 square feet plus the per cent of loss.

Another method of calculating the flow in irrigating-canals is to divide the total mean annual yield, less the evaporation and seepage from the reservoir, by $60 \times 60 \times 24 \times 100$, or 8,640,000, the number of seconds in 100 days; assuming that all of the yield will be used for irrigation, which is naturally the condition aimed at.

In calculating the areas of canals for city supply, recognition must be taken of the fact that the consumption is not uniform throughout the year, but may be 25 to 50 per cent greater than the average for one or even several days at a time, particularly in the summer, when the evaporation and seepage also are greater. The amount designed to be carried by the canal will be based upon these conditions, being at least 50% greater than the average daily consumption, aside from the evaporation loss.

In case more water were admitted to an open conduit than was being taken at the lower end, the banks or sides of the conduit would be overflowed, which would in most cases be disastrous. To prevent this the surplus water is provided for by specially constructed overflows, called waste-weirs or waste-ways, placed at intervals along the line.

In some cases no artificial conduit is constructed below the storage-reservoir, which is used as a regulator of flow only, but the discharge is conducted in the original channel of the stream to a distributing-reservoir or intake.

The lengths, dimensions, and carrying capacities of several American canals are given below.

TABLE No. 44.

SOME GREAT IRRIGATION CANALS.

(From Wilson's Manual of Irrigation Engineering.)

Name of Canal.	Locality.	Area Commanded, Acres.	Length, Miles.	Capacity. Feet per Second.	Grade.	Bed-width, Feet.	Depth Feet.	Cost per Acre Irrigated.	Cost per Second-foot of Water Used.
Bear River Canal..	Utah	200,000	150	1000	1 in 5280	50	7	$ 5.00	$ 125
Idaho Mining and Irrig. Co. Canal..	Idaho	350,000	70	2585	1 in 2640	40	10	2.16	190
Pecos Canal........	N. Mexico	200,000	75	1100	1 in 6707	45	6	5.00	690
Turlock "	California	176,000	93	1500	1 in 5280	70	7.5	14.50	730
King's River and San Joaquin Canal	"	90,000	67	600	1 in 5280	32	4.5	7.18	277
Calloway Canal....	"	80,000	32	700	1 in 6600	80	3.5	10.00	710
Arizona "	Arizona	60,000	41	1000	1 in 2640	36	7.52	10.00	700
Highline "	Colorado	90,000	70	1184	1 in 3000	40	7	13.00	600
Del Norte "	"	200,000	50	2400	1 in 2112	65	5.5	....	....

ART. 54. CLOSED CONDUITS.

Open conduits are seldom used on city water-supply systems, because of the danger of pollution, loss by evaporation, and variation of temperature. Closed conduits are hence provided, being generally made of water-tight construction. Should such a conduit fall below the hydraulic gradient it would receive internal pressure and must be of a construction to resist this. Masonry conduits can do so to but a very limited extent, and are hence adapted to those locations only which follow exactly along the hydraulic gradient. Such location generally requires considerable cutting, tunnelling, embankments, trestles or masonry bridges, and other expensive work; and for this reason metal or wooden conduits able

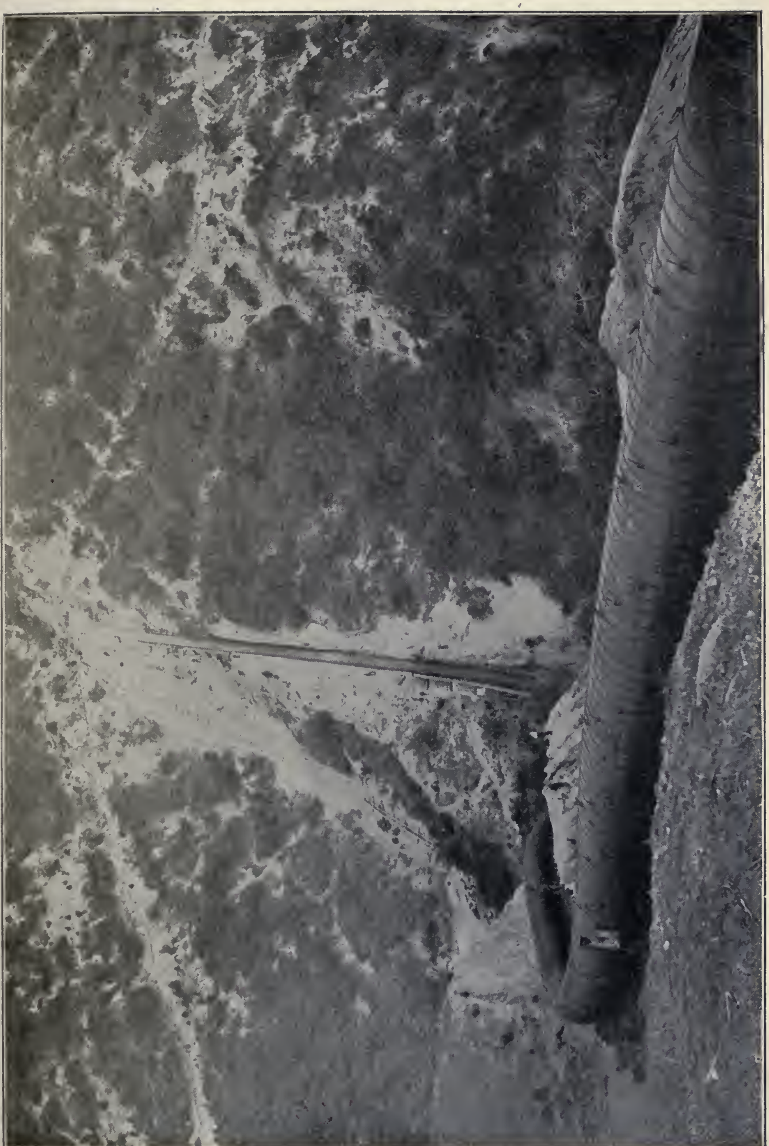

PLATE XI.—WOODEN-STAVE PIPE.
(From Trans. Am. Soc. C. E., Vol. XXXIII.)

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to resist internal pressure are ordinarily employed where possible. Where the amount of water is very considerable a trestle or even a masonry aqueduct may, however, be cheaper than a pressure conduit which rests upon or in the surface at all points. A masonry conduit is in some cases made to resist internal pressure by constructing it as the lining of a tunnel in rock, the pressure being received and resisted by the rock; an illustration of which is the new Croton Aqueduct.

On many irrigation-works open aqueducts are used for surface conduits; but for crossing valleys a pressure conduit following the surface is substituted. Such a conduit, of wood-stave pipe, is shown in Plate XI. The same construction is of course adapted to city water-supply systems. The majority of pressure conduits for these are now constructed of iron or steel plates for the larger, and cast iron for the smaller, sizes. Bored logs, indurated wood-pulp, and cement-lined sheet-iron pipes are used for small conduits in some plants. The first pipes used in this country were of bored logs, and spruce-log pipes eighty-five years old have been found in good condition.

When closed conduits are not under pressure their size and grade are calculated as in the case of open ones; and overflows or waste-weirs are similarly provided. When under pressure they must always flow full, and the formulas for flow in pipes are applied. No waste-weirs are then necessary except at the head-works; but flushing-out gates should be provided at the low points for removing sand and other deposits.

The pressure to be resisted by the tensile strength of the walls of the conduit is the hydraulic pressure due to the difference in elevation of the conduit and of the water in the open conduit or reservoir at its head. This pressure is not attained when the water is flowing, but only when a gate at the lower end of the pressure conduit is closed. When flowing, the pressure head equals the vertical distance between the conduit and the hydraulic gradient (see Art. 63), and this is the maximum pressure to be provided for if there be no gate at the lower end of the pressure conduit, or if an overflow be provided there at the level of the hydraulic gradient.

Both wood-stave and riveted-steel pipes have been constructed 72 inches in diameter, and may be made yet larger. Rock tunnels can be made of any size. That on the Croton Aqueduct has an area of about 118 square feet, or a diameter of 12 feet 3 inches.

TABLE NO. 45.

SOME CLOSED CONDUITS.

Location.	Material.	Dimensions, Inches.	Length, Miles.	Velocity of Flow, Feet per Second.	
Nashua, Boston.........	Masonry	126 × 138	7.0	6±	Gravity
Sudbury, "	"	108 × 92	15.9	3	"
Croton, New York ; Old	"	89 × 101.5	......	2.218	"
Croton, New York ; New	"	147 diam.	29.63	3.0	Pressure
Baltimore, Md..........	"	144 "	7.0	......	"
Denver, Colo...........	Wood	30 "	18.00	......	"
Caldwell, Idaho........	"	54 "	0.13	......	"
Bear Valley, Cal........	"	52 "	0.41	......	"
Ogden, Utah............	"	72 "	5.10	8.75	"
" "	Steel plate	72 "	0.87	8.75	"
Newark, N. J...........	"	48 "	21.00	5.0	"
Rochester, N. Y.......	"	60 "	0.30	......	"
	Cast iron	30 "	20.	......	
Coolgardie, Australia...	Steel plate	30 "	328.0	1.9	"

ART. 55. LOCATION OF CONDUITS.

The location of conduits when the loss of a few feet of head is not important is largely a matter of economy. When every available foot of head must be preserved, however, the alignment must be approximately straight and the grade as light as will give the desired velocity. To avoid excessive evaporation at low velocities, and to permit following the irregularities of surface in an air-line location, closed conduits, in many or most places under pressure, are used. There may be locations where the loss of head in passing around a basin or pocket will be less than in going directly across it with a deep loop or inverted siphon. A siphon proper is to be avoided where possible, although a number of these are in use.

If the question is one of cost only, a more circuitous line is frequently preferable. Although longer, it may avoid deep cuts and tunnels, may be open most of its length, may have considerable fall and hence smaller cross-section, and deep inverted siphons, calling for strong, high-pressure conduits, may be avoided. In many instances, however, deep cuts, tunnels, or inverted siphons may be the less expensive. Estimates of cost for different routes must generally be made for each section to determine the most desirable location.

Sharp curves should be avoided, since they cause loss of head and erosion of the outer banks of canals. In general the minimum radius of curvature of the inner side of a conduit should be about twice the product of the depth of water and the velocity of flow in feet per second.

Streams should always be crossed at a safe distance beneath their beds or above their highest flood-lines.

Swamps and other soils affording poor foundations should be avoided if possible; if not, artificial foundations must be carried down to rock or firm soil.

ART. 56. DISTRIBUTION SYSTEMS: IRRIGATION.

Water which has been impounded and brought to the borders of an irrigation district must still be distributed to a large number of farms throughout this district, and to all

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parts of each farm. This is generally accomplished by canals or flumes, but in some cases by pipes. As the limits of the district are approached the water to be carried becomes less and the canals or pipes smaller; the last length carrying but enough water to irrigate one farm in the time allowed for its use. From the canals water is usually diverted into smaller ditches, and in these led to all parts of each farm. From pipes the water is led through smaller pipes to hydrants, which discharge the water upon the ground directly or through a hose. For subsurface irrigation underground open-jointed pipes are used, fed from either canal or pipe distributary.

Irrigation is not continuous on any one farm, but water is applied from two to ten times each season, for periods of from 3 to 15 hours each; such application being called a service. The times of service for each farm are fixed beforehand by the district manager. Thus each farm draws water from the canal for but 10 to 40 hours in a season of 100 to 120 days, or $\frac{1}{240}$ to $\frac{1}{72}$ of the time. Hence a distributor which will carry but sufficient water to irrigate one farm during the time of its service periods may still serve 72 to 240 of such farms, which rotate in their services.

Distributaries should, where possible, follow natural ridges or elevated ground, that the water may flow by gravity to all parts of the territory on either side. Each will then serve the land to the bottom of its slope on either side, and should be designed of corresponding capacity. Small distributaries, where they can follow the surface contours, are generally ordinary ditches in excavation and embankment, the bottom being kept at or a little below the original surface; where they must rise above the surface for a short distance wooden box-flumes are used. The larger distributaries are sometimes lined with cement or concrete, the smaller with split clay pipes, where water is scarce and valuable. If the natural fall of the surface gives too great velocity, the distributaries are built on a flatter grade, and wooden chutes and falls are introduced at intervals.

As far as possible distributaries should be located at uniform distances apart. They should be as few and large as possible, since this tends to reduce the losses by evaporation and seepage, and the expense of maintenance.

Water is usually diverted from the main to the lateral distributaries by stop-gates or other checks in the main, and the amount admitted is regulated by a suitable gate or orifice in the mouth of the lateral. The amount discharged from a distributary to an irrigating farm is usually measured by the size of and head above an orifice through which it passes, and the duration of flow. The quantity is ordinarily expressed in "miner's inches" in the West, but second-feet is a unit more desirable for many reasons. The miner's inch is a variable quantity, being the same in no two States. In California it is about equal to $\frac{1}{50}$ of a second-foot, in Colorado to $\frac{1}{38}$; while in some States it is not a constant, but varies slightly with the conditions of flow or the amount measured.

ART. 57. DISTRIBUTION SYSTEMS: CITY SUPPLIES.

The distributing system of a city supply is always composed of pressure-pipes. The pressure varies, in this country, between 10 and 200 pounds, with perhaps exceptional cases outside of these limits. The static pressure in any part of a system fed by reservoir is that due to the difference in elevation of such point and the level of water in the distributing-reservoir. The pressure, in pounds per square inch, equals 0.434 times this difference of elevation, in feet. Since there will or may be times when no water is being consumed, all portions of a distribution system must be designed to withstand the static pressure.

The two chief objects of a city water-supply system are: to furnish water for domestic, manufacturing, and similar purposes; and to afford fire-protection. Use is also made of it in sprinkling streets and lawns, flushing sewers, etc. For these purposes the distribution system should reach every building, and permit of placing fire-hydrants at distances apart of not more than 500 feet in all occupied streets. The filling of sprinkling-carts, supplying of fountains, and flushing of sewers must also be provided for.

The pipes of a system must not only reach all these points, but they should be of such size that the required rates of flow may be obtained at any point. It is desirable that the pressure be such at every fire-hydrant that a stream through 400 or 500 feet of fire-hose may be thrown to the top of the highest building; but this is not always possible, especially where the buildings are excessively high.

An additional requirement is that the pipes and attached appurtenances be amply capable of withstanding the greatest pressure to be brought upon them.

It is desirable that all parts of the system be durable, requiring to be renewed only at long intervals of time. Also that it be so arranged that repairs, alterations, or renewals may be made, or breaks occur, at any point without interfering with the service at any other point. All parts of the system should be easily located and accessible.

It is especially desirable that the quality of the water be not impaired by any part or condition of the system; and that there be no waste or leakage at unknown or inaccessible points.

The reaching of all buildings usually requires that a pipe pass along one side of every lot. This does not always necessitate passing through each street, since in sections of many cities all buildings face upon one set of parallel streets, while those crossing these pass but the sides of corner lots;

in which case pipes upon the main streets only will pass all houses. In such sections of a city the lengths of cross-streets included between the main streets seldom exceed 300 to 400 feet, and if fire-hydrants be placed at the corners no intermediate ones on the cross-streets, and hence no pipes will be necessary. The filling of sprinkling-carts and providing for sewer-flushing and fountains will seldom require any pipes not demanded by the above considerations.

The arranging of the system to provide for restricting the interruption of service consequent upon repairs and breaks demands a means of cutting off any small section of it from the rest of the system. The smaller the section cut off the less the inconvenience caused. It is desirable that such cutting off may be effected quickly in case of a break or other accident. This is generally attained by inserting stop-gates or valves in the lines of the pipes. These can be placed at any desired distance apart, but one on each line at each corner is probably as close as it is generally desirable to place them. When so located not more than four gates need to be closed to cut out any short section less than one block long. Thus, in Fig. 18, a break at *S* would require the closing of gates at *A*, *B*, *C*, and *H* to exclude all water from that point while making repairs. The point *T* can be cut out by closing *I* and *J*; and *U*, by closing *C*, *D*, and *E*. In neither of these cases will any section except that between the gates named be deprived of water. If gates be placed on only every other corner on each line, twice the length of pipe will be put out of service by breaks or repairs.

Of the pipes and conduits which are usually placed in a street, all cannot occupy the centre. For several reasons it is generally desirable that the sewers occupy this position; and the water-pipe must therefore be on one side of the centre. Which side is not a matter of very great importance; but the north side offers the advantage of being warmer, and hence giving less opportunity for the freezing of the pipes. The same side of all streets throughout the city should be

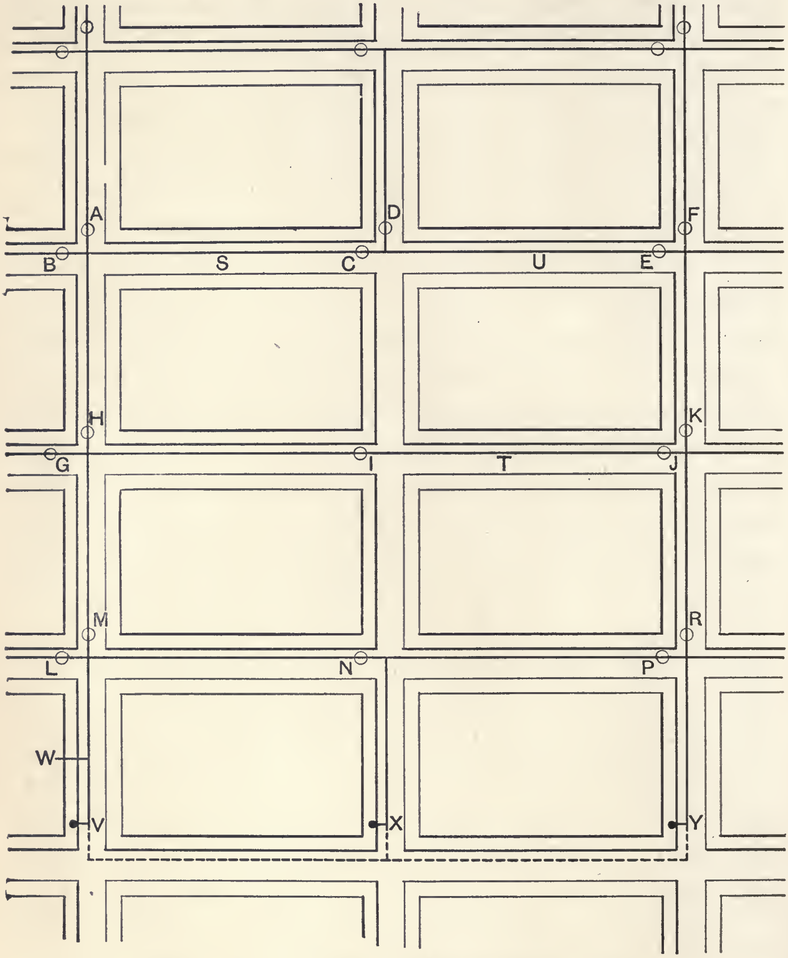


FIG. 18.—LOCATION OF MAINS AND VALVES.

used for the water-pipe, however—as the north and west sides—to facilitate ready location.

In order that the pipe may be quickly and readily located, it should be placed at a uniform distance from some fixed line of the street in all parts of the system. This line may be the centre of the street, the curbing, or the property-line. The use of the centre-line requires that this be first located, thus more than doubling the time and labor required. When streets and sidewalks are of different widths, the use of a constant distance from either curb or property-line is impossible or inadvisable. The author has adopted, as a satisfactory method, a distance from the property-line of such multiple of five feet as will bring the pipe between five and ten feet from the curb. The property-line, usually indicated by a fence or house-front, is easily found, and there is little danger of using the wrong multiple of five for the distance. For instance, in a 60-foot street with 12-foot sidewalks the pipe would be located 20 feet from the property-line.

The hasty location of a gate is often more important than that of the pipe—as in case of a break, when the flow in that section must be stopped immediately to prevent further damage. In many systems the memory of the superintendent or the maps in the office must be consulted before a gate can be located, and if either or both of these be temporarily lost great delay and damage may result. This can easily be avoided by placing the gates systematically, as in line with the curbing, in the middle of the cross-walk, or in range with the property-line. The last named offers the most ready method of location in winter, when the curb and cross-walk are hidden under the snow. The gates should be on a uniform side of the corner, also. In Fig. 18, all pipes being upon the north and west sides of the streets, and all gates on the northwest corners and in range with and at a known distance from the property- or fence-lines, their exact location can be found in a few seconds.

Post fire-hydrants—that is, those which stand above the ground two to four feet—can readily be found if their approximate location is known. (In winter, snow should never be allowed to cover them, but they should be kept shovelled out and ready for instant use.) Flush-hydrants, which are flush with the street-paving, should be located uniformly in a manner similar to that used for gates. Any other underground fixtures will be so few in number that the location of each can generally be fixed in the memory as well as recorded in the office.

The prevention of waste and leakage, and the withstanding of pressure, depend upon the selection of suitable material and obtaining of careful workmanship. The pressure to be provided for at any point is treated of in Chapter XI.

The maintenance of the quality of the water requires that all portions of the pipe, gates, and other appurtenances which come into contact with the water shall give to it no taste or odor or objectionable soluble matter. Some coatings used on iron pipes and gates give a tarry taste to the water for a short time, but this is not injurious and soon passes away. The most frequent violation of this principle is in the use of lead or lead-lined pipes, this metal being soluble in some waters. In general, soft waters dissolve lead and temporarily hard waters do not. The action upon a given lead surface diminishes with time, but may be so considerable as to cause lead poisoning in consumers. The only sure method of determining whether a given water attacks lead is to test it by actual contact with a bright lead surface for several days. Zinc also is soluble in some waters, but is not so dangerous to the human system as is lead. For waters which attack these metals, service-pipes of iron lined with tin or cement are recommended.

If the water contain any organic matter, its stagnation in a pipe deprived of light and air is likely to cause offensive decomposition, although this matter might have been of a harmless nature. Stagnation also favors the deposit in the pipes of any mineral matters carried in suspension. It is therefore desirable that the water in all parts of a distribution system be kept in motion as much as possible. If but one house be located in the southwest block of Fig. 18, as at *W*, the water between it and the dead-end, *V*, will lie stagnant, and if it become putrid may contaminate the water entering *W*. Or if no water be used at *W* for some time, the first to be drawn may be unfit for use. Another objection to the dead-end is that water drawn from the fire-hydrant *V* must all pass through the pipe *VM*; but if pipe be laid along the dotted lines, connecting with the system at *V*, *X*, and *Y*, water will reach the hydrant from both directions, and the velocity of flow and friction-head in *MV* will be reduced and the discharge from the hydrant increased. For both these reasons dead-ends are to be avoided where possible. When they cannot be, a fire-hydrant or other method of flushing out the "dead" water should be provided at each dead-end, and opened at intervals throughout the year.

It is evident that, with a system of pipes in which the water may circulate freely in all directions, the proper determination of sizes is a matter of some difficulty, since water will reach any point not in a dead-end through a number of pipes, and not through one alone. This determination will be considered in Art. 94, and the material and strength of pipes in the same article.

If there be considerable difference of elevation in different sections of a city, a desirable pressure-head in the upper ones will occasion an excessive head in the lower, if they be connected. This difficulty is overcome by dividing the city into two or more districts, each with its own system, and generally its own distributing-reservoir. In a few cases the "low service" system receives its supply from the "high service," but a pressure-regulator is placed at the junction, which cuts off the supply when the pressure in the lower system exceeds a certain amount. The great damage which might result should the regulator fail to either open or close when intended makes this plan a risky one, however.

The city of Providence, R. I., has constructed in the low-service district a supplementary fire system of mains of extra strength, which it has connected with its high service, and thus obtains high pressure for fire service in the lower and business part of the city.

QUERIES.

15. The Sweetwater Reservoir is connected with the irrigation-fields by a closed conduit. Had this been an open canal in earth, 10 feet wide and 10 miles long, what percentage of the supply would have been lost by evaporation and seepage, placing the last at 9 inches per day?

16. Calculate the area of cross-section of the above canal, if the velocity be 3 feet per second; all the available mean supply being used in 120 days.

CHAPTER X.

PUMPING SYSTEMS.

ART. 58. WHERE REQUIRED.

WHERE the impounding reservoir, stream, or lake forming the source of supply is at a lower elevation than the land to be irrigated or the highest building to be supplied thereby, the water must be raised to such elevation. Even though the reservoir be at a greater elevation, if the excess is not sufficient to overcome the friction in the conduit, pumping must be resorted to. Ground-waters are occasionally found at such elevations that pumping is unnecessary; but in most cases they must be raised from beneath the ground-surface, in some instances several hundred feet even. As a general statement, subject to many exceptions, however, impounded surface-supplies do not need pumping; while river, lake, and ground-waters do. The majority of exceptions to the latter statement are found among irrigation systems, where the water is not required above the surface of the ground.

Since a pump, like any mechanism, is subject to interruption or discontinuance due to breaks or other causes, while gravity never ceases to act, pumping is to be avoided where possible. If a ridge somewhat higher than the hydraulic gradient of a gravity conduit interpose in the line of this, it will generally be better to tunnel it than to pump over the summit. If such a tunnel cost less than a pumping plant plus the capitalization of all expenses of pumping and renewal, it offers the more economical solution of the problem, as well as the more desirable.

ART. 59. GENERAL DESIGN.

The water which is raised by a pumping plant must pass through a pressure-pipe, or pumping-main, either directly to the point of utilization, or to a storage- or distributing-reservoir, or to a standpipe. The first is called the direct-pumping system. This requires, aside from the distribution system, the construction necessary to conduct the water to the pump; the pumping plant and building in which it is housed; and the pumping-main. The pressure in a city supply, with direct pumping, depends upon that in the pump-cylinders, and when the pumps cease working the pressure falls to zero or to that of a gravity system. If the pump in such a plant breaks down, the supply is immediately cut off, unless a duplicate plant be provided; which is always desirable, but most so in a direct-pumping system. This system is not advised for city supplies, but is generally satisfactory for irrigation purposes.

In pumping to a reservoir, or an indirect-pumping system, there is required a pumping-main, a reservoir, and a conduit from this to the point of utilization. In some plants the pumping-main passes through and forms a part of the distribution system, and also acts as the conduit between this and the reservoir; the water passing directly to the distribution system, but being augmented by that from the reservoir when the amount pumped is less than the consumption; and the surplus, when it is greater than the consumption, passing to the reservoir. Such a system is called a direct-indirect.

In the direct-pumping system the pressure in a city supply is made to vary constantly by variations in the rate of consumption which the rate of pumping cannot be made to follow exactly. Moreover when, as in the case of fire, a greater demand is unexpectedly made upon the supply, the pumps may not for some time be able to meet it with increased delivery. For these reasons the indirect or direct-indirect system is preferable.

When, for want of an elevated site or other reason, a reservoir cannot be had, a water-column is frequently introduced to equalize the pressure; and if it be given considerable size of cross-section it will serve to temporarily supply a deficiency in pump-delivery. Such a water-column generally takes the form of a standpipe or water-tank. Where a water-cushion or pressure-regulator alone is desired a tall pipe of small section is used; as at Wichita, Kan., where a pipe 2½ feet in diameter and 150 feet high was erected. Where storage also is desired the cross-section area is considerably increased, as at Houston, Tex., which erected a standpipe 30 feet in diameter and 150 feet high, holding 144 times as much water as the former. The standpipe may be placed at or near the pump, or at some point in the distribution system; the highest ground in or near the city being ordinarily chosen.

The storage- or distributing-reservoir for a pumping system differs in no important particular from that in a gravity system. In order to reduce the length of pumping-main, in the indirect system the reservoir is placed as near the pump as sufficiently high ground can be found; but in the direct-indirect it is preferable to locate pump and reservoir on opposite sides of the city.

The pumping machinery must always be at the point of supply, or at a location which can be reached by gravity from such point. In some instances it is necessary or more economical to place one or more additional pumping-plants along the line of the pumping-main; as on the Coolgardie pumping-main, which is 328 miles long, the friction-head in which, if all supplied at one point, would require an almost impossible strength of pipe and pumps.

Art. 60. Intakes and Pumping-plants.

If water is taken by the pumps from a reservoir, the suction can pass from the pumps directly into this, or water can be led from the reservoir to a suction-well placed under or near the pumps. In either case it is desirable to place a gate-house in the reservoir, by means of which water can be drawn from any desired level of this.

If the supply is from tube-wells, these are practically made to serve as parts of the suction, and are all connected to a pipe and through this to the pump. This suction-pipe, all the connections, and the wells should be absolutely air-tight, since if they are not, a part of the energy of the pump will be wasted in sucking in air, and the pump itself may be damaged if this enters the cylinders. The collecting-pipe, or that to which all the wells are connected, should be at as low an elevation as the pumps. If wrought-iron pipe is used for this, particular pains should be taken to obtain perfect screw-threads, that the joints may be made tight. Cast-iron flanged pipe is better for this purpose, as tighter joints can be made with this. The size of the pipe should be such that the velocity of flow through it will not exceed one or two feet per second; and the pumps should be so located as to make the collecting-pipe as short as possible.

If the supply is from deep, non-artesian wells in which the water does not rise higher than a point 40 or more feet below the surface, a pump must be placed in each well to lift the water to the surface. If it is to be raised still higher, an additional surface pumping-plant is used, drawing water from a suction-well into which the deep-well pumps discharge.

If the supply is from a river, care should be taken to locate the intake at that point which will give the purest supply at all times. It should not be placed in an eddy, or in shallow or slack water at one side of the river, since matter in suspension is apt to collect here. It should always be at a distance below the surface at least twice as great as the diameter of the intake opening. It should be at such an elevation above the bottom that no sand or slit carried by the bottom layer of water can enter it. It should be placed below rather than above any pond or slack-water, unless additional pollution is added to the water there. It should generally be placed above the city, to avoid the pollution by street-drainage which probably enters the river, even if no sewers discharge into it. A screen is generally placed over the intake to exclude fish and floating matter; and it should be contained in and protected by a crib or masonry structure.

If the supply be from a lake it is generally desirable to place the intake at the lower end of the lake, in or near the channel, and to the windward rather than the leeward of this. It should be as far as practicable from the outlets of any sewers or polluted streams, being if possible on the opposite side of the main current from them.

The obtaining of these conditions often requires expensive construction. Heavy masonry intake-towers in deep or swift water; long and large intake pipes or tunnels connecting these with the pumps; and deep pump-pits protected from heavy floods in the adjacent river, are all called for in some instances. On the Great Lakes intakes are placed four or five miles from shore, and in the larger rivers of the Mississippi basin are intake-towers 50 to 140 feet high, with openings for taking water at various levels.

The intake-pipes should be well buried in the bed of the river or lake, to prevent injury by currents or waves, by the anchors of boats, or from any other cause. They are sometimes made of wrought-iron pipe imbedded in concrete, but cast iron is more substantial and durable if given an unyielding bed.

Anchor- or slush-ice collecting and freezing around intake-openings gives a great deal of trouble in some localities. This is caused by needles of ice which are prevented from freezing together by the motion of the water, and are drawn into the intake in this shape, generally freezing upon and choking the screen. These needles are not found in calm water, where they collect into sheet ice on the surface, and seldom on the windward side of a lake or river; and are not often drawn into the intake when this is well below the surface, and of such size that the velocity of flow into it is less than one-half foot per second.

The motive powers adopted for raising water include man or animal power; water-wheels and other hydraulic machinery; windmills; gas-, gasoline-, oil- and hot-air-engines; steam-engines; electric motors; compressed-air-engines; and in fact all kinds of motive power are used for driving pumps. Compressed air and steam are also used directly without the medium of pumps, in the air-lift and steam-siphon.

The pump and motor are generally placed in the same building, and in most cases are really parts of the same machine. Animal power is used but little if at all for any but private supplies. Windmills are used on few public supplies, not being adapted to pumping large quantities of water, and depending upon uncertain winds for power. Gas-, gasoline-, oil-, and hot-air-engines give excellent service in many small plants, but have not so far been used in large ones; and the same may be said of electric motors, although these and compressed-air-engines will probably come more into use in the near future. Hydraulic machinery has been used in a number of large plants, and gives excellent satisfaction where the water-power is always ample. There are not many locations, however, in which this is not likely to fail in times of drought, when the demand for water is greatest. It is ordinarily cheaper, however, than most of the other motive powers, and the rate of pumping can be increased at once to meet the demands of a fire upon a direct-pumping plant. A supplementary steam-plant is in some cases provided in connection with a hydraulic plant, to provide for pumping during low water.

By far the greatest number of pumping-plants have steam as a motive power; this being adapted to the largest as well as the smallest engines, and being under control at all times; although an immediate increase in delivery in case of fire cannot be obtained.

In deciding upon the motive power, consideration should be had of the accessibility of the plant and its general location. An electric plant requires within reasonable distance an electric power-station where power can be generated more cheaply than at the pumping-station; hydraulic machinery, a constantly abundant supply of water-power. Steam-plants require coal, wood, oil, or gas to supply the heat, and the means of reaching the plant with these must be considered. It may be more economical to place a steam-plant using coal near a railroad and some distance from the source of water-supply, and bring the water by gravity to the pumps, even though this require a somewhat greater lift and cost of construction.

In the pump proper, as used in city supplies, there is little variation except in detail. There are two general designs; the most efficient and one most commonly used consists essentially of a piston and valves so arranged that, by a reciprocating motion of the piston, water is sucked in through one set of valves and forced out through another; in the other, rotating valves or blades provide the suction and propelling power. The former are called reciprocating pumps, the latter rotary pumps.

There are other methods of raising water for irrigation; as by hydraulic rams, water-wheels, and other water-driven machines; and many devices for the direct lifting of water in buckets or barrels, few of which are used in this country except in private systems.

METHOD OF SUPPLY TO CITIES AND TOWNS IN THE UNITED STATES IN 1897.

Method of Supply.	Districts.				Total.
	North-eastern	South-eastern	North Central	West-ern.	
Gravity	490	41	11	194	736
Gravity and Pumping:					
Direct	62	7	2	15	86
To reservoir	38	1	2	13	54
To stand-pipe	11	2	2	5	20
Direct and to reservoir	1	0	0	1	2
" " " stand-pipe	3	1	3	1	8
To reservoir and stand-pipe	4	0	0	8	12
Total	119	11	9	43	182
Pumping:					
Direct	74	33	221	90	418
To reservoir	128	62	79	114	383
To stand-pipe	245	139	218	358	960
To reservoir and stand-pipe	26	11	14	20	71
Direct and to reservoir	33	11	27	45	116
" " " stand-pipe	41	20	115	185	361
" " " reservoir and stand-pipe	3	2	18	10	33
Total	550	278	692	822	2,342
Natural pressure	0	9	2	10	21
Grand total	1,159	339	714	1,069	3,281

A pump must not be placed at such an elevation above the water that it cannot lift this to its own level; which distance cannot ordinarily exceed 22 to 25 feet, although theoretically a 34-foot lift is possible. This limitation may necessitate placing the pump far underground in the case of non-artesian deep wells. Where a river is subject to a great rise of level during floods, the pump must be placed within 25 feet of the low-water level, although this may be below the flood-level, and in such a case provision must be made to protect the pump from flood-water.

The table on the preceding page shows the number of gravity, direct, indirect, and direct-indirect pumping systems in the United States in 1897.

QUERY.

17. In a city of 10,000 inhabitants is a direct-indirect system, with standpipe 150 high on such level ground that the bottom 100 feet can furnish no supply for fire-protection. If it requires two hours at night to get the steam and pumps ready for fire-service, what should be the diameter of the standpipe, the average discharge of one nozzle being taken as 200 gallons per minute?

CHAPTER XI.

HYDRAULICS.

ART. 61. STATICS.

IT is not intended to give here a demonstration of the principles of hydraulics; for these, reference may be made to any good treatise on this subject. But it is thought desirable to place here for convenient use a brief summary of the general principles and formulas applicable to water-supply engineering; also tables of coefficients and data for actual use.

(1) The pressure on any surface due to water not in motion (the surface of this water, or of some body of water with which it is freely connected being open to ordinary atmospheric pressure) is equal to the area of this surface, times the difference between the elevation of its centre of gravity and the elevation of the free surface of water, times the weight of a unit volume of water. Pressures are usually expressed as so many pounds per square inch, and the pressure on a square inch of any surface is approximately $P = 0.434H$ pounds, in which H is the difference of level between the free water-surface and the point in question, expressed in feet.

(2) When water is motionless in a system of pipes, throughout which it has free communication, it will stand at the same level in all connected branches wherever free to do so; also the same pressure will be exerted by the water in every part of the system which lies at the same level; and

the difference in pressure per square inch exerted at any two points will be 0.434 times their difference of elevation in feet. For example, if a pump exert by its piston a pressure of 100 lbs. per square inch (there being no motion of water in the pipes), at a point 50 feet higher the pressure would be $100 - (50 \times 0.434)$ or 78.3 lbs.

(3) The pressure of water is the same in all directions on all points in the same horizontal plane.

(4) Water pressure is always normal to the surface with which it is in contact. Thus in Fig. 19 the pressure is perpendicular to the back of the dam. If it were not, but were for instance horizontal, it could be resolved into two components, one normal and one parallel to the surface, and the water would rise up over the dam under the influence of the latter.

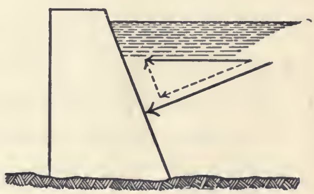


FIG. 19.—PRESSURE ON INCLINED SURFACE.

(5) By resolving a pressure into its components the pressure in any given direction may be found; but it follows from (4) that there can be no component along the surface under pressure.

(6) From the above it is demonstrable that the resultant pressure in any given direction, exerted outward from within or inward from without, upon a given surface of any pipe, vessel, or solid body containing or immersed in water, equals the pressure in this direction upon the projection of such surface normal to such direction and passing through the centre of gravity of such surface; this applying to plane surfaces, and to horizontal pressures on all surfaces; and approximately to all cases where the height of a vertical projection of said surface is small relative to the pressure head upon it.

The above statements are true only when the surface in question is free to normal atmospheric pressure, and is strictly true only when such pressure and that upon the free water-surface are the same. As a matter of fact, the pressure exerted by the water is the amount given above plus that exerted by the atmosphere upon the water-surface (about 14.7 lbs. per square inch); but is opposed, directly or indirectly, by the atmospheric pressure exerted in the opposite direction upon the surface considered.

(7) The centre of pressure upon any rectangular surface immersed in water and having one end coincident with the water-surface is in the line which, being in the rectangle, bisects the line of intersection of these two surfaces and is perpendicular to it, being one third the length of this line above the lower end of the rectangle. The rectangular surface may be vertical or inclined; but only the wetted surface is considered, any which is above the water not affecting the problem.

(8) The centre of pressure upon any parallelogram, rectangle, or square, having one edge in or parallel to the water-surface, may be found graphically as follows.

Let *abcd* be a parallelogram, *ab* and *dc* being parallel to the water-surface *wx*. Let *f* be the middle of *dc*, and *l* the middle of *ab*; continue the line *lf* to the water-surface, as at *e*. Draw *ea* and *eb*; also *gk* parallel to *ea*, and *gr* parallel to *fl*. Draw *hi* parallel to *ab* and *cd*, and half-way between them. Let *n* be the centre of *ho*, and *mr* equal $\frac{1}{3}rg$.

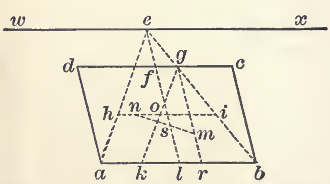


FIG. 20.—CENTRE OF PRESSURE UPON A PARALLELOGRAM.

Connect *m* and *n*; and where *mn* cuts *el*, or *s*, is the centre of pressure.

(9) The general rule for the centre of pressure of an immersed plane surface is: The distance from the water-surface to the centre of pressure, measured in the plane of the immersed surface and normal to the line of its intersection

with the water-surface, equals the quotient of the moment of inertia of such surface by the static moment, both with reference to that line, as an axis, which is formed by the intersection of the plane with the water-surface.

(10) The pressure upon the bottom of an immersed body is greater than that upon the top; and if the sum of the vertical downward components of the pressure upon all sides of such a body plus its weight be greater than the sum of the vertical upward components, the body will sink; if such sum be less, the body will rise and float upon the surface. A floating body will sink in the water to such a depth that the weight of water displaced by it just equals its own weight.

The force necessary to support an immersed body, or its downward pressure, will be as much less in water than in air as is the difference between the downward and upward vertical components of the pressure upon it; which is the weight of the volume of water displaced. If water finds its way under any part of a stone dam, the effective weight of such dam is reduced by 62.5 lbs. per square foot of horizontal wetted joint, times the depth of such joint beneath the water-surface.

Art. 62. Flow in Open Conduits.

The flow in a conduit is due to gravity alone, as brought into action by a fall in the water-surface. The velocity is affected by the size and form of cross-section of the conduit, and the nature of its surface. The velocity at any particular point may, however, be increased or decreased by conditions above or below such point. The ordinary formulas for flow consider only cases of uniform velocity and cross-section for some distance above and below as well as throughout the section under discussion.

Flow has been found to vary to a certain extent with the relation between the cross-section of the stream and the wetted part of the perimeter; and this relation, or $\frac{\text{cross-section}}{\text{wetted perimeter}}$, has been given the name "hydraulic radius," and is customarily represented by r. For a semi-circular or circular cross-section of flow r is one fourth the diameter.

(11) The velocity given by formulas is the mean velocity. That at the surface and along the wetted perimeter is less than this (except when wind blowing down-stream increases the surface velocity). The mean velocity in rivers is generally about 98% of the mid-depth velocity and from 70 to 85 per cent of the maximum surface velocity.

(12) The Chézy formula, $V = c\sqrt{rs}$, is generally used as the basis of velocity formulas, where V is the mean velocity in second-feet, c is a constant, r is the hydraulic radius, and s is the tangent of the slope or rate of fall of the surface (and also of the conduit).

Values of c adapted to different conditions have been ascertained by experiment. For metal, wood, concrete, or smooth masonry conduits of circular section Hamilton Smith gives the following values for c:

TABLE No. 46.

COEFFICIENT c FOR SMOOTH CIRCULAR OR SEMI-CIRCULAR CONDUITS.

Velocity V.	Diameters. $D = 4r$.												
	.05	.10	1	1.5	2	2.5	3	3.5	4	5	6	7	8
1 (?)	......	80.0	96.1	102.8	108.8	112.7	116.7	120.2	123.0	127.8	131.8	134.8	137.5
2	77.8	88.9	104.0	110.9	116.2	120.3	123.8	127.0	129.9	134.3	138.0	141.0	143.3
3	82.4	93.7	108.7	115.6	120.8	124.8	128.3	131.4	134.2	138.6	142.3	145.4	147.6
4	85.6	97.0	112.0	118.9	124.0	128.1	131.5	134.6	137.4	141.9	145.5	148.6	151.0
5	87.6	99.3	114.4	121.3	126.5	130.6	134.1	137.1	140.0	144.7	148.1	151.2	153.6
6	89.1	101.0	116.3	123.2	128.6	132.6	136.3	139.4	142.3	146.9	150.5	153.5	
7	90.0	102.4	118.0	125.0	130.4	134.6	138.2	141.5	144.5	149.0	152.7		
8	90.6	103.3	119.3	126.4	132.0	136.3	140.0	143.3	146.3	151.0	154.9		
9	90.7	104.0	120.4	127.7	133.3	137.7	141.6	145.0	148.1	152.8	156.7		
10	90.8	104.5	121.4	128.8	134.5	139.0	142.9	146.4	149.7	154.6			
11	90.9	104.7	122.0	129.7	135.6	140.2	144.2	147.7	151.0				
12	91.0	104.8	122.5	130.4	136.4	141.1	145.2	148.8	152.2				
13	91.0	105.0	122.9	131.0	137.1	141.0	146.1	149.8	153.2				
14	91.0	105.0	123.2	131.5	137.6	142.5	146.7	150.5	154.0				
15	91.0	105.0	123.6	131.8	138.0	142.9	147.2	151.1	154.6				
20 (?)	......	......	123.9	132.9									

Smith also gives the following values for rectangular conduits, $b =$ breadth of conduit, $d =$ depth of water flowing.

TABLE No. 47.

COEFFICIENT c FOR RECTANGULAR AND TRAPEZOIDAL CONDUITS.

Rectangular Conduits.										For depth of 1.64 ft., sides inclined 45°, and then vertical.	
Pure Cement. $S =$ .0049 $b = 5.94$		Brick (not very smooth). $S =$ .0049 $b = 6.27$		Unplaned Plank. $S =$ .00824 $b = 6.53$		Unplaned Plank. $S =$ .0015 $b = 6.51$		Unplaned Plank. $S =$ .006 $b = 1.575$		Unplaned Plank. $S =$ .0015 Bottom $b = 3.28$	
d	c	d	c	d	c	d	c	d	c	d	c
.18	116.5	.20	89.7	.15	101.4	.30	88.3	.34	94.5	.92	103.1
.28	125.1	.31	98.3	.25	101.4	.46	96.3	.44	97.3	1.19	110.4
.38	126.9	.41	98.8	.32	107.1	.72	104.2	.50	98.3	1.40	113.0
.43	132.4	.49	103.7	.38	109.8	.92	110.4	.53	97.1	1.55	116.2
.50	132.4	.57	105.1	.45	109.7	1.10	115.1	.62	102.3	1.69	117.6
.56	135.1	.65	103.7	.50	113.1	1.27	119.1	.70	104.6	1.82	117.3
.63	135.5	.71	106.3	.54	115.8	1.44	120.4	.79	105.3	1.93	118.0
.69	136.2	.77	109.0	.60	115.2	.....	......	.87	105.7	2.03	118.4
.76	137.2	.85	107.4	.65	115.8	.....	......	.95	107.9	2.13	119.0
.80	137.2	.90	110.8	.69	116.9	.....	......	.....	......	2.22	119.5
.86	137.8	.97	109.7	.74	117.1	.....	......	.....	......	2.30	119.4
.91	138.2	1.04	108.7	.78	119.0	.....	......	.....	......	2.37	120.9

For earth channels Kutter's formula probably gives the most accurate values for c, as also indeed for those referred to by the above tables. This formula, with feet and seconds as the units of measurement, is as follows:

$$c = \frac{\dfrac{1.81}{n} + 41.65 + \dfrac{0.0028}{s}}{1 + \dfrac{n\left(41.65 + \dfrac{0.0028}{s}\right)}{\sqrt{r}}}; \tag{13}$$

in which n is a coefficient of roughness of the wetted surface of the conduit, and is given values as follows:

(14) For channels of well-planed timber	.009
" " " neat cement, glazed sewer-pipe, or very smooth iron pipe, and butt-joint w. i. pipe.	.010
" " " 1 : 3 cement mortar or smooth iron pipe	.011
" " " unplaned timber, and ordinary cast-iron and lap-joint w. i. pipe	.012
" " " smooth brickwork	.013
" " " ordinary brickwork or smooth masonry	.015
" " " rubble masonry	.017
" " " firm gravel	.020
" " " rivers free from stones and weeds	.025
" canals and rivers with some stones and weeds	.030
" " " " in bad order	.035

If the velocity V be calculated for a given hydraulic radius and grade with a value for n of .011, an approximate value of V for other values of n can be obtained directly by multiplying the first value of V by

1.25	when	the	second n	is	.009
1.10	"	"	"	"	.010
1.00	"	"	"	"	.011
.90	"	"	"	"	.012
.80	"	"	"	"	.013
.70	"	"	"	"	.015
.60	"	"	"	"	.017

Considerable care must be used in selecting the proper value of n; and an actual measurement of the flow is always to be preferred to the best formula used with the most expert judgment. Pipes of metal or wood being of a uniform known material and cross-section, flow through them is capable of more accurate determination than that through channels of earth or ordinary masonry. These will be taken up more at length in Art. 63.

(15) In passing around curves the greatest velocity will be along the outer side, and that along the inner side may be zero, or even a reversed current or eddy. There will generally be cross-currents here, also, from the inner to the

outer side on top and in the reverse direction on the bottom; and the result of these will be erosion of the outer bank of a canal or river and deposits on the inner. Velocity at curves should therefore be small, or the bank should be paved or otherwise protected from erosion. The cross-currents also cause friction between the particles of water, which friction must result in a loss of head.

(16) The energy stored in a moving body of water varies directly as its mass and as the square of its velocity, or $E = \frac{W}{2g}V^2$, in which W is the weight in pounds of water passing a given point in one second, g is 32.16 feet, and V is the velocity of flow in feet per second. Since W varies as V, E varies as V^3.

(17) The impulse of a moving body of water equals $\frac{W}{g}V$, and hence varies as the square of the velocity. Any curve in the confining channel of a body of flowing water receives an impulsive pressure in the direction of original flow and must be strengthened to resist such pressure.

(18) The amount of pressure received is

$$\left(\frac{W}{g}V\right)(1 - \cos\theta),$$

in which θ is the angle between the original direction of flow and that after deflection. If θ is greater than 90°, cos θ becomes minus, and the second factor of the equation becomes greater than 1.

(19) A contraction or expansion of the area of cross-section by an offset causes a loss of head due to the eddies created; which can be avoided by sloping or rounding the offset so as to provide a gradual increase or decrease of sectional area.

(20) The theoretic value of the head lost in enlargement by a right-angle offset is

$$H_l = \frac{(V - V_1)^2}{2g},$$

in which V is the velocity of flow before, and V_1 that after enlargement.

(21) The theoretic value of the head lost in contraction by a right-angle offset is

$$H_l = C\frac{V_1^2}{2g},$$

in which C varies between 0 and $\frac{1}{8}$, according to the ratio between the two areas of cross-section.

This loss may be caused by projections in the sides or in the bottom of a canal or flume, and may be utilized to reduce velocity of flow without decreasing the fall. But the loss is due to eddies, and provision must be made for resisting erosion by these.

(22) If a river, canal, or other body of flowing water be obstructed by a weir, the surface elevation is raised at such point, and for a considerable distance back from such point, "back-water" being thus formed. A high overfall dam, or weir, in a river may cause considerable land to be flooded, and it is desirable to know to what extent this flooding will reach during both high water and the ordinary flow. This can be determined approximately from the formula

$$L = \frac{d_2 - d_1}{i} + D\left(\frac{1}{i} - \frac{c^2}{g}\right)\left[\phi\left(\frac{d_1}{D}\right) - \phi\left(\frac{d_2}{D}\right)\right];$$

in which d_2 is the depth of water just back of the weir, d_1 its depth at a distance L above the weir, i is the slope of the river-bed and original water-surface, D is the original depth of water, c is the coefficient in the formula $V = c\sqrt{rs}$, and $\phi\left(\frac{d_1}{D}\right)$ and $\phi\left(\frac{d_2}{D}\right)$ are abbreviations for logarithmic functions of these quantities, the values of which functions are obtainable from Table No. 48 (from Merriman's "Hydraulics," which see for fuller discussion).

TABLE NO. 48.

VALUE OF THE BACK-WATER FUNCTION $\phi\left(\frac{d}{D}\right)$.

$\frac{D}{d}$	$\phi\left(\frac{d}{D}\right)$	$\frac{D}{d}$	$\phi\left(\frac{d}{D}\right)$	$\frac{D}{d}$	$\phi\left(\frac{d}{D}\right)$	$\frac{D}{d}$	$\phi\left(\frac{d}{D}\right)$
1	∞	.948	.8685	.815	.4454	.52	.1435
0.999	2.1834	.946	.8539	.810	.4367	.51	.1376
.998	1.9532	.944	.8418	.805	.4281	.50	.1318
.997	1.8172	.942	.8301	.800	.4198	.49	.1262
.996	1.7213	.940	.8188	.795	.4117	.48	.1207
.995	1.6469	.938	.8079	.790	.4039	.47	.1154
.994	1.5861	.936	.7973	.785	.3962	.46	.1102
.993	1.5348	.934	.7871	.780	.3886	.45	.1052
.992	1.4902	.932	.7772	.775	.3813	.44	.1003
.991	1.4510	.930	.7675	.770	.3741	.43	.0995
.990	1.4159	.928	.7581	.765	.3671	.42	.0909
.989	1.3841	.926	.7490	.760	.3603	.41	.0865
.988	1.3551	.924	.7401	.755	.3526	.40	.0821
.987	1.3248	.922	.7315	.750	.3470	.39	.0779
.986	1.3037	.920	.7231	.745	.3406	.38	.0738
.985	1.2807	.918	.7149	.740	.3343	.37	.0699
.984	1.2592	.916	.7069	.735	.3282	.36	.0666
.983	1.2390	.914	.6990	.730	.3221	.35	.0623
.982	1.2199	.912	.6914	.725	.3162	.34	.0587
.981	1.2019	.910	.6839	.720	.3104	.33	.0553
.980	1.1848	.908	.6766	.715	.3047	.32	.0519
.979	1.1686	.906	.6695	.710	.2991	.31	.0486
.978	1.1531	.904	.6625	.705	.2937	.30	.0455
.977	1.1383	.902	.6556	.70	.2883	.29	.0425
.976	1.1241	.900	.6489	.69	.2778	.28	.0395
.975	1.1105	.895	.6327	.68	.2677	.27	.0367
.974	1.0974	.890	.6173	.67	.2580	.26	.0340
.973	1.0848	.885	.6025	.66	.2486	.25	.0314
.972	1.0727	.880	.5884	.65	.2395	.24	.0290
.971	1.0610	.875	.5749	.64	.2306	.23	.0266
.970	1.0497	.870	.5619	.63	.2221	.22	.0243
.968	1.0282	.865	.5494	.62	.2138	.21	.0221
.966	1.0080	.860	.5374	.61	.2058	.20	.0201
.964	0.9890	.855	.5258	.60	.1980	.18	.0162
.962	0.9700	.850	.5146	.59	.1905	.16	.0128
.960	.9539	.845	.5037	.58	.1832	.14	.0098
.958	.9376	.840	.4932	.57	.1761	.12	.0072
.956	.9221	.835	.4831	.56	.1692	.10	.0050
.954	.9073	.830	.4733	.55	.1625	.06	.0018
.952	.8931	.825	.4637	.54	.1560		
.950	.8795	.820	.4544	.53	.1497		

(23) The weight of individual particles of foreign matter which a given stream can carry varies (theoretically) as the sixth power of the velocity. The total amount which a stream can carry is uncertain, but has been found to be as high as 10% of the weight of water. The amount carried, as well as the size transportable, varies as some power of the velocity, and hence a retardation of the velocity tends to cause deposits. The following table, from Dubuat, is deduced from experiments in wooden troughs. Channels formed of such materials firmly compacted will not ordinarily be eroded by several times these velocities.

TABLE No. 49.

POWER OF CURRENTS TO TRANSPORT LOOSE MATERIALS.

Material.	Mean Velocity of Current, Ft. per Sec.
Fine clay	0.4
Fine sand	0.5
Small gravel	0.8
Large gravel	1.6
Water-worn gravel, about one inch diameter	3.5
Angular stones, 1½ to 2 inches diameter	4.5

ART. 63. FLOW IN PRESSURE CONDUITS.

The Chézy formula (12) is adopted as the basis of velocity formulas for pressure conduits also, c being determined for these by experiment. More experiments have been made upon cast-iron pipe than upon any other; but in recent years the increasing use of wood-stave and riveted iron and steel pipes has led to experiments with these also.

The values of c for different sizes of pipe and different velocities of flow in new cast-iron pipes, adapted from Smith's "Hydraulics" and obtained by him from the results of a large number of experiments, are given in Plate XII.

(24) Tuberculated pipe will give a much smaller discharge than new, partly because of decreased diameter and partly because of the greater roughness of surface. A 4-inch pipe

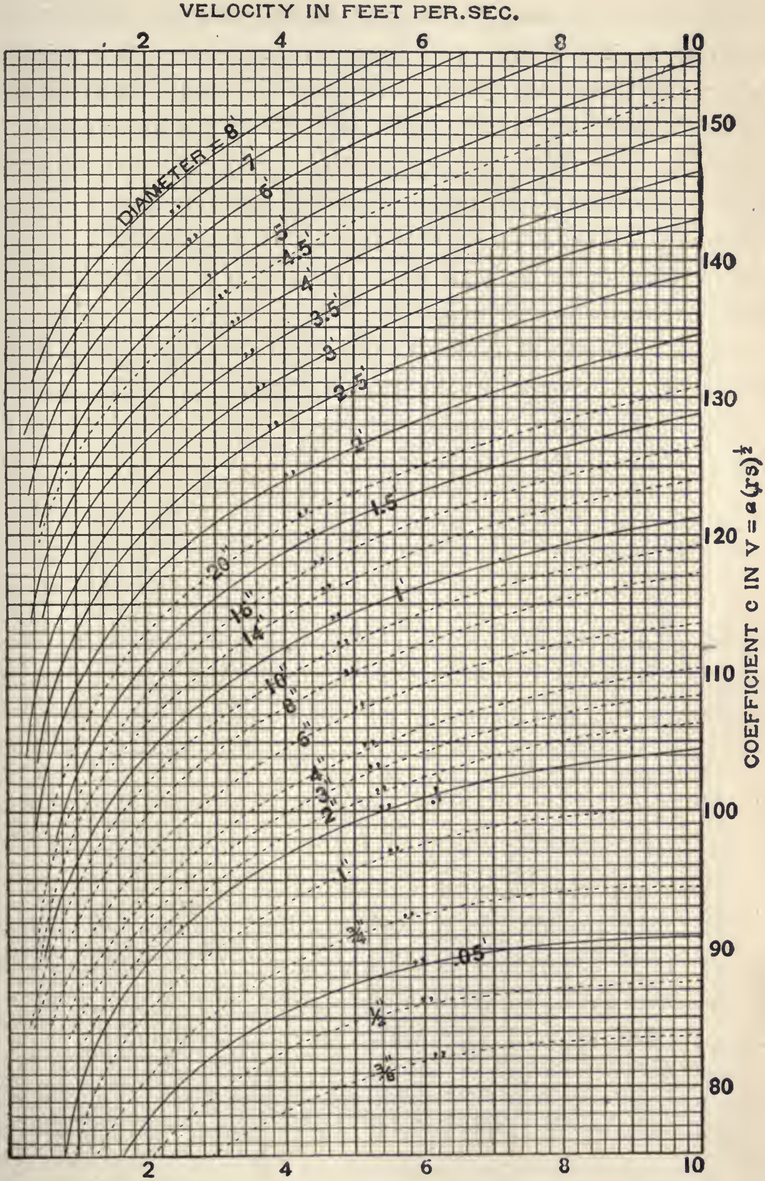


PLATE XII.—COEFFICIENT *c* FOR NEW CAST-IRON PIPES.
(From Coffin's "Graphical Solution of Hydraulic Problems.")

38 years old has been found to discharge but one eighth the amount indicated by the above diagram for a new 4-inch pipe; and the discharge of a 48-inch pipe was decreased 25 to 30 per cent by tuberculation; the difference in percentage of decrease in these two pipes being largely due to the fact that tubercles seldom attain a height greater than 1 or $1\frac{1}{8}$ inch, regardless of the size of pipe, and the available cross-section was hence reduced by a much larger proportion in the 4-inch than in the 48-inch pipe.

The necessity of giving values for c corresponding to so many sizes of pipe and velocities is avoided by giving values for n in Kutter's formula; or for f (called the "friction factor") in the theoretic formula

$$V = \sqrt{\frac{2ghd}{fl}}; \tag{25}$$

in which h is the head lost in overcoming friction in a pipe of a length l and diameter d. The table on page 219 gives values for f in this formula, or in its modified form,

$$h = \frac{flV^2}{2gd}, \tag{26}$$

for clean iron pipe, laid with close joints.

For very smooth pipes, similar to lead or brass, $3\frac{1}{2}$ inches or less in diameter, *and for these only*, the following formula is proposed by E. B. Weston, being deduced by him from a comparative study of a large number of experiments:

$$f = 0.0126 + \frac{0.0315 - 0.06d}{\sqrt{V}},$$

from which formula the table on page 220 was **prepared.** (See Trans. Am. Soc. C. E., vol. XXII. page 55.)

TABLE No. 50.

FRICTION FACTORS FOR PIPES.

(Compiled by Merriman from Smith, Fanning, and others.)

Diameter in Feet.	Velocity in Feet per Second.						
	1	2	3	4	6	10	15
0.05	0.047	0.041	0.037	.034	.031	.029	.028
0.1	.038	.032	.030	.028	.026	.024	.023
0.25	.032	.028	.026	.025	.024	.022	.021
0.5	.028	.026	.025	.023	.022	.020	.019
0.75	.026	.025	.024	.022	.021	.019	.018
1	.025	.024	.023	.021	.020	.018	.017
1.25	.024	.023	.022	.020	.019	.017	.016
1.5	.023	.022	.021	.019	.018	.016	.015
1.75	.022	.021	.020	.018	.017	.015	.014
2	.021	.020	.019	.017	.016	.014	.013
2.5	.020	.019	.018	.016	.015	.013	.012
3	.019	.018	.017	.015	.014	.013	.012
3.5	.018	.017	.016	.014	.013	.012	
4	.017	.016	.015	.013	.012	.011	
5	.016	.015	.014	.013	.012		
6	.015	.014	.013	.012	.011		

By (25) $V=\sqrt{\frac{2ghd}{fl}}$. But for a full pipe $r=\frac{d}{4}$, or $d=4r$; and $h/l=s$; consequently this equation becomes $V=\sqrt{\frac{8g}{f}rs}$. Comparing this with the Chézy formula, $V=c\sqrt{rs}$, we find $c=\sqrt{\frac{8g}{f}}=\frac{16.04}{\sqrt{f}}$; and by this equation Plate XII can be compared with Tables Nos. 50 and 51.

(28) The value of n in Kutter's formula for new cast-iron pipe is approximately .0115, but varies with the velocity, especially for pipes under 18 or 20 inches diameter; although this may be due to the greater effect in small pipes than in large of variation in the rugosity of the surface, due to differences in the application of the tar-coating or to tuberculation.

TABLE No. 51.

FRICTION FACTORS FOR SMOOTH PIPES.

Velocity in Feet per Second.	Diameters of Pipes in Feet and Inches.									
	0.0417′ ½″	0.0521′ ⅝″	0.0625′ ¾″	0.0833′ 1″	0.1042′ 1¼″	0.1250 1½″	0.1667 2″	0.2083 2½″	0.2500 3″	0.2917 3½″
0.10	0.1043	.1023	.1004	.0964	.0924	.0885	.0806	.0727	.0648	.0569
0.50	0.0536	.0527	.0518	.0501	.0483	.0465	.0430	.0395	.0359	.0324
1.00	.0416	.0410	.0404	.0391	.0379	.0366	.0341	.0316	.0291	.0266
1.20	.0391	.0385	.0381	.0367	.0356	.0343	.0319	.0296	.0274	.0250
1.40	.0371	.0365	.0363	.0347	.0338	.0324	.0304	.0283	.0262	.0239
1.60	.0356	.0348	.0350	.0332	.0323	.0312	.0292	.0272	.0252	.0232
1.80	.0342	.0336	.0340	.0321	.0313	.0302	.0284	.0265	.0246	.0226
2.00	.0331	.0327	.0322	.0313	.0305	.0296	.0278	.0260	.0243	.0225
2.50	.0310	.0306	.0300	.0294	.0286	.0278	.0262	.0246	.0230	.0214
3.00	.0293	.0290	.0286	.0279	.0272	.0265	.0250	.0236	.0221	.0207
3.50	.0281	.0278	.0274	.0268	.0260	.0254	.0242	.0228	.0214	.0201
4.00	.0271	.0268	.0265	.0259	.0252	.0246	.0234	.0221	.0209	.0196
4.50	.0263	.0260	.0256	.0251	.0244	.0238	.0227	.0216	.0204	.0192
5.00	.0256	.0253	.0250	.0244	.0238	.0233	.0222	.0211	.0200	.0189
5.50	.0250	.0246	.0244	.0238	.0234	.0228	.0218	.0207	.0196	.0186
6.00	.0244	.0242	.0239	.0234	.0229	.0224	.0214	.0204	.0193	.0183
6.50	.0240	.0236	.0235	.0230	.0225	.0220	.0210	.0200	.0190	.0181
7.00	.0236	.0233	.0230	.0226	.0222	.0217	.0207	.0198	.0188	.0179
7.50	.0232	.0229	.0227	.0222	.0218	.0214	.0204	.0195	.0186	.0177
8.00	.0229	.0226	.0224	.0220	.0215	.0211	.0202	.0193	.0184	.0175
8.50	.0225	.0223	.0221	.0216	.0212	.0208	.0200	.0191	.0182	.0174
9.00	.0222	.0220	.0218	.0214	.0210	.0206	.0198	.0189	.0181	.0173
9.50	.0220	.0218	.0216	.0212	.0208	.0204	.0196	.0188	.0180	.0172
10	.0218	.0216	.0214	.0210	.0206	.0202	.0194	.0186	.0178	.0170
11	.0214	.0212	.0210	.0206	.0202	.0198	.0191	.0184	.0176	.0168
12	.0210	.0208	.0206	.0203	.0199	.0195	.0188	.0181	.0174	.0166
13	.0206	.0205	.0203	.0199	.0196	.0193	.0186	.0179	.0172	.0165
14	.0204	.0202	.0200	.0197	.0194	.0190	.0184	.0177	.0170	.0164
15	.0201	.0200	.0198	.0195	.0191	.0188	.0182	.0175	.0168	.0162
16	.0199	.0197	.0195	.0192	.0189	.0186	.0180	.0174	.0167	.0161
17	.0196	.0195	.0194	.0190	.0187	.0184	.0178	.0172	.0166	.0160
18	.0194	.0193	.0192	.0188	.0186	.0183	.0177	.0171	.0165	.0159
19	.0193	.0191	.0190	.0187	.0184	.0181	.0175	.0169	.0164	.0158
20	.0191	.0189	.0188	.0185	.0182	.0180	.0174	.0168	.0163	.0157
25	.0184	.0183	.0182	.0179	.0177	.0174	.0169	.0164	.0159	.0154
30	.0179	.0178	.0177	.0174	.0172	.0170	.0165	.0161	.0156	.0152
35	.0175	.0174	.0173	.0171	.0169	.0167	.0162	.0158	.0154	.0150
40	.0172	.0171	.0170	.0168	.0166	.0164	.0160	.0156	.0152	.0148
50	.0167	.0166	.0165	.0163	.0162	.0160	.0156	.0153	.0149	.0146

(29) For wood-stave pipe, n and the friction-factor have generally been found less than for cast-iron pipe. For 30-inch pipe on the Denver water-works n was found to be .0096; for 18-inch on the Astoria water-works n was found

to be .00985, and c to be 132.88 when V was 3.605; and for 14-inch pipe n was found to be .0107 to .011, and c to be as follows:

Velocity in Feet per Second.	Corresponding Value of c.
0.7	102
1.2	111
1.5	112

An assumption of $n = .010$ is that generally made, and seems to accord well with experiments.

(30) For riveted pipe with butt-joints and countersunk rivets the coefficient c would probably be a little less than for wood-stave pipe, or n a little greater, on account of the tendency of the coating to pucker or form rugosities; when the rivets are not countersunk the velocity will be still less; and when lap-joints are used the coefficient diminishes with the thickness of the plates. For the 16-inch Astoria pipe, having plates .134 to .109 inch thick, c was found to be 112.3 when V was 4.58, and n was .011. This value was also obtained by Hamilton Smith for a similar pipe. For the East Jersey Water Company's pipe 4 feet in diameter, with plates 0.25 to 0.375 inch thick, n averaged 0.0148. For a wrought-iron riveted pipe at Holyoke, Mass., $8\frac{1}{2}$ feet diameter, with plates 0.36 inch thick, n was found to be .016. At Portland, Ore., n was found to be .0115; and c for 42-inch pipe, plates .203 to .375 inch thick, was 116; for 35- and 33-inch, plates .203 inch thick, c was 127 and 123 respectively.

The above formulas and coefficients are applicable only to uniform flow over an uninterrupted surface; and the h used is that lost in friction only, and not the total head. Where water enters a pipe there is a loss of head due to the contraction of the vein; a loss occurs in passing curves, partly closed gates, sudden contractions and expansions of area; and a part of the head is used in creating the velocity. That is, the total H existing between a free water-surface and any point in a connected pipe is divided up into several parts having different functions; or

$$(31) \qquad H = h_v + h_p + h_e + h_f + h_c + h_m;$$

in which h_e is the head lost at the entrance, h_f is that lost in friction, h_c that lost in curves, and h_m all other miscellaneous losses.

(32) The velocity of water flowing in any closed conduit is equal to $\sqrt{2gh_v}$, in which h_v is that portion of the total head on this point not consumed in overcoming friction or other resistance above this point, nor existing as pressure (h_p). This head is called the *velocity head*, and equals $\frac{v^2}{2g}$. The heads h_e, h_f, h_c, and h_m are lost for any practical use, being transformed into heat, but h_v and h_p are interconvertible and can both be utilized as work.

If water passes through an opening cut in a plate, having perfectly sharp inner edges and so formed that the water does not touch the plate after passing these edges, such a hole is called a standard orifice. In passing through such an opening the vein of water decreases in section for a short distance, and this contraction is called the "contracted vein." In the case of a circular opening the area of the smallest section is about .62 of the area of the orifice, or

(33) The coefficient of contraction c is 0.62. The velocity in this section is about .98 of the theoretical velocity, or the coefficient of discharge is .98 of 0.62, or 0.61. For a short, straight tube projecting into a reservoir and having sharp edges at its inner end

(34) The coefficient of discharge $c_d = 0.50$;

but if the tube is four or more times its diameter, we find

$$(35) \qquad c_d = 0.72;$$

and if a tube be fitted in front of a standard orifice of the same diameter, we find

$$(36) \qquad c_d = 0.816.$$

The head lost by the contraction is

$$(37) \qquad h_e = \left(\frac{1}{c_d^2} - 1\right)\frac{V^2}{2g},$$

which ranges in value from 0 to $.93\frac{V^2}{2g}$, being $.505\frac{V^2}{2g}$ for case (36).

By (26) we find $h_f = \frac{flV^2}{2gd}$.

$$(38) \qquad h_c = \frac{baV^2}{180 \times 2g},$$

in which a is the angle of curvature, and b has the following values for different diameters of pipe d, and radii of curvature R (according to Weisbach):

For $\frac{d}{R} = 0.2$	0.4	0.6	0.8	1.0	1.2	1.4	1.6	1.8	2.0
$b = 0.131$	0.138	0.158	0.206	0.294	0.440	0.661	0.977	1.408	1.978

(39) When a gate-valve is partly closed, the lost head due to this also is a function of $\frac{V^2}{2g}$, or $k\frac{V^2}{2g}$, in which, if the proportion of the vertical centre-height of the opening to the diameter of the pipe be called O, according to Weisbach

when $O = 1$	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{1}{4}$	$\frac{1}{8}$
$k = 0.0$	0.07	0.26	0.81	2.06	5.52	17.00	97.8

But these values were deduced from experiments on small valves only.

J. W. Smith, in 1894, found the discharge through a 30-inch gate-valve to be as follows:

Proportional height of gate	0.125	0.250	0.375	0.500	0.625	0.750	0.875	1.000
" area of opening	0.125	0.287	0.443	0.593	0.729	0.851	0.946	1.000
" discharge	0.086	0.178	0.274	0.384	0.533	0.734	0.936	1.000

(40) From (37), (26), (38), and (39),

$$H = \left(\frac{1}{c_d^2} + \frac{fl}{d} + \frac{ba}{180} + k + m\right)\frac{V^2}{2g};$$

in which m is a factor representing all miscellaneous losses.

(41) Also $V = \sqrt{\dfrac{2gH}{\dfrac{1}{c_d^2} + \dfrac{fl}{d} + \dfrac{ba}{180} + k + m}}$.

If the pipe is long, with few bends, and gates all open—the ordinary condition—most of these factors may be neglected, as insignificant compared with $\frac{fl}{d}$, and the equations reduce to (15) and (16).

(42) Since $V = c\sqrt{rs}$, and $r = \frac{V^2}{c^2 s}$, if c were constant the velocity in two pipes at the same grade would vary as the square roots of their diameters, and their diameters as the square of the velocities. c, however, is not constant, and it is found that V varies almost exactly as the $\frac{6}{11}$ power of s, and s as the $\frac{11}{6}$ power of V.

(43) If in Fig. 21 a pipe AB connect two reservoirs, but be closed at B, the water in each of the pipes CE, GL, and HK will stand on a level with the water-surface, c, by (2). There will be at G a pressure equal to $0.434GL$, and at H a pressure $0.434HK$. If B is opened, the pressure head at that point will be DB only; at E it will be EC minus the lost head h_e and also $\frac{V^2}{2g}$, or EM. If a line connect M and D,

cutting GL at F and HK at I, the pressure head at G is GF, and at H is HI; and the heads LF and KI have been lost in overcoming friction above the points G and H respectively. The line $MFID$ is called the hydraulic gradient. The dis-

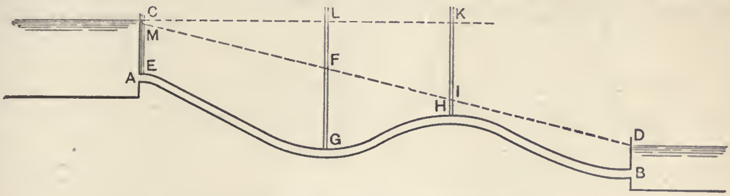


FIG. 21.—HYDRAULIC GRADIENT.

tance of this at any point below the level of the upper reservoir is the sum of $\frac{V^2}{2g}$, h_e, $\frac{fl}{d}\frac{V^2}{2g}$, and any other lost head h_m, when l is the distance from the reservoir to the point in question, measured along the line of pipe. If the pipe is straight and uniform in bore throughout, MD is a straight line. If the pipe rise at any point above the hydraulic gradient, the pressure head there becomes negative and the pipe is called a siphon. The water between this point and the outlet tends to run more rapidly than that above the siphon; but it can do so only if air be sucked in through leaks in the siphon or find its way up from the outlet. If this happens, the pipe above the siphon still flows under pressure, but that below it as an open conduit; and the pipe ceases to act as a siphon.

(44) If the pipe were of larger diameter at H than at G, the velocity V, and consequently $\frac{V^2}{2g}$, would be less, and hence KI would be less than it is. If, on the contrary, there were a contraction of the section at H, KI would become greater, and might cause the pressure head HI to become minus, or a vacuum to exist at H.

In Fig. 22, $B''L''$ is a pipe with numerous changes of section, and an orifice at L'', fed by a reservoir with a free surface at X, XL' being a horizontal line at the level of this surface. $XBCD \ldots L$ is the hydraulic gradient of this line. WB' is the head lost at entrance. WB is the velocity head. $CC' - BB'$ is the head lost in friction in the pipe $B''C''$. BB'' is the pressure head at B''; CC'' is the pressure head

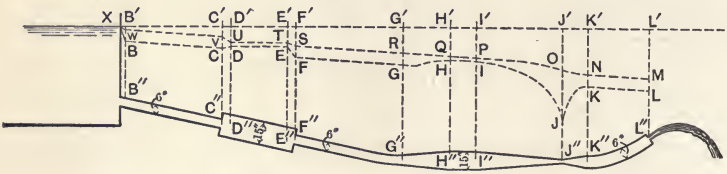


Fig. 22.—Hydraulic Gradient, Compound Pipe.

at C''; $CV = BW$ is the velocity head in this section—a constant since the velocity must be constant. $C'V - B'W = CC' - BB'$. $D'U - C'V =$ head lost by sudden expansion at $C''D''$. $E'T - D'U =$ friction head lost in the pipe $D''E''$, which is less per foot than that in $B''C''$, and hence the angle between UT and XL' is less than that between WV and XL'. The velocity is less in $D''E''$ than in $B''C''$, and hence DU is less than CV. At $E''F''$ is a slight increase in friction head due to the contraction, and the velocity head FS becomes equal to CV, and SR is parallel to WV. At $G''H''$ is a gradual increase of section and no loss of head except the friction head, which, like the velocity, gradually becomes less, and RQ is a curved line. QP is parallel to UT, and $HQ = IP = TE$. At IJ is a gradual contraction and increase of velocity, and PO curves downward as the friction per foot increases and consequently the angle between PO and XL'; and for a similar reason ON curves in the opposite direction. OJ is large since the pipe at J'' is very small, and h varies as V^2 and hence inversely as d^4. Part of $L''L$ is lost at the orifice L'' and part is added to ML in creating additional velocity for the jet at L''.

The line $WVU \ldots M$ must continually fall, since at every point some head is being consumed in friction and cannot be used again. (It is of course not destroyed, but is transformed into heat which cannot be utilized by ordinary practical methods.) The total head between this and the pipe equals pressure head plus velocity head, and either of these can be partly or wholly converted into the other or into friction head or work at any point, as is illustrated in Fig. 22. At any point the total head H at that point can be accounted for, and if all the divisions of H but one be known, this one must be the difference between H and the sum of the others. Thus, at G'', $H = G'G''$; $G'R = h_e$, or WB', plus the head lost in friction up to that point. If it be desired to know this, we have

$$h_f = H - (h_e + RG + GG''),$$

in which RG, the velocity head, $= \frac{V^2}{2g}$; and GG'' is the pressure head and can be measured by a pressure-gauge.

(45) A piezometer is a small pipe so attached to a water-pipe that its bore is normal to the direction of flow in the pipe, and the bore of the pipe is left smooth and continuous except for the orifice itself. Any internal pressure will then force water out of this orifice; and if a vertical glass tube be attached to the small piezometer-pipe, water will rise in this to balance such pressure, or as high as the hydraulic gradient. A piezometer should be placed only where the flow is parallel to the barrel of the pipe. For instance, in Fig. 22 it should not be placed between C'' and D'', where there are eddies.

(46) After a jet issues from an orifice it cannot be under pressure, and hence all the head not lost must be utilized in producing velocity—must be transformed into the energy of a moving stream. It is found that about 2% of the head existing at the orifice as pressure head is lost here, and 98% appears as additional velocity at the contracted vein.

(47) The path of a jet emerging freely from a vertical orifice is a parabola, whose equation is

$$y = \frac{x^2}{4h_v};$$

in which y and x are coordinates of any point in the path, from the orifice as a centre of coordinates. If the orifice is inclined, the formula becomes

$$y = x \tan \theta - \frac{x^2 \sec^2 \theta}{4h_v};$$

in which θ is the angle made with the vertical by the plane of the orifice.

(48) By the above formulas and tables the head lost by the flow in any pressure conduit or any part thereof can be found, when this flow is uniform. In a distribution system, however, water is being removed from the pipes at more or less uniform intervals of distance, and hence the velocity and head lost per foot of length continually diminishes. If we assume that the same amount of water is being discharged at uniform intervals along a length of pipe, the head lost in friction in a given length due to such discharge is practically one third of that which would be lost if the discharge all occurred at the end of this section; and if there is also a flow from the end of the section under consideration the lost head is equal to the above plus

$$f\frac{l}{d}\left(\frac{V^2 + Vv}{2g}\right), \quad \text{or} \quad h_f = f\frac{l}{3d} \cdot \frac{v^2}{2g} + f\frac{l}{d}\left(\frac{V^2 + Vv}{2g}\right);$$

in which l is the length of section considered, d the diameter of the pipe, V the velocity due to the flow at the lower end of this section, and $V + v$ that due to the flow at the upper end.

(49) If, while water is flowing through a pipe, a gate be

closed instantly, the momentum of the moving water will produce an impulse upon the gate and by transmission upon all parts of the pipe. This is called water-hammer. If the water were an inelastic bar, the impact would equal the weight of the entire moving column of water times the square of the velocity. Since it is not, but the forward particles are somewhat compressed, an interval of time is consumed in bringing all the energy of the moving water to bear upon the gate. Moreover, as each elementary section of water presses against the one in front it presses sideways with the same intensity, and the pipe yields to this pressure, and in doing so absorbs part of the energy. But the pipe returns to its normal size, and this and the elasticity of the water cause a reflex pressure-wave back from the valve, which pressure-wave travels back and forth along the length of the pipe until the energy is absorbed in the friction of water and iron molecules among themselves and against each other. Thus a high pressure-wave may pass through a pipe a dozen or more times, becoming less each time, before becoming inappreciable.

From experiments made at Cornell University in 1892–3 upon the effect of the sudden closing of a ½-inch tap on the end of a 1½-inch pipe, the following results were obtained.

	With Air-chamber.		No Air-chamber.	Air-chamber filled with Water.
	1.	2.		
Static pressure	29.5	28.5	27.5	28
Number of distinct blows	8	8	9	9
Maximum pressure	72.5	61.5	69.0	76.0
Minimum "	2.5	10	16	9
Time pulsations continued, in seconds	0.8	0.8	1 +	1.1
Pressure at end of one second	36	34.5	36	36
Ratio of increase of pressure	2.47	2.15	2.56	2.70

During one set of experiments there was an air-chamber a short distance above the tap; during another this was removed; and during the third it was filled with water. The time of closing the tap was about $\frac{1}{100}$ of a second.

Experiments were also conducted in a 2-inch pipe with a 2-inch gate-valve suddenly closed by a lever. From these the curves in the following diagram were obtained. From

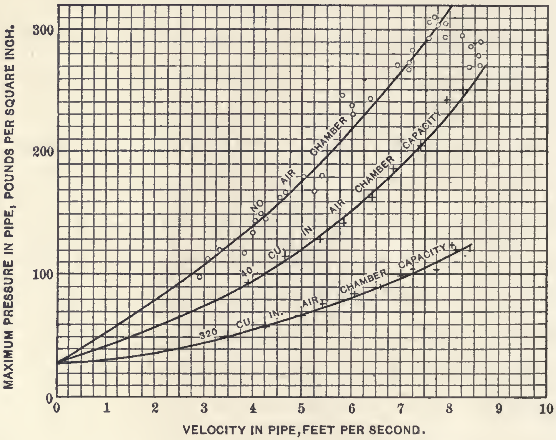


FIG. 23.—WATER-RAM IN 2-INCH PIPE.

this it is seen that a pressure of 300 lbs. was given by water-ram due to a velocity of 8 feet per second when no air-chamber was used.

No satisfactory formula for calculating water-ram has yet been advanced; but it is probable that the force varies as the square of the velocity, directly as the speed of closing the gate, and as some root of the length of moving water-column. It also probably increases in a piping system with the number and nearness of dead-ends. (See Proceedings Am. Soc. C. E., vol. XIV; Mechanics for August, 1884; a paper by Prof. Church in the Journal of the Franklin Inst. for 1890; a description of the above experiments by Prof. R. C. Carpenter in the Transactions of the Am. Soc. of Mechanical Engineers; and Trans. Am. Soc. C. E., vol. XXXIX. pages 1–17.)

ART. 64. FLOW IN HOSE, NOZZLES, ETC.

The loss of head in fire-hose, nozzles, fire-hydrants, and similar appliances follows the same general laws as that in pipes, with a difference in coefficients only. The following tables, from experiments by Freeman, give what are probably the most reliable coefficients obtained for hose and nozzles.

TABLE NO. 52.

JETS FROM SMOOTH NOZZLES. (FREEMAN.)

MAXIMUM VERTICAL AND HORIZONTAL DISTANCES TO WHICH JETS WILL BE THROWN BY THE PRESSURES INDICATED.

Indicated Pressure at Entrance to Nozzle, Pounds per Sq. In.	From ¾-inch Nozzle.				From 1-inch Nozzle.				From 1¼-inch Nozzle.			
	Height in Feet.		Horizontal Distance in Feet.	Discharge in Gallons per Minute.	Height in Feet.		Horizontal Distance in Feet.	Discharge in Gallons per Minute.	Height in Feet.		Horizontal Distance in Feet.	Discharge in Gallons per Minute.
	A	*B*			*A*	*B*			*A*	*B*		
10	20	17	.....	52	21	18	.....	93	22	19	.....	148
20	40	33	72	73	43	35	77	132	44	37	83	209
30	59	48	.....	90	63	51	.....	161	66	53	.....	256
40	78	60	112	104	83	64	133	186	86	67	148	296
50	93	67	....	116	101	73	.....	208	107	77	.....	331
60	104	72	136	127	117	79	167	228	126	85	186	363
70	114	76	.....	137	130	85	.....	246	140	91	.....	392
80	123	79	153	147	140	89	189	263	150	95	213	419
90	129	81	.....	156	147	92	.....	279	157	99	.....	444
100	134	83	167	164	152	96	205	295	161	101	236	468

Column A gives the average height reached by the highest drops in still air; B, the maximum height at which an effective fire-stream is furnished. The horizontal distances are those reached by the furthest drops in still air, at the level of the nozzle; the stream being effective for fire purposes for only about half this distance.

TABLE NO. 53.

FRICTION FACTOR, f, IN $2\frac{1}{2}$-INCH HOSE. (FREEMAN.)

Velocity in feet per second.... $v =$	4	6	10	15	20
For unlined linen hose........ $f =$	0.038	0.038	0.037	0.035	0.034
" rough rubber-lined cotton $f =$	0.030	0.031	0.031	0.030	0.029
" smooth " " $f =$	0.024	0.023	0.022	0.019	0.018
Discharge in gallons per minute $=$	61	92	153	230	306

(50) The diagram, Plate XIII, shows the pressure head corresponding to given quantities of discharge through ordinary, best-quality $2\frac{1}{2}$-inch rubber-lined fire-hose and smooth nozles; this including both head lost in friction in the hose and that necessary to produce the increased velocity in the jet. The column headed "Open Butts" is for hose alone; the others for the length of hose indicated, and nozzle of the size indicated by the column heading. Thus 500 feet of hose with a $1\frac{1}{4}$-inch smooth nozzle attached would require, to discharge 200 gals. a minute, a pressure at the upper end of the hose of a little over 64 lbs. or about 150 feet pressure head; while the same discharge with the nozzle detached would require a pressure head of about 110 feet.

(The velocity of flow in a $2\frac{1}{2}$-inch hose equals the discharge in cubic feet divided by .034.) The head lost in mill-hose is about twice as great as that given in the table; and that lost in unlined hose about 2.3 times as great.

(This diagram is slightly modified from one prepared from Freeman's experiments by Mr. Ernest Stenger and published in *Engineering News*, Feb. 10, 1898.)

(51) The head lost in fire-hydrants will naturally vary with the hydrant used, and these vary materially in design. The only important tests of fire-hydrants known to the author are those made by Mr. Charles L. Newcomb at Holyoke, Mass., in 1897–8 and described in a paper read before the Am. Soc. of Mechanical Engineers in May 1899. He found the friction loss in some hydrants to be four times as great as that in others. The loss when discharging 500 gals. per minute through two hydrant-butts or nozzles, with a pressure of 30 lbs., is shown in the diagram on page 234.

The total loss is seen to be from 0.78 to 2.5 lbs., or 1.8 to 6 feet head. The loss in passing the nozzle is seen to be, in some hydrants, many times that in the barrel; a mechanical defect which could and should be remedied.

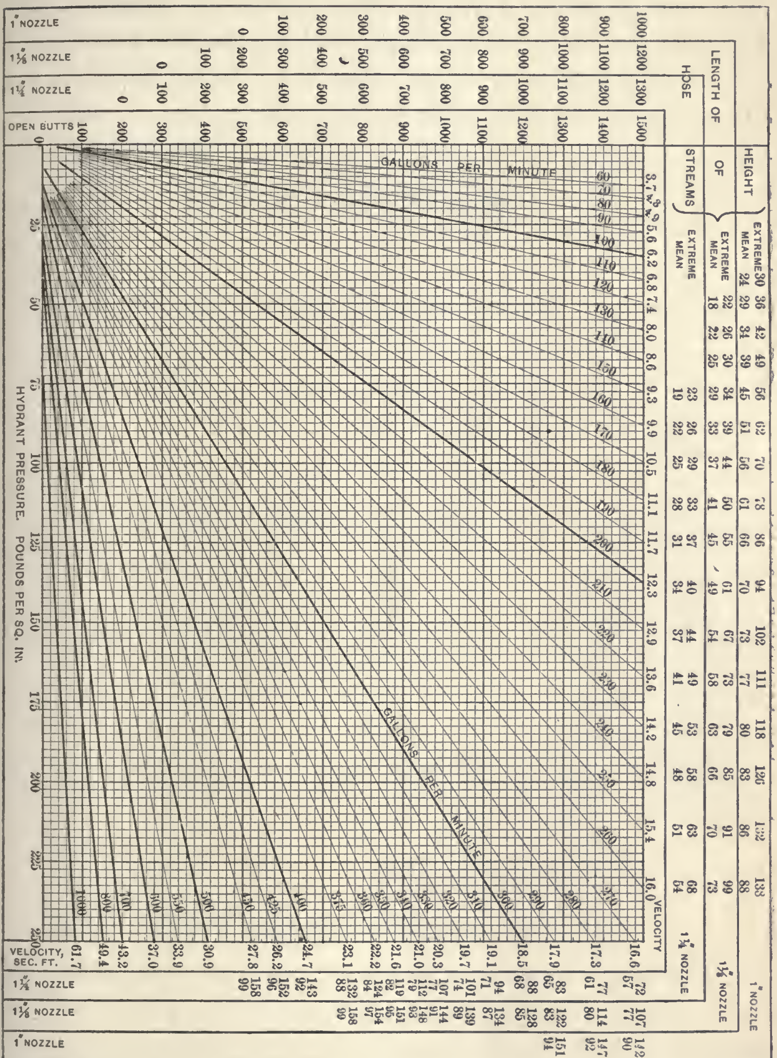


PLATE XIII.—FIRE-STREAM DIAGRAM FOR ORDINARY, BEST QUALITY, 2½-INCH RUBBER-LINED HOSE.

In the fall of 1893 Dexter Brackett found the friction loss in a post-hydrant, 6¾ inches inside diameter, 6-inch rubber valve, 4½-inch nozzles or outlets, to be 4 lbs. for a discharge of 500 gals. per minute, and 16 lbs. for 1000 gals., a large proportion of this loss being at the nozzle-valve.

No.	Name.	Wheight, Pounds.	Inside Volume, Cu. Ft.	No.	Name.	Weight, Pounds.	Inside Volume, Cu. Ft.
1	Beaumont	290	0.69	12	Holyoke, gate, 4 in. barrel	356	1.02
2	Chapman, gate	435	1.39	13	" " 6¼ " "	500	2.16
3	" "	425	1.44	14	" compression	669	1.20
4	" "	734	2.29	15	" gate, 5¼ in	538	0.77
5	" 3-way	577	1.63	16	" gate, 4-way	580	2.24
6	" 4-way	580	2.24	17	" " 6-way		
7	Coffin, gate	467	1.59	18	Ludlow	497	1.87
8	" compression	506	1.44	19	Mathews, 5 inches	750	1.42
9	Corey, 4 inches	418	1.55	20	" 4-way	757	1.50
10	" 5 inches	560	2.24	21	Pratt and Cady	688	2.28
11	Glamorgan, 4 inches	531	1.22				

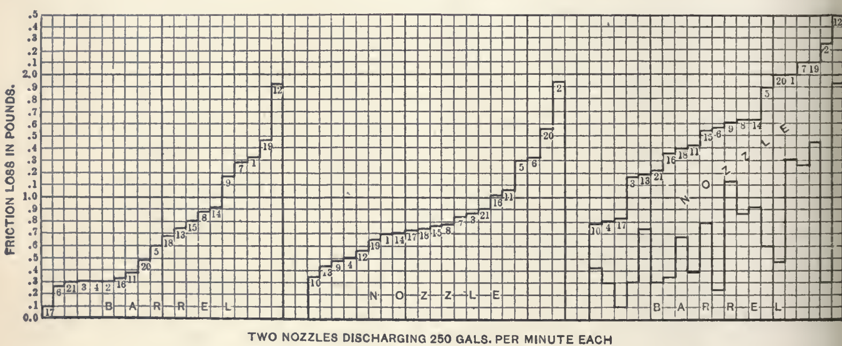


FIG. 24.—FRICTION LOSSES IN FIRE-HYDRANTS.

ART. 65. MEASUREMENT OF WATER.

For many purposes it is desirable to measure the amount of water which is flowing in a natural or artificial channel or pressure conduit. The above formulas enable approximations to be made when the head and the character and dimensions of the conduit are known, and when this is uniform in cross-section. More accurate methods are needed, however, in many instances.

Small quantities of water can be caught in a pail or barrel and measured or weighed, the latter being generally the more accurate. Or an orifice can be used, preferably a circular one. For larger quantities, weirs can be placed in the stream; or for either small or medium amounts in pressure-conduits meters can be used. For large streams the only practicable method used is to obtain the velocity at different points in a cross-section of known area, by the use of floats or current-meters.

The weight of pure water varies with its temperature. The following table, by Hamilton Smith, gives the weight of distilled water.

TABLE No. 54.

WEIGHT OF DISTILLED WATER PER CUBIC FOOT.

Temperature, Degrees.	Weight in Pounds.	Temperature, Degrees.	Weight in Pounds.	Temperature, Degrees.	Weight in Pounds.
32	62.416	85	62.169	155	61.106
34	62.420	90	62.118	160	61.006
36	62.422	95	62.061	165	60.904
38	62.423	100	61.998	170	60.799
39.3	62.424	105	61.933	175	60.694
40	62.423	110	61.865	180	60.586
45	62.419	115	61.794	185	60.476
50	62.408	120	61.719	190	60.365
55	62.390	125	61.638	195	60.251
60	62.366	130	61.555	200	60.135
65	62.336	135	61.473	205	60.015
70	62.300	140	61.386	210	59.893
75	62.261	145	61.296	212	59.843
80	62.217	150	61.203		

Water as ordinarily found in nature weighs somewhat more than this: .05 to .20 lbs. for river-water, and 1.5 to 1.8 lbs. for sea-water. Ice weighs about 57.2 to 57.6 lbs.

(52) In using circular orifices the head is measured from the centre of the orifice to the water-surface above. The back of the orifice plate should extend as a plane for at least three times the diameter of the orifice on all sides of the same, and the area of the reservoir should be so much larger

than that of the orifice that there is practically no velocity of the water therein, or "velocity of approach." The head should be considerable to avoid a vortex, and to render errors in its measurements proportionately small. The equation for discharge through a circular orifice is

$$(53)\quad q = \tfrac{1}{4}c\pi d^2\sqrt{2gH}\left(1 - \frac{1}{128}\frac{d^2}{H^2} - \frac{5d^4}{16384H^4} - \frac{105d^6}{4194304H^6} - \text{etc.}\right);$$

in which c is a coefficient, d is the diameter of the orifice, and H the total head upon the centre of the same. If H is more than $5d$ this equation can be abbreviated, without reducing its practical accuracy, to the form

$$(54)\qquad q = \tfrac{1}{4}c\pi d^2\sqrt{2gH} = 6.2989cd^2\sqrt{H}.$$

The value of c for vertical circular standard orifices differs with the head and diameter. The following values were deduced by Hamilton Smith from the best experiments. (From Merriman's "Hydraulics.")

TABLE No. 55.

COEFFICIENT c FOR VERTICAL CIRCULAR STANDARD ORIFICES.

Head H in Feet.	Diameter of Orifice in Feet.						
	0.02	0.04	0.07	0.1	0.2	0.6	1.0
0.4	....	0.637	0.624	0.618			
0.6	0.655	.630	.618	.613	0.601	0.593	
0.8	.648	.626	.615	.610	.601	.594	.590
1.0	.644	.623	.612	.608	.600	.595	.591
1.5	.637	.618	.608	.605	.600	.596	.593
2.0	.632	.614	.607	.604	.599	.597	.595
2.5	.629	.612	.605	.603	.599	.598	.596
3	.627	.611	.604	.603	.599	.598	.597
4	.623	.609	.603	.602	.599	.597	.596
6	.618	.607	.602	.600	.598	.597	.596
8	.614	.605	.601	.600	.598	.596	.596
10	.611	.603	.599	.598	.597	.596	.595
20	.601	.599	.597	.596	.596	.596	.594
50	.596	.595	.594	.594	.594	.594	.593
100	.593	.592	.592	.592	.592	.592	.592

The limit of error in measuring by circular orifices can by care be kept within 1%. Coefficients for square and rectangular orifices are not as generally reliable as those for circular, and are not given. Values for these may be found in Hamilton Smith, Jr.'s "Hydraulics" and Merriman's "Hydraulics."

(55) A weir is an upright obstruction, similar to a dam of stone or timber, over which water flows. In hydraulics, by a standard weir is meant one in which the inner face is a vertical plane, and the edge of the weir, or its crest, is sharp and similar to that of a standard orifice. Weirs are generally made rectangular, but are triangular or trapezoidal in some cases. The crest of a rectangular weir may extend to the side of the flume or canal conducting the water to it, or it may be a rectangular notch cut in the weir-plate, the vertical edges being bevelled similar to the horizontal one. In the latter it is said to have end contractions, in the former the contractions are said to be suppressed. If the contractions are not suppressed, the weir-plate should extend as a plane on each side of the weir a distance at least three or four times the depth of water on the crest.

It is very difficult to measure the depth of water over the crest, and instead is taken the height, H, above the crest of the surface of the water before it begins to curve toward the weir.

The formula for discharge is

$$(56)\quad q = c\tfrac{2}{3}\sqrt{2g}\,l(H + nh)^{\frac{3}{2}} = 5.343cl(H + nH)^{\frac{3}{2}};$$

in which q is discharge in cubic feet per second;

c is a coefficient of discharge;

l is the length of crest of the weir;

n is a coefficient of velocity of approach;

h is the head of velocity of approach $= \dfrac{V^2}{2g}$.

Table No. 56 gives values for c for weirs with end-contractions and for those in which these are suppressed; and for waste-weirs or weir-dams having flat crests.

Francis, from his classic experiments on weirs at Lowell in 1854, deduced the expression

$$(57) \qquad q = 3.33 l H^{\frac{3}{2}}$$

for weirs with end-contractions, and

$$(58) \qquad q = 3.33(l - 0.2H)H^{\frac{3}{2}}$$

for weirs with both contractions suppressed; or, allowing for velocity of approach,

$$(59) \qquad q = 3.33 l\left[(H + h)^{\frac{3}{2}} - h^{\frac{3}{2}}\right] \text{ and}$$

$$(60) \qquad q = 3.33(l - 0.2H)\left[(H + h)^{\frac{3}{2}} - h^{\frac{3}{2}}\right].$$

Most of these experiments were made on weirs 10 feet long (all were quite large ones) and with heads ranging from 0.4 to 1.6 feet; and his formulas are best adapted to such proportions.

(61) For a trapezoidal weir the coefficient may be made constant by a proper sloping of the sides. If this slope be made 1 : 4, the equation $q = 3.36 l H^{\frac{3}{2}}$ is found to be fairly exact.

Many designs of meters are on the market which will measure the flow through them with an error of less than 3%. In most of these a revolving or oscillating piston or diaphragm moves a registering train of wheels, the number of revolutions being recorded, not as such, but as the number of gallons or cubic feet which trial has shown must pass through the meter to cause this number of revolutions.

(62) The Venturi meter is particularly adapted to large pipes and volumes of flow. It acts upon the principle illus-

TABLE No. 56.

COEFFICIENTS FOR WEIRS.

CONTRACTED STANDARD WEIRS.

Effective Head in Feet.	Length of Weir in Feet.								
	0.66	1	2	3	4	5	7	10	19
0.1	0.632	0.639	0.646	0.652	....	0.653	....	.655	.656
0.15	.619	.625	.634	.638	....	.640	....	.641	.642
0.2	.611	.618	.626	.630	....	.631	....	.633	.634
0.25	.605	.612	.621	.624	....	.626	....	.628	.629
0.3	.601	.608	.616	.619	....	.621	....	.624	.625
0.4	.595	.601	.609	.613	....	.615	....	.618	.620
0.5	.590	.596	.605	.608	....	.611	....	.615	.617
0.6	.587	.593	.601	.605	....	.608	....	.613	.615
0.7	....	.590	.598	.603	....	.606	....	.612	.614
0.8	....	....	.595	.600	....	.604	....	.611	.613
0.9	....	....	.592	.598	....	.603	....	.609	.612
1.0	....	....	.590	.595	....	.601	....	.608	.611
1.2	....	....	.585	.591	....	.597	....	.605	.610
1.4	....	....	.580	.587	....	.594	....	.602	.609
1.6	....	....	....	.582	....	.591	....	.600	.607

STANDARD WEIRS, CONTRACTIONS SUPPRESSED.

0.1	....	....	....	....	....	0.659	0.658	0.658	0.657
0.15	....	....	0.652	0.649	0.647	.645	.645	.644	.643
0.2	....	....	.645	.642	.641	.638	.637	.637	.635
0.25	....	....	.641	.638	.636	.634	.633	.632	.630
0.3	....	....	.639	.636	.633	.631	.629	.628	.626
0.4	....	....	.636	.633	.630	.628	.625	.623	.621
0.5	....	....	.637	.633	.630	.627	.624	.621	.619
0.6	....	....	.638	.634	.630	.627	.623	.620	.618
0.7	....	....	.640	.635	.631	.628	.624	.620	.618
0.8	....	....	.643	.637	.633	.629	.625	.621	.618
0.9	....	....	.645	.639	.635	.631	.627	.622	.619
1.0	....	....	.648	.641	.637	.633	.628	.624	.619
1.2	....	....	....	.646	.641	.636	.632	.626	.620
1.4	....	....	....	....	.644	.640	.634	.629	.622
1.6	....	....	....	....	.647	.642	.637	.631	.623

WASTE-WEIRS AND DAMS, CONTRACTIONS SUPPRESSED.

(From Francis' formula, $q = 3.01lH^{1.53}$.)

0.1	....	....	....	....	....	0.556	0.555	0.555	0.554
0.15	....	....	0.556	0.554	0.552	.550	.550	.549	.549
0.2	....	....	.556	.553	.552	.550	.550	.549	.549
0.25	....	....	.556	.553	.551	.550	.549	.548	.546
0.3	....	....	.557	.554	.552	.550	.548	.547	.546
0.4	....	....	.559	.557	.554	.552	.550	.548	.546
0.5	....	....	.564	.560	.558	.555	.552	.550	.548
0.6	....	....	.568	.564	.561	.558	.555	.552	.550
0.7	....	....	.572	.568	.564	.562	.558	.554	.553
0.8	....	....	.578	.573	.569	.566	.562	.559	.556
0.9	....	....	.581	.576	.572	.569	.565	.561	.558
1.0	....	....	.586	.579	.576	.572	.568	.564	.560
1.2	....	....	....	.587	.583	.578	.574	.569	.564
1.4	....	....	....	....	.588	.584	.579	.574	.568
1.6	....	....	....	....	.593	.589	.584	.578	.571

trated in Fig. 22, page 226, at J, except that the pipes at both ends are the same size. Assuming that the loss by friction in the meter is so small that it may be disregarded, we have

$$h_P + h_v = h_P' + h_v', \quad \text{or} \quad h_P - h_P' = h_v' - h_v;$$

in which h and h' are the head above the meter and at the throat J respectively. Now h_P and h_P' can be measured by pressure-gauges or piezometers; also $v : v' = d'^2 : d^2$, and $h_v : h_v' = v^2 : v'^2$; hence

$$h_v : h_v' = d'^4 : d^4, \quad \text{or} \quad h_v' = \frac{h_v d^4}{d'^4}.$$

Also $h_v : h_v' - h_v = d'^4 : d^4 - d'^4$; hence

$$h_v = \frac{d'^4(h_v' - h_v)}{d^4 - d'^4}.$$

Or, substituting,

$$h_v = \frac{d'^4(h_P - h_P')}{d^4 - d'^4};$$

in which d and d' are the diameters of pipe and throat respectively, and can be measured.

$$q = ca\sqrt{2gh_v} = c\frac{ad'^2}{\sqrt{d^4 - d'^4}}\sqrt{2g(h_P - h_P')};$$

in which c is a coefficient which must generally be ascertained by experiment for each meter, but is almost 1.

(63) For determining the velocity of large streams floats may be used, traversing as long a stretch of the stream as can be found of practically uniform cross-section; the time it takes floats in different positions in the cross-section to traverse this distance being carefully noted. Floats should be used which expose as little of their surface as possible to the wind, and a day should be selected when there is little air stirring. A more accurate method is to use a current-meter, which consists essentially of a wheel with cups upon its circumference, and which is revolved by the current, the

velocity of revolution depending upon that of the current. Each meter is rated to find coefficients by which to reduce the rate of revolution to rate of flow. Readings are taken at a number of points in a cross-section of known area, the greater the number of these points the better.

QUERIES.

18. In Fig. 22, if the pipe have the diameters given, that at J'' being 2 inches, and if $B''C''$, $C''F''$, $F''G''$, and $K''L''$ be each 200 feet long, $G''H''$ and $I''J''$ each 10 feet, and $H''I''$ and $J''K''$ each 5 feet, the whole being new cast-iron pipe, what would be the discharge per second through a 1-inch circular orifice at L'' if $L''L'$ is 100 feet?

19. A valley has the shape of an isosceles triangle, 3 miles wide on the base and 5 miles long, with soil of clayey loam. If a dam be placed at the apex, what should be the length of the waste-weir or spillway to give a maximum depth of flow over it of 3 feet? How much higher than the spillway should the top of the earth embankment be? Use the rainfall records for eastern New York.

20. A Holyoke gate fire-hydrant, 4-inch barrel, two nozzles, is connected by 10 feet of 4-inch pipe to a 6-inch main 500 feet long, which in turn is fed by a 12-inch main 1000 feet long. What elevation must the reservoir have above the hydrant-nozzles to enable these, both discharging at once, to give a combined discharge of 500 gallons per minute, no other water being drawn from the pipes at the same time?

CHAPTER XII.

DAMS AND EMBANKMENTS.

ART. 66. MATERIALS FOR CONSTRUCTION.

DAMS have been built of earth, loose rock ("rock-fill"), timber, iron, and steel, and of concrete, stone, and brick masonry; and cores of timber, iron, and masonry have been used in many dams of the first and second classes. Of these, stone masonry is the most substantial and tightest, concrete masonry next, timber probably comes next (within the life-time of the timber), and loose rock and earth last. Iron and steel have not been used for any but movable dams, except as a facing or core. A poorly designed or constructed masonry dam may of course be weaker or more porous than a timber or earthen one; but the above order will hold generally for properly designed and well-built dams.

Timber dams are most applicable for use as weir-dams—that is, those over whose crests water flows—because of the tendency of the timber to rot if alternately wet and dry. Earth dams, on the other hand, will surely fail if any water whatever flows over them. Rock fill dams are not intended to serve as weirs, and, although an occasional small flow over their crests may not prove fatal, should not be designed to serve as such.

Masonry is the only material adaptable to weir-dams of permanent construction. It is also used for high-crest dams which are not intended to pass water, when absolute stability or tightness is desired, or when the height of the dam exceeds 60 to 75 feet (although earth dams have been constructed much higher; as the Honey Lake Valley, Cal., dam, 118 feet high, with a puddle core). Earth dams are practically adapted to locations where there is not a sufficiently firm foundation to support a high masonry dam, or where the cost of this would be excessive; as in the case of the Honey Lake Valley dam, which was so inaccessible that haulage brought the cost of cement up to $8.25 per barrel. If the distance to bed-rock or excessive cost render the use of masonry inadvisable for a weir-dam, a timber dam is probably the only alternative; but if the work is extensive, the dam more than 40 or 50 feet high, or a permanent construction desired, masonry must be used and carried down to bed-rock at whatever depth.

It may sometimes be desirable, for financial reasons, to construct a temporary dam of timber a little above the best location for a masonry dam, and build a masonry dam at this location some time within twenty years, for which length of time a well-constructed timber dam should be perfectly sound.

The most careful consideration of all the conditions should be taken by the engineer in deciding upon the character of a proposed dam.

ART. 67. MASONRY DAMS. GENERAL CONSTRUCTION.

Masonry is practically not flexible, and hence a perfectly inflexible foundation is necessary if there are to be no unknown and unprovided-for strains in the structure which may cause its rupture. Comparatively small and low dams may be set upon heavy timber foundations resting upon piles or a solid hardpan or gravel foundation, but only the greatest care to prevent undermining of the foundation and a large factor of safety in the masonry can render such a dam secure. Close sheet-piling should be carried entirely across the upper end of the foundation, placed in a trench which has been dug down to and a foot or two into hardpan or clay, and puddle well rammed into the trench on each side of the sheeting. The sheeting should be carried up two or three feet higher than the foundation and the space between it and the masonry filled with concrete or puddle. (See Fig. 25.) The founda-

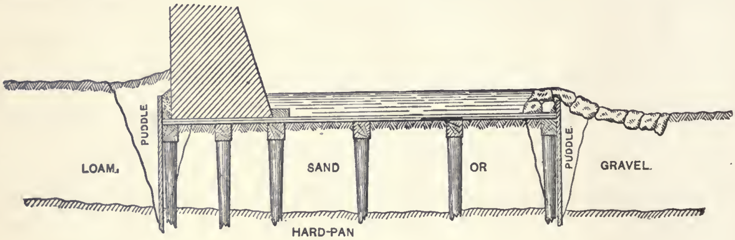


FIG. 25.—DAM ON TIMBER FOUNDATION.

tion should be carried down-stream from the dam for a distance at least equal to the height of the dam, to act as an apron to prevent the falling water from undermining the foundation. At the end of the apron should be placed sheet-piling similar to that already described. In the case of a weir-dam or spillway a low dam is sometimes placed at the end of the apron to form a water-cushion at the face of the dam.

No masonry dam more than 10 or 12 feet high should be placed upon a timber foundation, but all such should be carried down to rock. All loose or decomposed rock on the foundation should be removed, and if the rock have a smooth top surface a shallow trench should be excavated into it the entire length of the dam, or the bed should be cut in steps, to prevent leakage and the sliding of the dam on its bed. Every square foot of the foundation should be thoroughly cleaned and washed, and covered with a thick bed of mortar just before the masonry is laid upon it; and the spaces between the dam and the sides of the excavation—if any was made—should be cleaned out and filled with concrete.

The ends of the dam should be carried to bed-rock, if possible, and treated as was the bottom. If rock does not extend up to the level of the crest of the dam at its ends, these should be carried some distance into the banks to prevent leakage around them, or tight masonry river-walls should be carried for some distance up-stream from the dam. The banks just below the dam, if of earth, must be protected from wash by similar walls carried for a short distance down-stream from the dam.

If the dam forms a reservoir, or it is desired for any reason to draw off the water at any elevation lower than the crest, pipes or other conduits must generally be carried through the dam. These should never be given uninterrupted smooth outside surfaces, but should be provided with flanges or other projections around their circumferences to prevent water from following along them through the wall. Probably the best plan is to place two or three wide flanges around the pipe near the upper face of the wall and bed these thoroughly in concrete, having first cleaned them and the pipe and given both a coat of rich Portland cement-mortar. Other flanges should be placed at intervals of 10 to 20 feet if the dam exceed this thickness.

In some cases a tunnel is carried through the dam, open at the lower end, and terminating at the upper in a watertight well rising above the water surface, in which tunnel the conduits are laid. This is an excellent plan for large dams and conduits.

In building a stone dam, care should be taken to break courses at every joint; the masonry being the most uncoursed rubble, except the faces, which should be broken ashlar. Every stone and its bed must be perfectly clean and have a damp surface when it is laid, and all spaces must be absolutely filled with mortar or fine concrete. If a tight dam is to be built these precautions must be conscientiously observed, although it may be necessary to discharge half the masons during the first week to effect this. The dam should, as far as possible, be so carried up that the top of the finished masonry is at all times approximately horizontal. All stone should be set by derrick; and no dressing of stone done on the wall, which is likely to disturb or jar stones already set. A dam built in this way may be made perfectly tight under any practicable head.

Concrete has been used for a number of dams, and still more have had a base of concrete used to fill up the irregularities in a rock foundation. A concrete dam is considered to be more porous than a stone dam. To reduce the porosity to a minimum the amount of cement used should be 10% more than enough to fill the voids in the broken stone or gravel used. The mixing of the dry sand and cement and that of the mortar and stone must be thorough; no more water must be used than will bring moisture to the surface on ramming; all surfaces of concrete in place must be cleaned of all dirt, loose stone, and porous concrete, and thoroughly dampened and plastered with mortar before more concrete is placed thereon; and the concrete must be thoroughly but lightly rammed. All surfaces of concrete in place should be rough or irregular; and the top surfaces of the finished concrete should be kept approximately level. Several large dams have been successfully built of concrete; as that at Butte City, Mont., 120 feet high, and the San Mateo dam, 170 feet high.

Brick has been used in the construction of a few dams, as the Belubula dam, New South Wales, in which the top 36 feet 9 inches is of brick, upon a concrete base having a maximum height of 23 feet. Brick is deficient in weight, both per unit mass and per cubic foot, and is not generally so durable as stone or concrete.

ART. 68. MASONRY DAMS: DESIGNING.

The cross-section of a masonry dam is generally designed on the assumption that the dam is a rigid, monolithic mass. That this may be the case was demonstrated by a Minneapolis dam, which, in February 1899, was revolved about its toe as an unbroken mass through an angle of about 25°, the cause being the pressure exerted by ice in the pond above. This dam was 18 feet high, 12 feet through the bottom, and 535 feet long. It is doubtful if a high dam would remain unbroken under such conditions, but it will be seen to be an assumption on the safe side to base a design on this condition.

A dam may yield in one of three ways: it may be overturned as just described, or the part above any horizontal plane may so revolve; it may slide as a whole on the foundation, or any part may slide on the part below it; or it may yield by crushing of the masonry or of the foundation. The forces acting are: the weight of the masonry; the pressure of the water; the dynamic effect of a river-current, of waves, and of logs, ice, or other floating solid matters; and the pressure of the wind. The last can act upon but a small part of the rear of the dam, and when acting upon the face of a full dam only increases its stability. If the dam be empty, wind-pressure upon its face may have some effect. The wind will not probably exceed 50 lbs. per square foot of vertical surface. If we call it 62.5 lbs. we find the pressure to be $\dfrac{62.5h}{62.5\frac{h^2}{2}} = \dfrac{2}{h}$ of the pressure due to water on the same face, h being the height of the dam. The force of the blow delivered by floating logs or ice equals $\dfrac{WV}{gt}$. It is not probable that any such matter will strike the dam when there is a depth over the crest of more than 3 feet; at which depth the velocity of flow over the crest will be about 8 feet per second, and the energy of the moving logs or ice will be about W, and the impulse about $\frac{W}{4}$. This is somewhat increased, in case a floating body strikes the dam, by the friction of the water, to an amount probably not exceeding 25% of the weight of the body. A tree or log would not, therefore, probably strike a dam-crest with an impulse greater than 4500 lbs.; and probably a height of 2 feet on the crest and 2250 lbs. impulse would be high enough for most cases.

The greatest impulse due to waves will probably not exceed 3000 lbs. per square foot, since this was the maximum found for ocean waves by Stevenson at Bell Rock Lighthouse, and 6100 the maximum of all his observations.

The pressure due to the river-current will seldom be appreciable in amount, except at the very crest, and here will probably never exceed 100 lbs. per square foot; it can hence be neglected in practical designs.

The pressure due to water will be 62.5 times the area of the wetted face, times the distance from the centre of gravity of such face to the water-surface. In calculating it is convenient to assume a section of the wall one foot long as a unit. The horizontal pressure on this will be $\frac{62.5d^2}{2}$, in which d is the depth of water.

The weight of masonry for any given case can be ascertained by actual test. By Table No. 60, page 260, we see that the concrete of the Sweetwater Dam weighed 164 lbs. per cubic foot; and that of those for which the data are given the weight varies between 134 and 160 lbs. Baker in "Masonry Construction" gives the following weights for masonry:

TABLE No. 57.

WEIGHT OF MASONRY.

Kind of Masonry.	Weight in Pounds per Cu. Ft.
Brickwork, pressed brick, thin joints	145*
" " ordinary quality	125
" " soft brick, thick joints	100
Concrete, best	160*
" porous	130
Granite or limestone, well dressed throughout	165*
" " " rubble, well dressed with mortar	155*
" " " " roughly dressed with mortar	150
" " " " well dressed, dry	140
" " " " roughly dressed, dry	125
Mortar, dried	100

Since dam-masonry should be the best of its kind, with all joints well filled, the weights after which stars are placed may be used: and 160 lbs. will be used in calculations in this work.

In a dam of rectangular section, Fig. 26, if W is the weight of the masonry and P the horizontal pressure due to a depth d of water, the point of application of P is $\frac{d}{3}$ above the bottom, and that of W is the centre of the base, C.

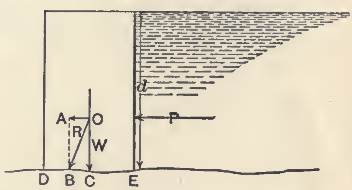


FIG. 26.—RECTANGULAR DAM.

The pressure P, $= \frac{62.5d^2}{2}$, tends to cause the dam to slide, and this tendency is resisted by the friction between the dam and the base, and the tendency at any horizontal joint is resisted by the friction in that joint. This friction is equal to fW, in which f is a friction coefficient, values for which are given in Table No. 58.

If we assume a coefficient of .65, the friction resistance would be $160 \times 0.65 = 104$ lbs. per cubic foot of masonry. In the above figure, if d be 10 feet and the width of the dam be 6 feet, the pressure will be $\frac{62.5 \times 100}{2} = 3125$ lbs.; and the friction resistance will be $10 \times 6 \times 104 = 6240$, and the factor of safety against sliding will be 2. As a matter of fact, if the dam be constructed as described above, there will be no possibility of sliding except as a whole upon a timber foundation; and in such a dam a timber should be bolted to the foundation in front of the masonry to prevent sliding. (See Fig. 25, page 244.)

TABLE No. 58.

COEFFICIENT OF FRICTION FOR MASONRY.

Kind of Masonry.	Coefficient
Soft brick	0.75
Hard brick	0.70
Granite, point-dressed	0.70
Hard limstone	0.65
Concrete blocks	0.65
Granite, well-dressed	0.60
Limestone on oak	0.40

In the case of a trapezoidal dam, Fig. 27, with an inclined

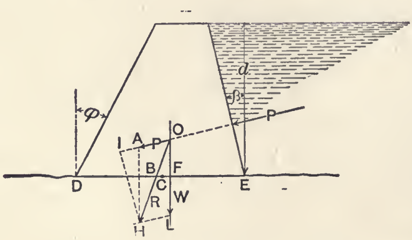


FIG. 27.—TRAPEZOIDAL DAM.

back, the pressure $P = \frac{62.5d^2}{2 \cos \beta}$ (see Art. 61, (6)); but the horizontal pressure is, as before, $\frac{62.5d^2}{2}$ or $31.25d^2$, and the friction $= fW$.

The tendency to crushing is due to a combination of all the pressures named, and is resisted by the strength of the masonry. This resistance to crushing is approximately as follows in different classes of stone when tested in cubes.

TABLE NO. 59.

CRUSHING STRENGTH OF CUBES OF STONE.

(Baker's "Masonry Construction.")

Kinds of Stone.	Ultimate Crushing Strength.			
	Pounds per Sq. Inch.		Tons per Sq. Foot.	
	Min.	Max.	Min.	Max.
Trap-rocks of N. J.	20,000	24,000	1440	1730
Granite	12,000	21,000	860	1510
Marble	8,000	20,000	580	1440
Limestone	7,000	20,000	500	1440
Sandstone	5,000	15,000	360	1080

The strength of masonry, however, is found to depend to only a minor degree upon that of the stone, but to be largely affected by the character of mortar and of bond. The limits of safe pressure have been assumed as follows by Baker:

For concrete	5 to 15	tons per sq. ft.
" rubble	10 " 15	" " " "
" squared stone	15 " 20	" " " "
" limestone ashlar	20 " 25	" " " "
" granite ashlar	30	" " " "

"The maximum pressure on the granite masonry of the towers of the Brooklyn Bridge is about $28\frac{1}{2}$ tons per square foot (about 400 lbs. pressure per square inch). The maximum pressure on the limestone masonry of this bridge is about 10 tons per square foot (125 lbs. per square inch)." "The limestone masonry in the towers of the Niagara Suspension Bridge failed under 36 tons per square foot and were taken down—however, the masonry was not well executed." (Baker.)

Comparing these limits with Table No. 57, we see that rectangular walls give the maximum allowable pressure at the base when having the following heights:

Concrete	60 to 200 feet
Rubble	130 " 190 "
Squared stone	180 " 240 "
Limestone ashlar	240 " 300 "
Granite ashlar	360 "

The effect of water-pressure, however, is to both increase the total pressure and to make the pressure near the toe greater than the mean pressure; and the above heights are not permissible for rectangular *dams*.

In Fig. 26, page 249, let the length OC represent the weight of a unit section of the dam, and that of OA the water-pressure; then OB, or R, represents the resultant of these two forces in both intensity and direction, the resultant cutting the base at a distance BC from the centre of the base. From the parallelogram of forces we see that $W : P = \frac{d}{3} : BC$, or $BC = \frac{Pd}{3W}$.

It is demonstrable that if the resultant R cuts the base at a distance from D less than one third DE, the masonry at E is in tension; or, if it yields to tension, the pressure is distributed over less than the whole base; and when R passes through D the whole pressure is concentrated at that point. When $DB = \frac{1}{3}DE$ there is neither tension nor compression at E, and the pressure at D is twice the mean, or $\frac{2R}{DE}$. If R cuts the base at C ($P = 0$), the pressure is distributed evenly over the base. In the last case the intensity of pressure cannot be sufficient to overturn the dam, or the factor of safety against overturning is infinity. When R passes through D the dam is on the point of overturning, and the factor of safety is 1. The factor for any position of R is

$$F = \frac{CD}{CB}$$

which equals 3 when R is just at the limit of the middle third of the base.

In Fig. 27, R can be calculated as follows: Draw HI perpendicular to OA produced; then $AI = W \sin \beta$, $OI = P + W \sin \beta$, $HI = W \cos \beta$, and

$$OH = R = \sqrt{(P + W \sin \beta)^2 + (W \cos \beta)^2} = \sqrt{P^2 + 2PW \sin \beta + W^2}.$$

When there is no water-pressure W alone acts, and this must fall within the middle thir l of the base, or FE must not be less than $\frac{1}{3}DE$.

$$FE = \tfrac{2}{3}H \tan \beta + \tfrac{1}{3}H \tan \phi + \tfrac{1}{2}T + \frac{TH^2 (\tan \beta - \tan \phi)}{12TH + 6H^2 (\tan \beta + \tan \phi)}$$

and

$$FC = \tfrac{1}{6}H \tan \phi - \tfrac{1}{6}H \tan \beta + \frac{TH^2 (\tan \phi - \tan \beta)}{12TH + 6H^2 (\tan \phi + \tan \beta)};$$

in which T is the width of the crest of the dam, H is the height of the dam, and C is the centre of the base. The factor of safety against overturning is $\dfrac{CE}{FC}$ when the reservoir is empty, and $\dfrac{CE}{BC}$ when it is full.

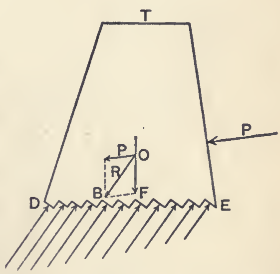


FIG. 28.—MAXIMUM PRESSURE ON A HORIZONTAL JOINT.

The maximum pressure will be that due to the total force acting, or the resultant R, and in a plane at right angles to this. If we conceive the resultant as resisted by a number of parallel forces, as in Fig. 28, we find the total pressure to be distributed over an area $DE \cos FOB$, and the average pressure per unit area is

$$\frac{R}{DE \cos FOB} = \frac{\sqrt{P^2 + 2PW \sin \beta + W^2}}{DE \cos FOB}.$$

As an illustration, let $H = d = 40$ feet, $DE = 30$ feet,

$T = 6$ feet, $\beta = 10°$; P then equals $31.25 \dfrac{1600}{\cos 10°} = 5077$ lbs.; $W = 160\left(\dfrac{6+30}{2}\right)40 = 115{,}200$ lbs.; $R = 116{,}189$ lbs.; $FOB = 2°\ 28'$; $\dfrac{R}{DE \cos FOB} = \dfrac{116{,}189}{29.9722} = 3877$ lbs., or 1.94 tons per square foot.

$$FE = \tfrac{2}{3}(7.053) + \tfrac{1}{3}(16.947) + \tfrac{1}{2}\left(6 - \frac{240(9.894)}{2880+240(24)}\right) = 16.076 \text{ feet}$$

and $$FC = 1.924;$$

$$OF = \frac{d - 2FE \tan \beta}{3}; \quad \text{and} \quad BF = OF \tan FOB = 0.47.$$

In this dam the resultant therefore cuts the base to the right of C, and the tendency of the dam is rather to revolve backward than forward, even with the full water-pressure on.

If $T = 0$, $DE = 20$ feet, $H = 40$ feet, $\beta = 10°$, then P, as before, $= 5077$ lbs.; $W = 160\left(\dfrac{20 \times 40}{2}\right) = 64{,}000$ lbs.; $R = 65{,}074$; $FOB = 4°\ 24'$; the maximum pressure $= 3264$ lbs. $= 1.63$ tons per square foot; $FE = 9.8$ feet; $FC = 0.2$ feet; $BF = 0.892$ feet; whence $BC = 0.692$ feet, and the factor of safety against overturning is $\dfrac{10}{.692} = 14\tfrac{1}{2}$.

If there were no wind and no other pressure except that of motionless water, a dam might be constructed with a triangular cross-section. But these conditions never exist in practice. Waves or floating logs may give a pressure of 3000 lbs. per square foot, and the friction at a joint one foot from the top should therefore equal this. If the dam is not a weir-dam this may be effected by carrying the top of the masonry above the highest waves. This may be (see page 168) 4 feet above the water-surface. If the top of the dam be 10 feet in width, it should extend $\dfrac{3000}{.70 \times 10 \times 160} = 2.6$ feet above the maximum wave-height, or 6 feet 8 inches above the water-surface. Above the water-surface there may be exerted by waves a pressure of 4×3000 lbs. per lineal foot, to resist which there must be a cross-section area above this point of $\frac{12,000}{.70 \times 160} = 107$ square feet, which can best be obtained by making the dam say 16 feet wide on top and 7 feet above the water-surface. This will be necessary only when there is a large sweep of water above the dam. For ordinary cases the pressure will not exceed one fifth of this amount, and 20 square feet above the water-surface will be ample, or say 5 feet wide and 4 feet high; which reduction is also made permissible by the fact that adhesion of mortar will offer an appreciable assistance to friction, and that the masonry is uncoursed. If a dam is to act as a weir, d (which should for this and all other cases be taken at the highest water possible to occur) will be greater than H, and the proper values for these should be used in the equations. $d - H$ will be the depth of flow over the crest caused by the maximum rate of run-off (see Art. 50). A weir-dam should have an area of section above the first joint below the crest sufficient to resist the pressure due to $d - H$, and an impact from logs, etc., of say 2500 lbs. per lineal foot. To insure this the crest is generally made quite broad and the crest-stone quite heavy. In addition it is well to clamp the crest-stones, or cap-stones, to each other and to the dam beneath (see Plate XIV), and also to incline them toward the back of the dam, to cause a glancing rather than a straight blow to be struck by floating objects.

If a high weir-dam be made trapezoidal in section, the falling water will cause considerable shock to the foundation and ebullition of the water, and tend to undermine the dam or to loosen the bonds of its masonry by jarring. To prevent this, when the dam is on soft rock or is more than 15 or 20

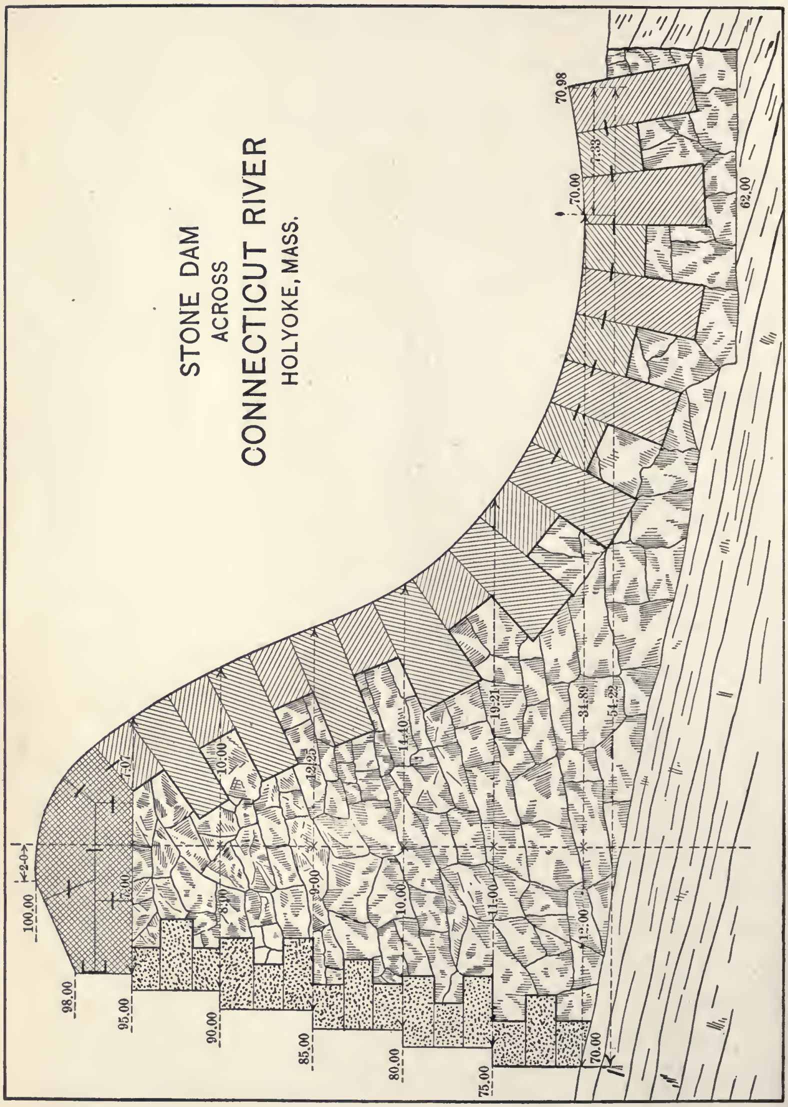


PLATE XIV.—WEIR-DAM AT HOLYOKE, MASS.

feet high it is advisable to curve the toe concave upward, as in the Holyoke Dam, and also curve the front of the crest, the face-outline thus formed causing this to be called an ogee dam. For smaller dams the force of the water may be broken by stepping the face at each course of masonry, the cost of cutting the face-stone to an ogee curve being too considerable a proportion of the whole cost in a small dam, while the danger from the falling water is very much less in low than in high dams. For the same reason low dams not intended to act as weirs are generally made trapezoidal in section, the top width and the height being decided upon, and the bottom width being made sufficient to insure the stability of the dam.

For high dams, of which the cost will be very great, it is desirable to use no more masonry than is necessary to insure their absolute stability. No formula which will enable the form of such a dam to be calculated directly has ever been found; but the calculation can be made for each of a series of horizontal joints, beginning at the top and taken as close together as desired, and a curve passed through the points thus found. The conditions are: (1) The resultant pressure both when the dam is full and when empty must be inside the middle third of the joint and as near its limit as possible. (2) The length of joint must be sufficient, relative to the area of vertical section above it, to prevent crushing (it being remembered that the maximum pressure is twice the mean when (1) has been observed). (3) The weight above any joint must be sufficient to prevent sliding. (For a thorough theoretical discussion of economic profiles, see Wegmann's "Design and Construction of Masonry Dams.") Wegmann's practical profile, closely following the theoretic, is shown in Plate XV; in which is also shown, in dotted lines, the Austin Dam across the Colorado River, and the portion of the new Croton Dam serving as a weir.

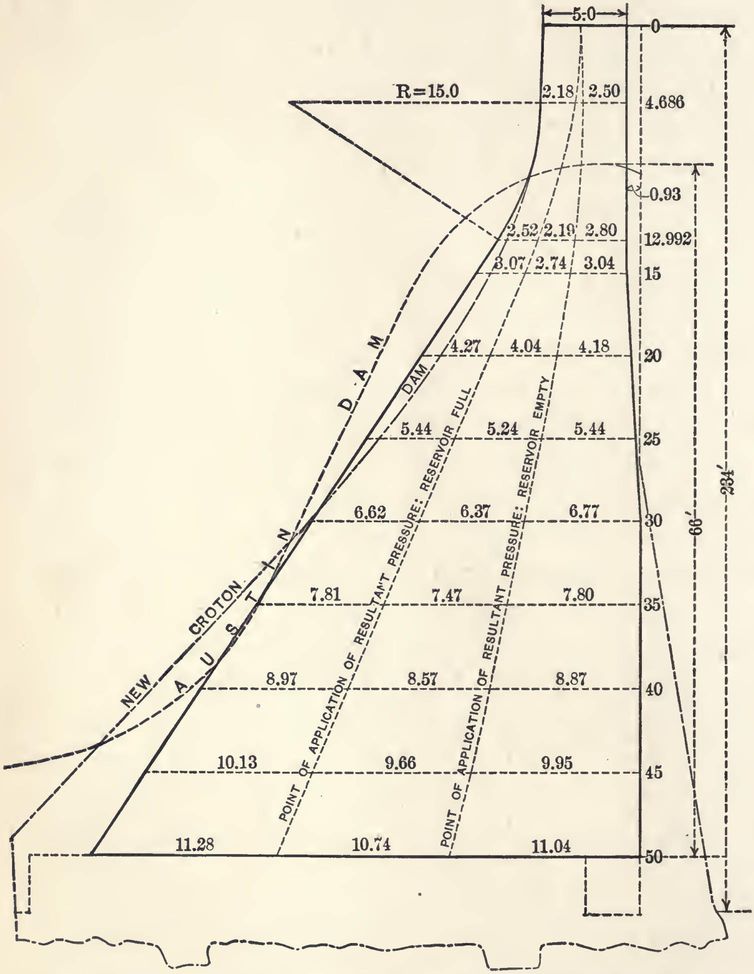


PLATE XV.—PRACTICAL ECONOMIC PROFILE OF HIGH MASONRY DAM.

To provide the greatest amount of storage possible a dam must be raised to a level with the highest permissible water-line; but if a large amount of water passes over the dam, the water in the reservoir will rise above this line; and the crest must therefore be lowered to an elevation as much below the high-water line as will be the greatest depth of flow over the weir. Both these conditions are sometimes met by placing on top of the weir "flash-boards" which practically raise the crest of the weir temporarily. The use of these is not recommended, however, unless they be made of such a strength, or so designed, that they will break or open before the water rises to the high-water line.

Dams may be either straight, curved, or polygonal in plan; each elementary vertical section being designed to resist by itself all forces tending to destroy it. If the dam be curved in plan, however, the masonry may act as an arch, the ends abutting against solid bed-rock. It is thought by some that the arch effect cannot come into play unless the section of the dam is too light to support itself by gravity. Very little is known of the law of strains in such a dam; but that it can act as an arch is demonstrated by the Bear Valley and Zola dams (see Table No. 60, page 260), the former of which could not stand for a minute if acting as a gravity-dam only. The arch form undoubtedly gives an additional margin of safety to a dam; but conservative engineers are not yet ready to include this in their calculations as a definite factor of resistance to overturning. In some locations, as already stated, a curved dam may permit of a more economical location than a straight one.

ART. 69. ROCK-FILL DAMS.

In many parts of the Western United States sites of proposed dams are at such a distance from any railroad, and transportation to them is so difficult, that a masonry dam

TABLE No. 60.

DATA OF MASONRY DAMS.

(From Wegmann's "Masonry Dams.")

Dam.	Country.	Date of Construction.	Depth of Water in Feet.	Total Height above Bed-rock in Feet.	Thickness at Top in Feet.	Thickness at Base in Feet.	Area of Profile in Square Feet.	Max. Pressure in the Masonry in Tons of 2000 lbs. per Square Foot.	Weight of Masonry per Cubic Foot.	Length at Top in Feet.	Length at Base in Feet.	Plan.
Alamanza	Spain	Prior to 1586	......	67.88	9.84	33.73	1496	14.33	......	292	45	Curved. $R = 86.07$
Elche	"	Ab. 1570–90	64.64	76.12	29.52	39.37	2615	13.00	......	230	60	" $R = 205.33$
Lampy	France	1776–1782	51.35	52.17	16.08	36.65	1229	......	......	......	......	......
Puentes *	Spain	1785–1791	153.54	164.24	35.73	144.29	16349	8.12	......	925	70	Polygonal
Zola	France	About 1843	119.76	123.02	19.03	41.83	3645	8.12	......	205	23	Curved. $R = 158$
Nijar	Spain	1843–1850	82.03	90.33	24.28	67.59	5386	7.68	......	356	....	"
Furens	France	1862–1866	164.04	170.60	9.91	161.02	10712	6.82	......	328	30	Curved. $R = 828.38$
Ternay	"	1865–1868	112.70	124.68	13.12	81.69	8355	7.16	147.0	......	......	" $R = 1312.4$
Habra †	Algiers	1865–1873	116.81	124.68	14.11	95.00	5584	6.31	134.0	1476	......	Straight
Boyd's Corner	United States	1866–1872	......	78.00	8.60	57.00	2039	......	......	670	200	"
Gileppe	Belgium	1869–1875	147.64	154.20	49.22	216.50	18708	6.14	143.5	771	269	Curved. $R = 1640.4$
Geelong	Australia	......	......	60.00	2.50	44.00	1214	4.00	143.0	226	......	" $R = 300$
Villar	Spain	1870–1878	162.30	170.33	14.75	154.50	11596	9.60	143.0	546	......	" $R = 440$
Bouzey ‡	France	1882	......	75.46	13.12	45.93	1961	11.26	......	1545	......	Straight
Harniz	Algiers	1885	114.84	134.52	16.40	91.21	5629	11.25	......	532	131	"
Bridgeport	United States	1886–1888	......	40.00	......	......	......	......	......	640	50	"
Vyrnwy	England	1882–	......	146.00	20.00	117.80	8972	8.70	160.8	1350	......	"
Tâche	France	1888–1892	......	161.43	......	......	......	11.00	150.0	......	......	Curved. $R = 1312.4$
San Mateo	United States	......	......	170.00	20.00	176.00	16660	......	......	700	......	" $R = 637$
Sodom	" "	1888–1893	......	98.00	12.00	53.00	......	......	......	500	150	Straight
Tansa	India	......	......	118.00	12.00	100.00	......	9.00	......	8800	......	......
Bear Valley	United States	1884	60.00	64.00	3.17	20.00	537	......	......	450	......	Curved. $R = 300$
Sweetwater	" "	1886–1888	90.00	98.00	12.00	46.00	2347	......	164.0	380	......	" $R = 222$
Mouche	France	...–1890	95.00	101.00	11.5	......	......	6.70	134.0	1346	......	Straight
Austin	United States	...–1892	60.00	67.00	......	......	......	......	......	1275	......	"
Butte City	" "	......	......	120	10	83	......	......	......	350	......	Curved. $R = 350$
Titicus	" "	......	......	135	18	75	5209	......	......	534	......	Straight
New Croton	" "	......	150	238	18	185	......	......	......	600	......	"

* Ruptured by weak foundation. † Destroyed; cause, poor material. ‡ Destroyed by sliding on poor foundation.

becomes a very expensive structure. In many cases there is, however, plenty of rock at or near the surface, and narrow cañons are available for a dam site. In a few of such locations dams have been constructed by simply depositing rough stone, as it was blasted out and with no dressing, in an embankment of the desired height, the stones being carefully arranged, smaller stones filling the interstices of the larger to give them a stable position and prevent after-settling. The front and back faces are generally lined with dressed rubble, dry or in cement, or with quarry stone laid carefully by hand and well chinked with spalls. Such dams are not expected to be absolutely tight, and should never be used as weir-dams. They are not adapted to the construction of storage-reservoirs, because of their porosity; but will generally silt up in time and become reasonably tight.

Since more or less water will find its way through a rock-fill dam the foundation should be on rock, as any other material is liable to erosion which may prove fatal to the dam. Since the dam acts as an embankment without cohesion, the side slopes must not be steeper than the natural "angle of repose" of the material. This will generally require a slope of at least 1 : 1; but it may be made $\frac{1}{2}$ or even $\frac{1}{3}$ to 1 on the back when this is faced with a heavy, well-built wall. In construction the material should be carried up in approximately horizontal layers, the middle being always kept a little lower than the faces; and the face-stone should be kept up to the level of the loose rock. The greatest care should be used to make the dam compact by the use of spalls and quarry-chips filling all crevices; but no earth should be used, except that clean sand might to advantage be sifted down into the crevices of stone already laid. The faces should be of one- and two-man stone, well bonded and chinked.

The back of a rock-fill dam is in some cases made more

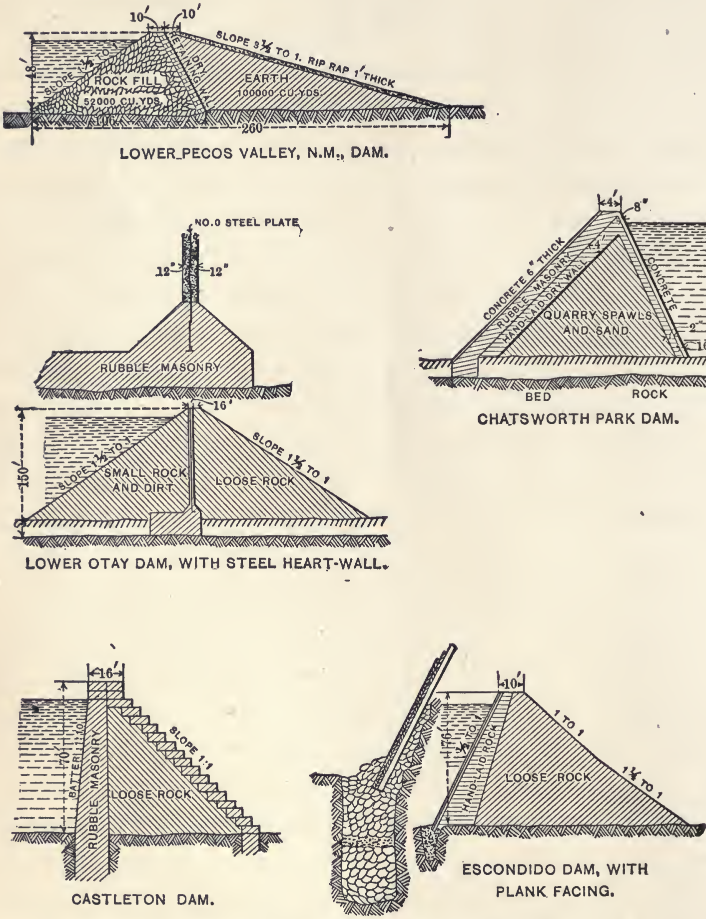


PLATE XVI.—ROCK-FILL DAMS.

nearly water-tight by banking earth against it, as in the case of the Idaho Company's Dam, in which the earth is 3 feet thick at the top and 20 feet at the bottom; and the Pecos Valley dams (see Plate XVI); or by wooden sheathing, as in the case of the Walnut Grove Dam, in which two thicknesses of 3-inch planking were used, with tarred paper between. In one—the Lower Otay Dam—an attempt to make the dam water-tight was made by the use of a heart-wall or sheeting through the centre of the dam, composed of No. 0 to No. 3 steel plates, imbedded in concrete 1 foot thick on each side. This construction was carried to the top of the dam, and to a masonry foundation on bed-rock, through its entire length. The loose rock, with a slope of $1\frac{1}{2}$ to 1, was carried down only to the ground-surface.

Still another style of rock-fill dam is the Castlewood Dam, which has both faces of rubble masonry, the upper being 6 feet thick on top and 12 feet on the bottom, and the lower from 5 to 7 feet in thickness, the heart being dry-laid rubble.

Table 61, page 264, gives a partial list of rock-fill dams.

ART. 70. TIMBER DAMS.

Timber dams cannot be considered as other than temporary structures, which must ultimately fail by the rotting of the wood; and they are seldom very tight. Their general construction is that of cribwork weighted with stone, and faced with planking. They are placed upon rock or boulder and gravel foundations—sometimes upon clay or other firm soil. Their chief or only advantage over masonry is in cost, which will ordinarily be but one-fifth to three-fifths as great for timber dams in a wooded country.

If the bottom is hard, cribs are weighted with stone and sunk directly upon it; but if soft, rip-rap or loose stone is

TABLE No. 61.

DATA OF ROCK-FILL DAMS.

Name.	Location.	Height.	Length on Crest.	Slope of Upper Face.	Slope of Lower Face.	Cost.	Remarks.
Bowman	California	100	425	1 to 1	1 to 1	$132,000	Part cribwork; upper face planked.
Eureka Lake	"	68.2	......	....	....	35,000	
Fordyce	Yuba, Cal.	75	650				
Walnut Grove	Arizona	110	420	0.5 to 1	0.65 to 1	110,000	Destroyed Feb. 1890 by overflowing.
Otay	San Diego, Cal.	130	545	1.5 to 1	1.5 to 1	....	Steel heart-wall.
Castlewood	Denver, Colo.	63.5	586	0.1 to 1	1 to 1	....	Rubble masonry faces.
Idaho	Boise River	43	220	1.5 to 1	1.5 to 1	....	Earth backing.
Escondido	California	76	380	0.5 to 1	1 & 1.25 to 1	86,946.21	Plank facing.
Morena	San Diego, Cal.	160	470	1 to 1	....	....	Facing of asphaltic concrete.
Chatsworth Park	California	41.3	159	0.57 to 1	0.57 to 1	9,000	
Pecos Valley	Eddy, N. Mex.	52	1686	0.5 to 1	1.5 to 1	170,000	Backed with earth embankment having a slope of 3½ to 1.
" " *	" " "	48	1380	0.5 to 1	1.5 to 1	....	

* Destroyed in August 1893 by insufficient spillway; rebuilt.

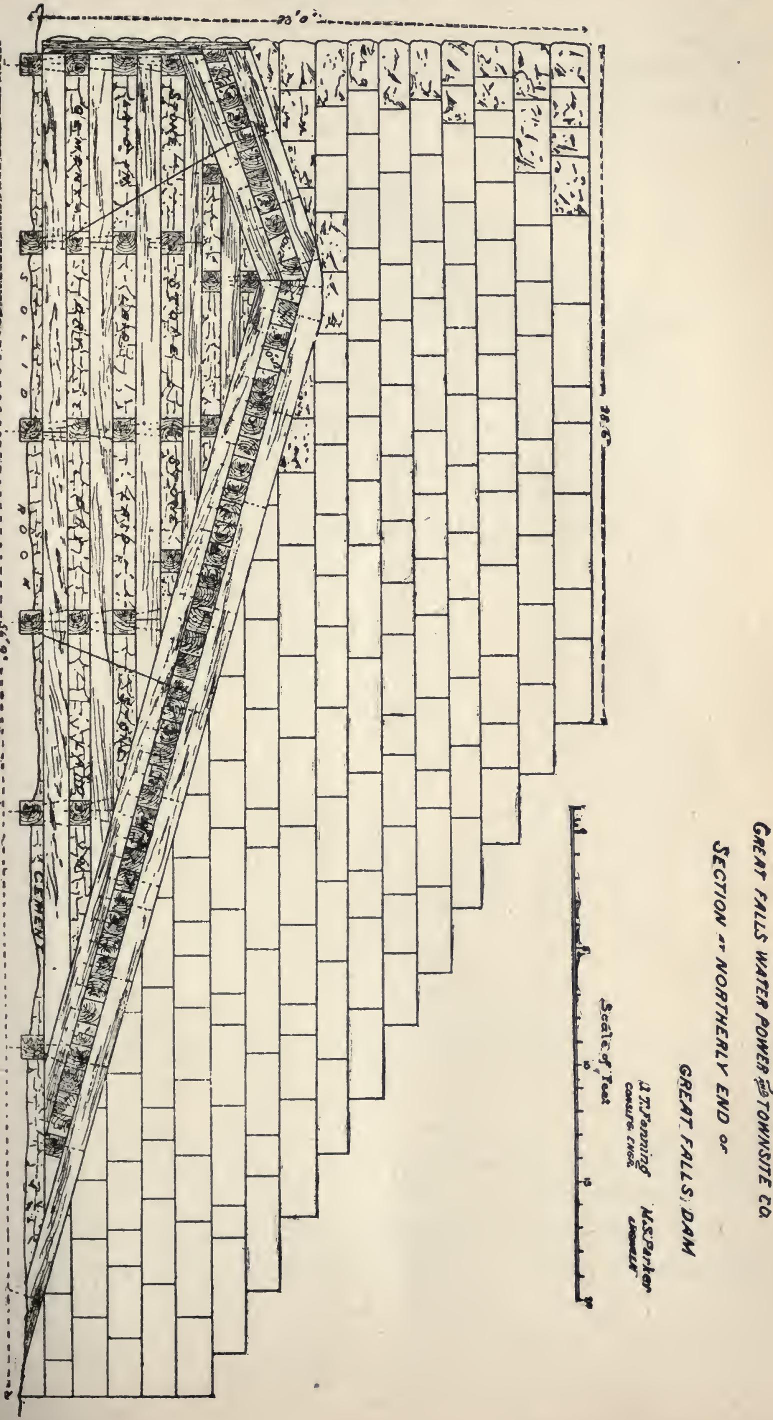


FIG. 29.—GREAT FALLS DAM. (From Trans. Am. Soc. C. E., vol. XXVII.)

placed over the bottom at the site of the dam and allowed to settle as far as possible, extending for some distance above and below the dam, and when levelled off the cribs are sunk

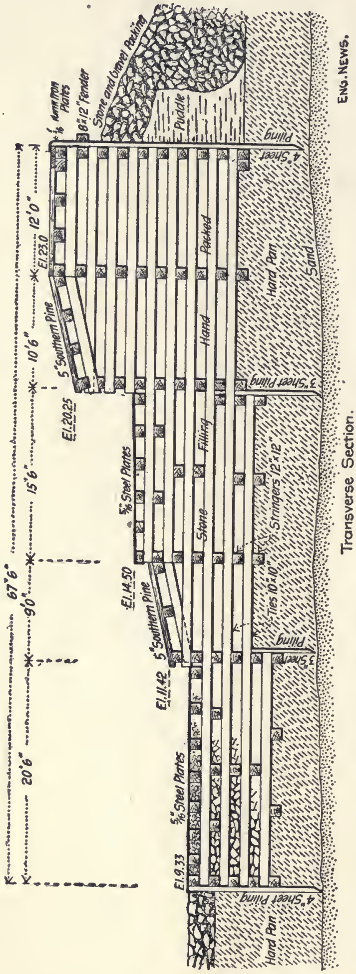


FIG. 30.—SEWALL FALLS DAM. (*Eng. News*, vol. XXXI.)

upon this. The timber is usually 12 × 12 inches or larger, drift-bolted together.

A timber dam, 14 feet high, was constructed at Great Falls, Mont., in 1891–2, across the Missouri River (Fig. 29);

and in 1892 and 1893 one was built across the Merrimac River at Sewall Falls, 497 feet long and 13.6 feet high, costing $120,000 (Fig. 30). Still another type is shown in Fig.

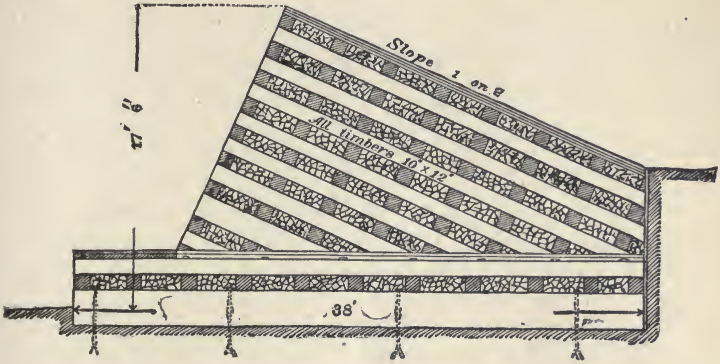


FIG. 31.—CROSS-SECTION OF BEAR RIVER WEIR.
(From Wilson's "Irrigation.")

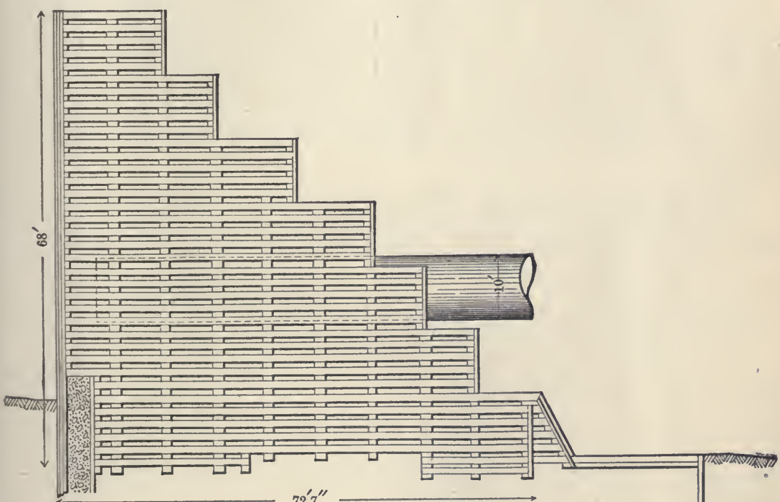


FIG. 32.—TIMBER DAM AT BUTTE, MONTANA.

31; and in Fig. 32 one 68 feet high and 500 feet long, built across the Big Hole River at Butte, Mont., in 1897–8.

ART. 71. EARTH EMBANKMENTS.

Earth has probably been used for more dams and embankments in connection with water-works than all other substances combined, and serves this purpose admirably if properly designed and constructed; but it can never, under any condition, be used for a weir-dam, but a spillway or waste-weir must be constructed in connection with it. Many earth dams have failed, but in the great majority of cases this has been because a too-small spillway has forced water to flow over the dam. An earthen dam is almost never absolutely tight, a slow seepage continually carrying a small amount of water through it, and if this becomes appreciable in amount or velocity the destruction of the dam is threatened. The great requisites of a safe earth dam are that it shall be so compact that no water can pass through it as a stream, however fine; and that there shall be no continual smooth surface, as of pipe or masonry, through it which the water may follow.

The pressure due to the head tends to force water in a reservoir through the earth embankment, and this is resisted by the friction in the pores. The greater the distance through which the water must pass the greater the friction and the less the seepage. Hence the bottom of an earth dam should be much thicker than the top. At 75 or 80 feet the saturation of the earth reaches an amount which some engineers would fix as a limit, although earth dams have been built more than 100 feet high.

The faces of an earth dam must be at least as flat as the angle of repose of the material used: and this when the material is either moist or wet. This angle for different soils is approximately as follows:

Kind of Soil.	Angle of Repose.	Equivalent Slope.	Coefficient of Friction.	Weight, Lbs. per Cu. Ft.
Earth, dry	40°	1.2 to 1	0.84	90
" moist	45	1 to 1	1.00	95
" very wet	32	1.6 to 1	0.62	115
Sand, dry	35	1.4 to 1	0.70	100
" moist	40	1.2 to 1	0.84	110
" very wet	30	1.7 to 1	0.58	125
Gravel, round	30	1.7 to 1	0.58	100
" sharp	40	1.2 to 1	0.84	110

From this it would appear that an embankment of earth should be given a slope of at least $1\frac{1}{2}$ to 1, and one of sand $1\frac{3}{4}$ to 1. To insure stability, however, and reduce percolation, and to prevent undue wash of the banks by rain, it is customary and advisable to give an inside slope of 2 to 1 and an outside of 2 or $2\frac{1}{2}$ to 1. If the dam is quite high a still greater thickness at the bottom than these slopes would give may be desirable to prevent percolation; also there is danger that the entire lining of the inside slope may, on account of its great mass, slip down the slope. On the outside of a high bank the rainfall upon it gradually collects in rivulets and tends to wash the soil. To prevent these objectionable results an offset or berme 5 or 10 feet wide is generally made about half-way up each face of high embankments.

If the slope is less than $2\frac{1}{2}$ to 1 at the top of an embankment more than 40 or 50 feet high, it should be given this slope below that distance from the top on the inside, and a slope of 3 or 4 to 1 on the outside.

The top of the embankment should be at least 6 or 8 feet wide, and if the dam is more than 20 or 30 feet high it should be 10 to 20 feet wide. It is generally made wide enough to act as a driveway. The embankment should be carried at least a foot higher than the highest waves coincident with the highest water—about 3 to 6 feet above high water, depending upon the length of the reservoir. The elevation above high water should also be at least 18 inches plus the depth to which frost reaches.

On the outside berme, gutters should be placed to catch the drainage from the upper slope, and drains to lead to the toe of the dam.

In the above the embankment has been considered to be of uniform homogeneous material. In many embankments, however, a core-wall is carried through the middle, constructed of masonry or water-tight puddle; and in others a water-tight lining of puddle or concrete is placed on the face of the embankment. In the latter case the bank is supposed to offer stability only and need not be water-tight; in the former that portion of the bank outside the core-wall is for stability only, that inside assists in preventing percolation. When a core-wall is provided the outer slope need not generally be flatter than 2 to 1.

While a core-wall is made as tight as possible, and in many cases is used for this purpose only, its chief value ordinarily is to prevent woodchucks, muskrats, crawfish, and other digging animals from making openings through the bank. Some engineers object to the use of a core-wall, claiming that since it and the remainder of the dam are not homogeneous cracks will open between during settling, to the weakening of the dam. It is undoubtedly true that a comparatively water-tight dam may be made of uniform material throughout, and one perfectly safe from all but the boring of animals, and small reservoirs may very properly be so built. But for large dams or those whose rupture would be attended with great damage or loss of life, the author would wish to use a masonry core-wall; and he would not recommend a puddle core-wall except to secure tightness where there could be obtained only a small amount of clay or other puddling material. The masonry core-wall may be of stone, brick, or concrete. It need not be heavy enough to resist any considerable pressure, this being sustained by the embankment.

Not only should a dam be tight itself, but it should be upon a water-tight foundation—rock, hardpan, or a thick clay stratum. If water work under a dam through a soil subject to erosion, a cavity will be created which will sooner or later cause the destruction of the dam. A seamy rock may permit the passage of water under or around a dam without endangering its stability; but this is undesirable because of the loss of water, and the dam may be destroyed

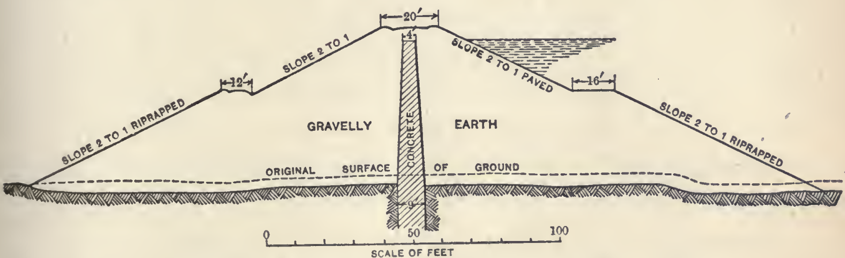


FIG. 33.—OAK RIDGE RESERVOIR DAM.

thereby; as was a dam at Roanoke, Va., in 1888. In the latter case the crevices should be sought out and filled with concrete or stone masonry; or it may be necessary to build a masonry wall as a lining to the seamy sides of a reservoir.

If the rock or hardpan at the bottom of the valley is covered with porous material—as is generally the case—this should be removed and the embankment founded on only solid, continuous impervious material. If the previous surface material is quite deep and the dam high this may require the excavation of an enormous amount of material. Thus a dam 40 feet high, top 10 feet wide, side slopes 2 and 2½ to 1, will have a bottom width of 190 feet (the San Leandro Dam has a bottom width of 700 feet), and if 10 feet of soil be removed this will require the excavation of 70 cubic yards per running foot of bank. To avoid the great expense and delay occasioned by this, a puddle or masonry cut-off wall is generally carried to the impervious stratum, and continued as a core-wall, if such be used, or stopped a few feet above the base of the embankment if no core-wall be used; or, if a lining be given to the dam, the cut-off wall is placed under and joined to the foot of this lining (see Fig. 36, page 275). The Oak Ridge Reservoir Dam of the East Jersey Water Company (Fig. 33) is a good illustration of a dam with a concrete core (or heart) and cut-off wall.

Not only the bottom but also the ends of a dam must make an impervious union with the natural soil. For this purpose the core-wall should be carried to bed-rock at the ends of the dam if the rock rise there above the water-level;

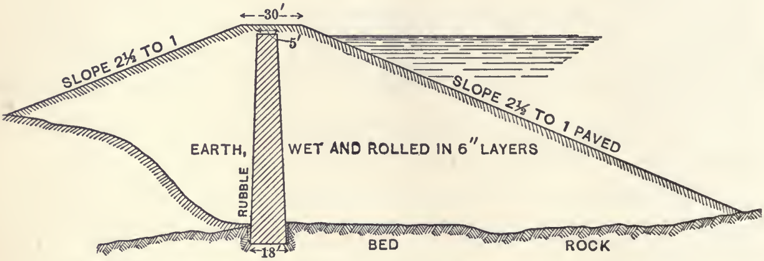


FIG. 34.—RESERVOIR M DAM; NEW YORK WATER-SUPPLY.

or for some distance into the bank if it do not. Or, if there be no core-wall, the embankment should be extended until it reaches an impervious material, or for a distance into the sides of the valley equal to, say, 10 feet plus half its height if no impervious material be found short of this.

Almost every character of soil has been used for embankments: and there are few from which a reasonably good embankment cannot be made. Probably the best material is a sandy loam with a small amount of clay intermixed, and the worst is micaceous clay. Contrary to a wide-spread opinion, clay is not a good material for embankments in any but small quantities. A tough clay it is difficult—almost impossible—to get into a compact homogeneous mass; if wet it cracks open on drying out; and, most serious of all, if the smallest trickle of water once finds passage through it, a few minutes suffice to enlarge this into a break, so rapidly does it dissolve in running water. An embankment of clay is often tighter at first than one of any other material, but the danger of its rupture increases with age. A bank containing a large amount of gravel or sand, on the other hand, may leak at first, but becomes tighter and stronger with age. A reservoir embankment 16 feet high, built of fine sand, with slopes of $1\frac{1}{2}$ to 1 covered with loam, has been found to suffer no appreciable leakage. Gravel as taken from the bank, consisting of stones, sand, and loam, makes an excellent embankment material. "Gravel capable of being puddled will do anything that clay was ever used for in water-works practice, and will do it better" (Clemens Herschel). Hardpan, the most impervious soil-mixture found in nature, contains a large proportion of gravel.

If the following materials are obtainable, they may be used to advantage in the proportion given, to make an excellent puddle, the amounts here given furnishing one cubic yard:

Coarse gravel..................	$\frac{3}{4}$	cubic	yard
Fine gravel or coarse sand......	$\frac{1}{4}$	"	"
Fine sand....................	$\frac{1}{8}$	"	"
Clay or loam..................	$\frac{1}{8}$	"	"

These materials should be thoroughly mixed, the clay being broken up or cross-cut into fine pieces, made slightly damp, spread in 6- to 9-inch layers, and thoroughly rammed, or rolled with grooved rollers. If the gravel and sand retain their natural moisture no water should be used in the material. Embankment material is seldom wet too little, often too much. Natural hardpan, if broken up and rolled dry, will make a tight dam if water be admitted behind it slowly, the clay taking up water by capillary attraction and swelling. Puddle should never be made wet enough to quake, as it would then be porous upon draining or drying out.

During construction all surfaces coming into contact should be rough. The ground upon which the dam is built should be stripped of all the soil which is porous or contains roots or other vegetable matter, and then ploughed or spaded up, that the new and old material may unite; also the top surface of the embankment should be rough when more

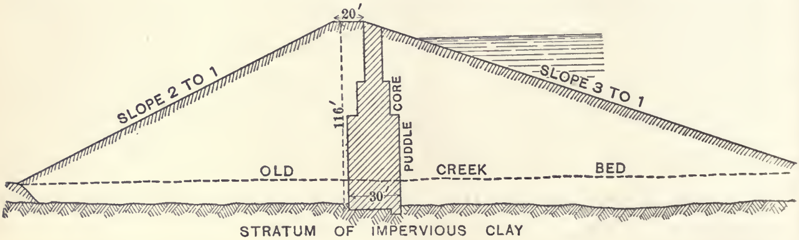


FIG. 35.—HONEY LAKE VALLEY DAM.

material is placed upon it, and the rollers used should be grooved. An earth embankment should never be placed upon rock, unless a masonry core-wall be used and carried down 3 feet or more into the bed-rock, as a water-tight joint cannot be made between earth and a smooth rock-bed.

In building up the embankment the centre should be kept somewhat lower than the faces; and these should extend one or two feet beyond the line of the finished bank and afterward be trimmed off, since the edge of a bank cannot be properly compacted. No large stones, sticks, roots, or any matter which can decay should be permitted in an embankment. If the material needs wetting, this might better be done before the fresh layer is put on, rather than after; the bond between the fresh and old material being thus made more thorough, and the material being less liable to cling to the roller.

Nothing should extend through the dam when this can be avoided, and any pipe or other conduit which must pass through it should be furnished with a number of flanges and other projections, or if of masonry should be left as rough as possible; and the best of puddle, mixed and rammed with the greatest care, should surround it. Where possible it is ordinarily better to place the spillway or the overflow-pipe (which may be used if the reservoir is small and does not receive drainage direct) at one end of the dam on the natural soil; or the overflow-pipe and the conduit may be carried in a tunnel which passes through the dam and terminates at its upper end in a tight inlet-tower, as in the case of a masonry

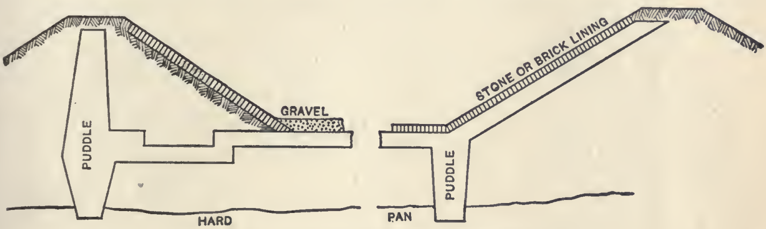


FIG. 36.—PUDDLE-LINED RESERVOIRS.

dam. Such a tunnel should be in every case carried down to bed-rock throughout its length. If a pipe or conduit be carried through the embankment it should rest, not on masonry piers 12 feet apart—a construction too often adopted, to the endangering of the pipe by breaking between supports—but upon bed-rock or hardpan, or a continuous foundation of the roughest possible masonry in cement-mortar carried down to rock or other firm bottom, and provided with projecting or cut-off walls to prevent water from following the masonry through the dam. Next to insufficient spillways, improperly built conduits through embankments have been the cause of the greatest number of ruptures in earth dams.

Where there is a masonry waste-weir in the centre of the

TABLE No. 62.

DATA OF EARTH DAMS.

Name.	Location.	Greatest Height.	Length on Crest.	Width of Crest.	Slope of Upper Face.	Slope of Lower Face.	Heart-wall.	Remarks.
Dudley Brook.........	Windsor, Vt..........	31	170	4	1½ to 1	1½ to 1	None	Clay and gravel; masonry spillway, no leakage.
Salisbury Brook..	Brocton, Mass........	23.2	1524	16	2 to 1	1½ to 1	Concrete, 3′ to 1½′	Clayey gravel; masonry spillway, no leakage.
Stony Brook	Cambridge, Mass.	25	706	20	1½ & 3 to 1	2 to 1	Masonry, 8′ to 3′	Gravel; leaks around end.
Acushnet River........	New Bedford, Mass...	....	650	20	2 to 1	2 to 1	Puddle, 12′ to 4′	Fine gravel, coarse and loamy sand.
Waterville............	New York............	17	240	10	2 to 1	2 to 1	Masonry, 3′ to 1½′	Distributing-reservoir.
Compton Hill..........	St. Louis, Mo.........	26	....	20	1½ to 1	2 to 1	Puddle, 7′ to 4′	Puddled bottom; clayey loam. Storage-reservoir.
Reservoir "M"	Titicus River, N. Y...	100	1000	30	2½ to 1	2½ to 1	Rubble, 18′ to 5′	
Oak Ridge............	Pequannock Riv., N.J.	60	620	20	2 to 1	2 to 1	Concrete, 9′ to 4′	Overflow detached from dam.
Honey Lake Valley....	California............	90	960	20	3 to 1	2 to 1	Puddle, 25′ to 10′	Crest 6′ about high water.
Dam No. 5............	Boston, Mass..	70	1865	14	2 to 1	2 to 1	Concrete, 10′ to 2′	Central masonry waster-weir.
Pilarcitos	California............	95	650	26	2¾ to 1	2 to 1	Puddle, 20′	Heart-wall 46ft. below base of dam.
San Andreas..........	"	95	650	25	3½ to 1	3 to 1	" 20′	" " 48 " " " "
Crystal Springs........	"	50	560	30	3½ to 1	3 to 1	" 25′	" " 98 " " " "
San Leandro..........	"	110	600	40	3 to 1	3 to 1	" 50′ to 40′	" " 40 " " " "

dam it is an excellent plan to carry all pipes or conduits through this, as was done at No. 5, Boston Water-works.

Table No. 62 gives the data of only a few earth dams, as an illustration of general practice.

Art. 72. Hydraulic Dam-construction.

Two or more dams have been constructed in California by what is known as the hydraulic method, one of which, La Mesa Dam, is "66 feet high, 251.5 feet thick at the base, and 20 feet wide at the top, the materials for which were transported and deposited in place by flowing water, by the process known to miners as 'ground-sluicing,' the surplus water from the flume (the San Diego Flume) being used for this purpose and at the same time stored in the reservoir as it was being formed back of the dam." "The volume of material handled was 38,000 cubic yards, which had to be brought an extreme distance of 2200 feet, and stripped from an area of 11.5 acres to a mean depth of 2 feet. . . . The water used was from 300 to 400 miner's inches—6 to 8 second-feet. . . . From the main ditch at various points laterals were carried down the slope of the hill toward the dam on a grade of 6%, dividing the ground into irregular zones of 50 to 100 feet in width by several hundred feet in length, reaching back to the top of the ridge. In sluicing, these divisions were stripped off clean to bed-rock, beginning next to the dam and working back to the head ditch, the water being carried along the upper side of the strip to the lower side across the end of the division where the ground-sluicing was progressing.

"The fall from the upper line, or clear-water ditch, to the lower side of the zone was as great as the slope of the ground would admit of—the greater the fall the more rapid the sluicing. The work done was highly satisfactory as long as this slope was not flatter than about 1 in 4. . . . As the stream secured its load of earth and gravel it was conveyed along the line of the lower ditch by 24-inch wooden-stave pipes until deposited on the embankment. . . . The pipes were found to wear very rapidly, and were lined first with strips of wood and then with strap-iron or tire-steel. Cast-iron pipe is found to be preferable for this sort of service. . . . It is apparent that an embankment built in this manner is compacted as thoroughly as it could be by any process of rolling, and is not subject to further settlement. It is also manifest that the finer materials are by this process precipitated in the interior of the fill, and that the particles are in a general way graded in size from the outside toward the

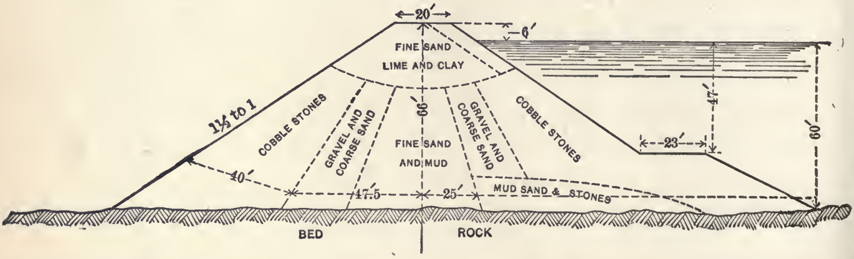


FIG. 37.—LA MESA DAM. HYDRAULIC CONSTRUCTION.

centre." (See Fig. 37.) (From "Reservoirs for Irrigation," by James D. Schuyler, in the U. S. Geological Survey Report for 1896–97.)

It is evident that the localities where this construction is possible are limited in number.

ART. 73. RESERVOIR LINING.

Storage-reservoirs are seldom less than an eighth of a square mile in area, and it would ordinarily be impracticable to line or cover these; and the lining is not often necessary to secure tightness, since the reservoir is in most cases formed by a dam across a narrow valley, which dam, or its core-wall, can be carried as a cut-off wall to rock or clay on both bottom and sides.

A distributing-reservoir, however, is more frequently constructed on a mountain side or top, and is built largely or wholly by excavation and embankment. This construction will in many cases produce a reservoir which is porous on the sides of the excavation; and on the bottom also, if rock or hardpan is not reached. Such a reservoir will require either that a core-wall carried down to clay or rock entirely surround the excavation, or that the reservoir be lined with some impervious material.

When rock occurs over the entire bottom of the reservoir or a considerable portion of it, it is practically impossible to render this tight by any material other than masonry, either as a lining or core-wall. Since the latter would call for a trench excavated to rock entirely around the reservoir, the masonry is in most if not in all cases placed as a lining. If the excavation extends for any distance into rock, the face of this is ordinarily made vertical; and the earth should be faced with a water-tight retaining-wall, since it is apt to slip on the rock-surface. Such a construction is shown in Fig. 38, page 281, a section of the Manchester, N. H., high-service reservoir. The bottom may be left unlined if the rock has no seams and is perfectly smooth; but it will generally be advisable to give it a lining of concrete—from 6 to 12 inches if to prevent leakage; otherwise sufficient to level off the surface and render easier the drawing off of all the water and the cleaning of the reservoir. The retaining wall should be practically water-tight; but is generally backed with earth to add to the imperviousness and stability, and should therefore be capable of retaining this without overturning when the reservoir is empty.

When rock forms the bottom of the reservoir but is not excavated at all, the earth may be given a slope of $1\frac{1}{2}$ to 1 if only 8 or 10 feet deep, but should be 2 to 1 if 15 or more feet deep. The use of clay puddle for a lining to such a reservoir might be adopted, but cannot be recommended as always satisfactory, concrete being a safer material for both bottom and sides. Embankment which is to be lined should be compacted with particular care, and but a small amount of water used in the material.

A reservoir entirely in earth may be lined with either concrete or clay. If the former be used the bottom should be thoroughly rolled with heavy rollers to compact it; and a layer of gravel spread before rolling will be of assistance in securing this condition. If the bottom be of clay, this treatment will often make an artificial hardpan which will be as good as a clay puddle. If hardpan be reached in the bottom it will ordinarily be necessary merely to level this off and roll it with a heavy flat roller. If no clay or hardpan be reached a puddle lining is advisable. This may be mixed in proportions of clay, 1 part, sand and fine gravel, 1 part, and coarse gravel, 2 parts. This should be put on in 4-inch layers and thoroughly rolled, the total thickness being 1 to 3 feet, depending upon the depth of water. This should be protected in some way from penetration by fish, etc., wash of water, and injury during cleaning of the reservoir. Six inches or so of gravel, with dry walls on the slopes, is often used for this purpose; but a better lining is made of 4 to 8 inches of concrete.

Clay lining is now seldom used for embankments, because the lining is apt to slip and to be loosened by frost, to crack open if alternately wet and dried, and to break if a settlement occurs in the bank; and because the same amount of puddle can generally be used to better advantage as a heart-wall. A small reservoir on porous soil may require to be lined throughout, and in this case a puddle bottom-lining may be used and continued either as a slope lining or as a heart-wall (see Fig. 36, page 275). Clay lining should always be protected from injury; that on the slopes, by masonry.

Additional tightness has been secured when vertical walls are used, by building practically two walls around the reservoir, one outside the other, and with a narrow annular space

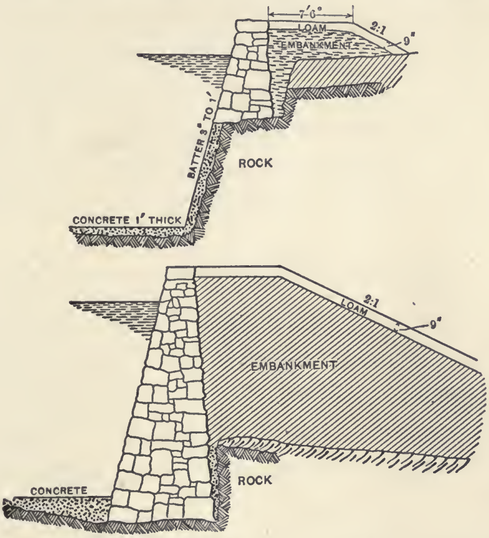


FIG. 38.—RESERVOIR IN ROCK EXCAVATION, MANCHESTER, N. H.

between them which is filled with rich Portland cement-mortar, with or without the admixture of fine stone or gravel.

Asphalt has been used as a lining in many Western and a few Eastern reservoirs, being placed upon concrete or directly upon the earth embankment. "The natural properties of asphalt seem to render it a useful material from which to form a water-tight coating for reservoirs. It is insoluble in water, and neither acids nor alkalis dissolve it or affect its cohesion; hence it imparts no taste or color to water coming in contact with it. It is elastic and will yield to a considerable settlement of the surface on which it lies without cracking or losing its integrity. It is easily repaired, and new material can be made to unite perfectly with the old, wherever a patch may be needed." (Jas. D. Schuyler, Trans. Am. Soc. C. E., vol. XXVII. page 629.) Trouble was experienced in the earlier use of asphalt because of its tendency to flow on the side slopes, more particularly above the water-surface when exposed to the hot sun. It is thought that this difficulty has been overcome in more recent practice by use of the proper grades of material. "The first coat should be liquid asphalt, so called, which has great penetrating qualities, enters fully into the foundation-soil and takes root, so to speak. This coating has great adhesive qualities which are of great value, but on the other hand it is utterly lacking in ability to stand the sun's heat.

"The second coat, the sun-proof coat, should consist of hard rock asphalt heated up to 300° F. and applied hot. This coating is both water-proof and sun-proof, but is utterly lacking in adhesive qualities, and were it not for the first coat underneath could be taken up readily like a carpet." (L. J. Le Conte, in Trans. Am. W. W. Ass'n, June, 1896.)

Asphalt has also been used to give elastic or expansion joints between sections of concrete lining, as in the Astoria, Ore., Water-works Reservoir, the bottom lining of which was formed of 6 inches of concrete, $\frac{3}{8}$ inch of mortar, and a coat of asphalt, the concrete being made in 20-foot sections joined by asphalt, which served its purpose admirably, the desirability of its use being shown by a change of $\frac{3}{8}$ inch in the width of joints under various temperatures. Such expansion joints are desirable in large continuous surfaces of concrete, since the coefficient of expansion of this is practically the same as that of iron; that is, a sheet 200 feet long would change about an inch and a quarter in length if the range of temperature were 80° F.

Another method of using asphalt is illustrated by the side lining of this reservoir, which was formed of concrete coated with asphalt, and covered with a layer of brick dipped in hot

OF CALIFORNIA

asphalt and laid flat, a final coating of asphalt being given to the whole.

In southern California a dam has been lined with concrete formed of broken stone (all grades including dust being used) and asphalt, and this construction would seem to offer some advantages. (See Trans. Am. Soc. C. E., vol. XXXV. p. 70.)

In the East, asphalt has been used for rendering tight defective reservoirs at Philadelphia and at Payson Park, Boston, both originally lined with concrete, and has proved satisfactory for this purpose.

ART. 74. COVERED RESERVOIRS.

An impounding-reservoir is of such size as to render a covering impracticable, and distributing-reservoirs only have been covered. Covering is most frequently desired when the supply is from ground-water, which is so often given an objectionable taste by the presence of algæ. By using covered reservoirs the water is delivered to the consumer without having been at any time exposed to the light, without which very few algæ can exist. A covering is also desirable to exclude dust and other atmospheric impurities and to prevent malicious pollution; also to protect the water from heat and maintain a uniform temperature and to prevent loss by evaporation. A covered reservoir should be ventilated.

The covering may be of timber, metal, or masonry. The former is generally the cheaper but least satisfactory. It is generally constructed by resting horizontal beams upon timber or iron posts or brick piers, the beams being spaced equally over the entire area of the reservoir and covered with plank. For small reservoirs but 40 or 50 feet in diameter an ordinary circular wooden roof has been used, covered with tin or slate. For somewhat larger reservoirs steel roof construction has been used. This, however, offers little protection from the sun's heat. At Quincy, Ill., a reservoir 415 by 317 feet was covered with timber by the first method (see Fig. 39). At Pasadena, Cal., five reservoirs are covered with timber roofs; the largest, 330 by 540 feet, using 2-inch iron

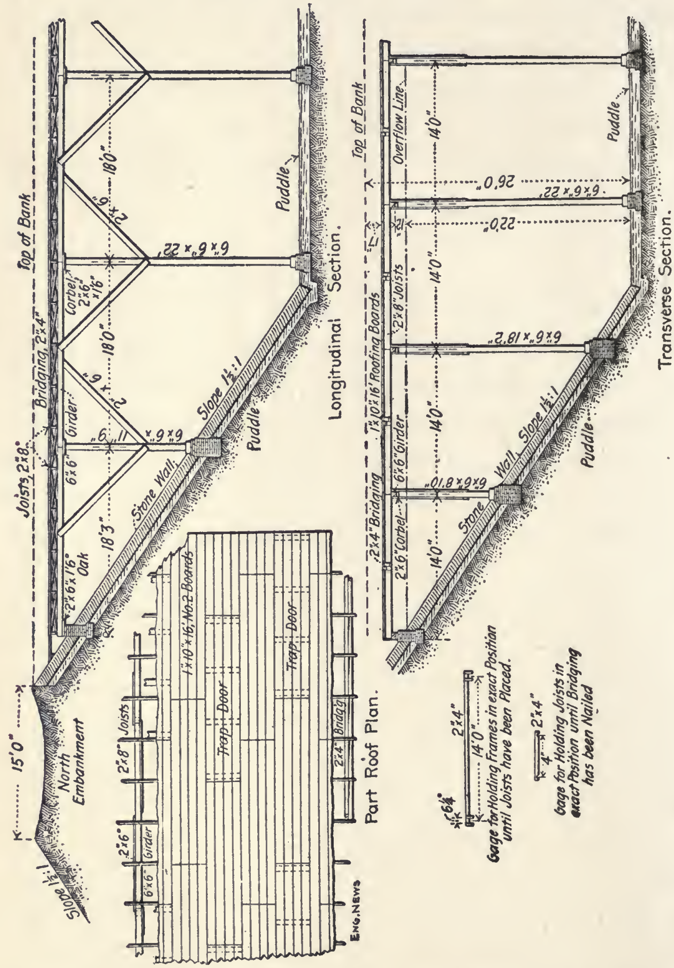
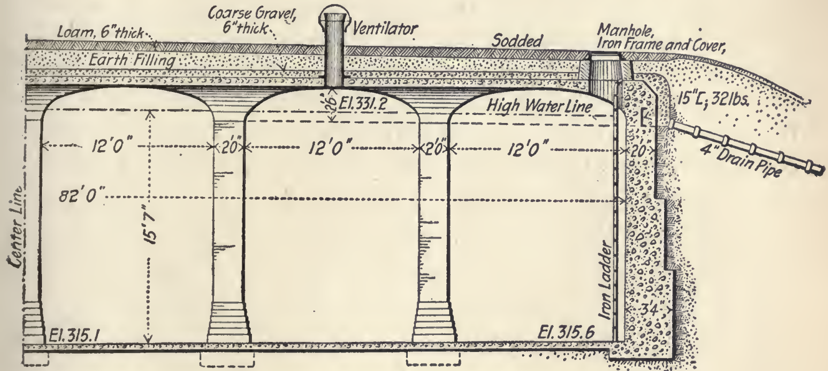


FIG. 39.—COVERED RESERVOIR, QUINCY, ILL. (From *Eng. News*, vol. XXXIX.)

pipe for posts and 4″ × 10″ × 18′ girders. No snow falls in this climate; but the weight of this must be considered in designing roofs in Northern localities.

A roof of brick or concrete masonry is generally covered with earth to protect it from accident and to assist in excluding the heat of the sun. Such a covering in the form of a circular dome was used on a 50-foot circular reservoir at Coshocton, Ohio, the roof costing $779. At Brookline and Wellesley, Mass., arches resting on masonry piers were used, the arches being groined elliptical in the latter (see Fig. 40);



Half Vertical Section.

FIG. 40.—COVERED RESERVOIR, WELLESLEY, MASS.
(*Eng. News*, vol. XXXVIII.)

and in the former, covering arches were supported by brick piers and connecting arches or lintels. The latter plan was also adopted at Hudson, Wis., the covering arches being of hollow tile, however, instead of brick. Hollow tile was used at Waltham, Mass., also for a dome covering.

A reservoir was constructed at Boise City, Idaho, in 1891, by tunnelling into a sandstone mountain for a depth of 125 feet, the tunnel being 20 feet wide, 7 high, and plastered with ½ to 1½ inches of cement-mortar; a covered reservoir for artesian well-water being thus formed. This method is of course applicable to few localities.

QUERIES.

21. Describe fully the purpose of each detail of construction shown in Fig. 25.

22. A dam is built of granite masonry, has a vertical up-stream

face, is trapezoidal in section, is 5 feet wide on top, 20 feet wide on the base, and is 35 feet high. What are the factors of safety against overturning and sliding when water flows 2 feet deep over the crest? What is the maximum pressure per square foot on the foundation? What thickness of base must a concrete dam have, its height and crest-width being as before, to give the same factor of safety against overturning?

23. If the Bear Valley Dam (see Table 60) has a trapezoidal section and vertical up-stream face, and were resisting the water-pressure by gravity alone, where would the resultant cut the base when water was flowing 1 foot deep over the dam? How high could the water rise above the foundation before the resultant would move out of the middle third of the base?

CHAPTER XIII.

PURIFICATION OF WATER.

ART. 75. GENERAL METHODS.

THE aims of purification may be considered to be fourfold: to remove bacteria, matter in suspension, matter in solution, and color. Bacteria might be included under the head of "matter in suspension," but their importance and the special consideration which they receive make it desirable to classify them alone. The methods used for purification are in general: sedimentation; straining, or mechanical filtration, with or without the use of coagulants, commonly called the American method; bacterial filtration, in which the organic matter is oxidized, commonly called the English method; chemical purification, including softening of hard waters, and removing iron in solution by aeration; and distillation.

It is only within a very few years that it has been realized that a method of purification which was adapted to one water might not be adapted to another. Thus, mechanical filtration gives excellent results in East Providence, but was a failure at New Orleans; and sand-filtration, while successfully used in scores of European plants, has proved inapplicable to some of our muddy streams. During the past two years exhaustive investigations made at Louisville, Pittsburg, and Cincinnati, and lesser ones at Quincy, Ill., and other places have vastly increased our knowledge of the possibilities and limitation of different methods of purification; and the experiments conducted by the Massachusetts State Board of Health during the past eleven years have been of the greatest value, particularly in discovering the natural laws upon which bacterial action depends.

ART. 76. SEDIMENTATION.

Sedimentation has already been referred to (Art. 34) as a method of purification. It is, however, extremely slow in its action, except in removing the grosser matters in suspension. The lower Ohio and Mississippi rivers during floods contain large quantities of clay particles, many of them not more than .00001 inch diameter, which are present in the water for weeks at a time. These settle very slowly, weeks and even months being required to wholly clear such water by this means. In time, however, all matters in suspension, including all or most of the bacteria, will be deposited. Sedimentation requires that the water be perfectly quiet or have a very slow motion and hence "settling-basins" are necessary in which water can be stored, while the consumption is being derived from duplicate basins; or the water is continuously drawn from large basins so constructed that the velocity of flow shall be very low and uniform throughout the basin, these being called "continuous-flow basins." A clear-water reservoir should be provided to receive the clarified water from either perfect rest or continuous-flow basins. At Omaha, Neb., five basins in a series are used, and the proportion of sediment deposited in each is shown by the fact that the first two are cleaned once in two weeks, the next one once a month, the next once in six weeks, and the next, which is practically a clear-water basin, once in a year. Geo. W. Fuller found, in the Louisville experiments, that in quiet water 75% of the matter in suspension was deposited in twenty-four hours; but that little more was removed during several days following. When there was a great deal of fine clay in the water but 50% was removed in this time. From the Cincinnati experiments he concluded that "sedimentation for more than three days would not be practicable, because: (1) the fine clay particles remaining after seventy-two hour's settlement subside very slowly, the percentage of decrease per day being seldom more than 5% of the original for the fourth day and steadily decreasing thereafter; and (2) the cost would be excessive." The latter is evident when we consider that, if the method of perfect rest be adopted, basin-capacity must be provided equal to the total consumption during the period of rest plus a certain amount, say 25%, additional for sediment, with at least one additional basin which will be out of service while being emptied and filled.

From investigations of sedimentation at St. Louis in 1886 it was concluded that the system of perfect rest was the most efficient and economical; the time allowed, however, must be decided with reference to the character of the suspended matter in the water to be clarified. From the experiments on continuous flow it was concluded that a flow of only 1200 gals. per day per square foot of cross-section of basin would give an effluent comparable with that from perfect rest. The length of basin would need to be such that twenty-four to seventy-two hours would be consumed by each gallon of water in passing from the inlet to the outlet. Later investigations have only confirmed these conclusions.

In some cases, where water is ordinarily clear, is occasionally slightly muddy, but for infrequent periods of three or four days only is quite muddy, clarification of the last-named water may be avoided by providing storage-reservoirs capable of supplying the consumption for this period, and clarifying only the slightly muddy water. If the suspended matter in this latter is mostly clay, however, the time required for complete clarification will be as great as for the muddiest water, and filtration would be preferable.

The great fluctuations in the amount of suspended matter carried by some streams is illustrated by the Ohio River, which, during the Louisville experiments, contained matter varying from 1 to 5311 parts by weight per million of water.

It might be stated as a general conclusion that sedimentation will effect a considerable clarification, the extent of this depending upon the fineness of the material carried; but that if much of this is fine clay the resultant water will hardly be fit for domestic use; and that the proportion of bacteria removed will be approximately the same as that of the fine clay. It will be seen, however, that sedimentation may be a valuable preliminary to more thorough purification.

ART. 77. ENGLISH METHOD OF FILTRATION.

The English method of filtration has been used in more water-works systems, not only in Europe but in this country also, than any other. It is essentially a comparatively slow filtration through sand. (Comparatively with reference to the American system of filtration; the sand filtration of sewage is conducted at rates much lower than that of water.) The water is flooded upon a bed of sand from $2\frac{1}{2}$ to 5 feet thick, and passes through this at a rate of $1\frac{1}{2}$ to 3 million gallons per acre daily. The German government has fixed 2,600,000 gals. per acre per day as the maximum rate of filtration permissible, or about four vertical inches per hour.

The sand which forms the filtering material rests, in most filters, upon a bed of fine gravel, and this in turn upon coarser gravel; and the filtered water is drawn off by underdrains. This construction is necessary, since sand in contact with the drains would be washed into them, and the material placed around them must also be sufficiently porous to permit full access of water to them. Hence the bottom layer is often of broken stone, the next of gravel one third the size, and successive layers of material 2 to 6 inches thick follow, each having grains, say, one third the size of those in the next lower layer: as, for instance, stones, 10 mm. diameter, 3.5 mm. diameter, 1.2 mm. diameter, and .4 mm. diameter. The top layer of sand should not at any time become less than 12 to 15 inches thick, and is generally made originally 30 to 40 inches to permit of removing clogged material. In the Albany, N. Y., filter-beds, which were put into service in August 1899, and are the largest in the country and probably represent the most advanced ideas, it was specified that "The lower 7 inches shall consist of broken stone or gravel, which shall remain upon a screen with a mesh of 1 inch, and which has but very few stones over 2 inches in diameter. Above this shall be placed 2½ inches of broken stone or gravel which has passed a screen with a mesh of 1 inch, and which remains upon a screen with a clear mesh of ⅜ inch, and above this shall be placed 2½ inches of broken stone or gravel, which has passed a screen with a mesh of ⅜ inch, and which is coarser than the ordinary sand, and entirely free from fine material. . . . The filter-sand shall be clean river, beach, or bank sand, with either sharp or rounded grains. It shall be entirely free from clay, dust, or organic impurities, and shall, if necessary, be washed to remove such materials from it. The grains shall, all of them, be of hard material which will not disintegrate, and shall be of the following diameters: Not more than 1%, by weight, less than 0.13 mm., nor more than 10% less than 0.27 mm., at least 10%, by weight, shall be less than 0.36 mm., and at least 70%, by weight, shall be less than 1 mm., and no particles shall be more than 5 mm. in diameter. The diameters of said grains will be computed as the diameters of spheres of equal volumes. The sand shall not contain more than 2%, by weight, of lime and magnesia taken together and calculated as carbonates."

The size of sand-grain is generally expressed in terms of the "effective size" and "uniformity coefficient," this method of classification originating with Allen Hazen, the designer of the Albany plant. The effective size is defined as "such that 10% of the material is of smaller grains and 90% is of larger grains than the size given. The results obtained at Lawrence indicated that the finer 10% have as much influence upon the action of the material in filtration as the coarser 90%." The uniformity coefficient is "a term used to designate the ratio of the size of grain which has 60% of the sample finer than itself to the size which has 10% finer than itself."

The fine top layer is the filtering layer, but the most effective agent in removing the finest suspended matters is a covering of slime, called by the Germans Schmutzdecke, which forms upon the surface and in the top layer of the sand. It is generally compact, membranous, and highly impervious, probably formed of a jelly-like material produced through bacterial agency. This covering, with the top inch or two of sand, does 75 to 85 per cent of the work of straining, and retains the finest matters, including clay and many bacteria.

A small amount of matter in suspension is removed by the sand alone, but the principal change which goes on here—and which is also carried on in the Schmutzdecke above—is the bacterial oxidation of the organic matter, both in suspension and solution, into the form of nitrites and nitrates. This latter operation requires time, and for this reason the rates given above have been found to be the maximum desirable. Another reason for fixing a low rate is, that if by force a high velocity through the Schmutzdecke is obtained, this is apt to be broken; the immediate cause of this breakage being possibly a washing away of part of the supporting sand into the under layer of gravel, owing to the too-high velocity.

The removal of bacteria is attributed both to the straining action of the gelatinous covering and to the removal of their food-matter by its oxidation.

It is seen that the sand forms, not the filtering material itself except to a minor degree, but rather the support and habitat for the purifying agents. These are not formed immediately, and no method has yet been found for supplying them artificially; it therefore follows that a new filter, or one from which the top layer has been removed, can effect the most thorough purification only after these agents have become re-established—a result which is perfected only after several days or even weeks of use, although a passable degree of efficiency is often attained in three or four days. In the filter at Lawrence, Mass., "in the first three days 75% of the number of bacteria applied came through the filter. In the next three days 30% came through"; and "during the first three weeks' use the number of bacteria in the effluent rapidly decreased to 2% of the number applied." At Altona, Germany, when the filter has been scraped and new sand applied, 1880 bacteria have been found in the effluent after one day, 752 after two days, 208 after three, 156 after four, 102 after five, and 84 after six days, the rate before scraping having been 42 per cubic centimeter.

As suspended matter collects on and in the gelatinous covering, the pores of this and the sand became so choked as to interfere with the passage of water through the filter. In fact, soon after the filter is put into service it is found that the head of water on the filter must be increased to produce the required velocity of percolation. The limit of depth of water permitted upon European filters is placed at 36 to 52 inches, or even in some cases at 24 inches. When the limit is reached the water on the filter must be drawn off and wasted, and the top layer of compact material removed; this layer amounting generally to from $1\frac{1}{2}$ to $\frac{1}{4}$ inch, or the least that can well be handled uniformly. Water is then turned slowly onto the bed — usually from below — until it is thoroughly filled and covered, when the regular filtration is renewed. When the removal of sand has reduced the thickness of that remaining to 12, or better 15 inches, new sand must be placed upon the filter. The lower the limit of head of water the more often must the filter be cleaned; and aside from the expense, this is objectionable because of the decrease in efficiency for some time after cleaning, and the waste of water, the quantity of which may be already deficient. For these reasons Hazen recommends placing the limit of depth of water in the filter at 5 or 6 feet, believing that no harm to the filter or its top coating will be occasioned by this if the *rate* be not increased above the limit. It is probable that this high limit is most applicable to the finer sands, which furnish a more solid and continuous support for the surface film; but that the pressure exerted might carry this down into the pores of a coarse sand. It would therefore seem necessary that each filter determine for itself the maximum head allowable; the head from time to time, while less than this, being kept sufficient to maintain the rate of percolation constant.

The following table gives the rates of filtration in several European plants, the majority being between 1,600,000 and 2,700,000 gals. per acre per day. As stated above, the latter is thought to be the highest desirable for ordinary conditions.

Oxygen is required for the oxidizing of organic matter in the water, and sufficient for this purpose is ordinarily contained in the water itself. If this be very impure and thus demand much oxygen, or if it for some reason contain little or no oxygen, this is generally supplied by allowing the water to drain out of the filter at short intervals of time, and the pores to become filled with air. Water is then turned onto the filter from above and the air in the sand interstices supplies the necessary oxygen. This method, called intermittent filtration, is used at Lawrence, Mass., and is that adopted in many cases for sewage purification. It will probably be required for few river-waters, however.

TABLE No. 63.

RATES OF FILTRATION IN EUROPEAN CITIES. (MASON.)

Place.	U. S. Gallons per Acre per Day.	U. S. Gallons per Square Foot per Day.	Vertical Inches per Hour.
Berlin (Tegel)	3,179,880	73½	5.0
Oporto	13,895,640	319	21.3
Zurich	7,492,000 to 10,672,000	172 to 245	11.5 to 16.4
Stuttgart	2,134,440	49	3.3
Altona	2,613,525	60	4.0
Liverpool	2,613,525	60	4.0
Chelsea	2,178,000	50	3.3
East London	1,655,280	38	2.5
Grand Junction	2,570,000	59	4.0
Lambeth	2,700,000	62	4.1
New River	2,570,000	59	4.0
Southwark	1,873,000	43	2.7
West Middlesex	1,655,280	38	2.5

Since filter-beds must occasionally be put out of service to be cleaned, and should remain so until, after several days of slow filtration and wasting of the effluent, its efficiency again becomes normal, duplicate beds are necessary for furnishing a continuous flow. If but two beds be employed, the capacity must be double that required for constant use; if three be used, one and a half times; and if ten beds be used, nine of which can filter the entire supply at any time while the tenth is being cleaned, their total area will be but 11% greater than that required for constant use. This, however, assumes that each bed will filter without cleaning nine times as long as is required for cleaning it and restoring its efficiency. If this latter be taken as seven days, then each bed must require cleaning but five or six times a year. With many filters this is satisfactory, although others may require cleaning once a week during certain seasons. The frequency of scraping, however, is a function of the quantity filtered and not directly of time. In Europe the quantity filtered between scrapings ranges between 25 and 160 million gallons per acre. The time is reduced by the sediment carried by spring floods, and the algæ occurring in summer and autumn. It will in many cases be found more economical, and in most will yield better results, to remove 50 to 75 per cent of suspended matter from muddy water by sedimentation, before filtering. By this means the area of filters required may be reduced 50% or more. The area will be $\frac{\text{quantity of water}}{\text{rate of filtration}} \times$ (1 + proportion of the area required for continuous use which is out of service during cleaning, which may be .10 to .15 or even .50).

The rate of filtration which it is desired to maintain is obtained by regulating the head; and although this will vary with the state of clogging due to sediment, its amount is primarily dependent upon the resistance offered by the sand and gravel. It is desirable to know the amount of head which will be required, not only as a guide in the use of the filter, but to permit of intelligent designing. The frictional resistance of sand to water, when there is no clogging, is found to be such that

$$v = cd^2\frac{h}{l}\left(\frac{t + 10^\circ}{60}\right),$$

in which v is the velocity of water in meters per day, (= 1,071,576 gals. per acre per day); $c = 1000 \pm$; d is the effective size of sand in millimeters; h is the head; l is the thickness of sand through which the water passes; t is the degrees Fahrenheit. This law seems to hold good only when the uniformity coefficient is below 5 and the effective size is between .01 and 3 mm. The rates for sands and gravels of different sizes are given in the following table:

Table No. 64.

RATE OF PERCOLATION THROUGH DIFFERENT SANDS, WITH VARIOUS HEADS, AT 50° FAHR.

(Compiled from Mass. State Bd. of Health, Report for 1892.)

$\frac{h}{l}$	Effective Size in Millimeters, 10 Per Cent Finer than															
	0.10	0.20	0.30	0.40	0.50	1.00	3.00	5.00	8.00	10.00	15.00	20.00	25.00	30.00	35.00	40.00
	Rate of Flow in Meters per Day.															
.0005	.005	.02	.045	.08	.125	.50	4.50	10	20	30	50	80	110	150	200	250
.001	.01	.04	.09	.16	.25	1.00	9.00	21	41	58	100	148	205	275	370	450
.002	.02	.08	.18	.32	.50	2.00	18.00	40	78	110	190	275	370	480	590	710
.004	.04	.06	.36	.64	1.00	4.00	36.00	77	150	208	350	480	610	740	870	1000
.005	.05	.20	.45	.80	1.25	5.00	45.00	94	178	241	400	550	695	835	980	1120
.006	.06	.24	.54	.96	1.50	6.00	54.00	112	207	275	450	620	780	930	1090	1240
.008	.08	.32	.72	1.28	2.00	8.00	72.00	142	252	340	530	720	900	1090	1270	1450
.010	.10	.40	.90	1.60	2.50	10.00	90.00	173	300	385	610	830	1030	1220	1410	
.015	.15	.60	1.35	2.40	3.75	15.00		238	378	480	760	1030	1260	1480		
.020	.20	.80	1.80	3.20	5.00	20.00		300	467	580	890	1180	1470			
.030	.30	1.20	2.70	4.80	7.50	30.00		400	615	750	1110	1450				
.050	.50	2.00	4.50	8.00	12.50	50.00		560	885	1060	1490					
.100	1.00	4.00	9.00	16.00	25.00	100.00		930	1310	1550						
.500	5.00	20.00	45.00	80.00												
1.000	10.00	40.00	90.00	160.00												
2.000	20.00	80.00	180.00													

If, for instance, a flow of $2\frac{1}{2}$ meters per day (2,679,000 gals. per acre) be desired through 18 inches of sand of effective size of 0.40 mm., 6 inches of gravel of 1 mm., and 6 inches of 3 mm., the total head above the sand required will be $.023 + .0025 + .0005$ feet $= .026$ feet. This head is the minimum, and will apply only for new filters, or those just cleaned. As the surface clogs the head will increase. With any given amount of clogging, the discharge can be increased in any proportion by increasing the head by the same proportion. In addition to this head is that necessary to cause the flow of the effluent through the drains into the clear-water basin.

The sand used in European filters has effective sizes varying between .17 and .44 mm., the former being used in Holland. In the majority of plants, however, the size is from .31 to .40 mm., and this may be taken as the range of the most practicable sizes. Finer sands remove a larger percentage of bacteria, and may be advisable for water always clear; but they will clog quickly with ordinary freshet river-water and require too frequent scraping. The coarse sand, on the other hand, will permit the sediment to penetrate too far into the bed, and hence require too thick a layer of sand to be removed; and their efficiency in removing bacteria will not be so great. There is also danger that the rate of filtration will, through carelessness, be permitted to become too great.

The under-drains should be of such size, grade, and frequency that they are never full at any point. They are usually made of sewer-pipe, with joints open or wrapped with burlap or cheese-cloth, and laid in slight depressions in the bottom of the filter. The bottom of one London filter is entirely covered with 3-inch drain-tile; while in another the drains are formed of brick on edge with a covering of brick laid flat and close together, no mortar being used. The latter method was used at Ilion, N. Y.

Fig. 41 shows a section of the Albany filter-beds, of which there are eight, each 258 feet by $121\frac{1}{8}$ feet; all being

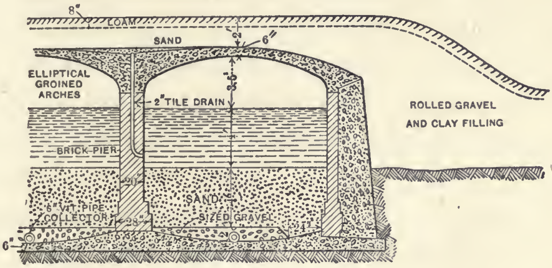


FIG. 41.—SECTION OF ALBANY FILTER-BEDS.

roofed over by elliptical groined arches of concrete, supported by 680 brick piers.

There should be, connected with the outlet of each filter-bed, a weir or other appliance for the discharge, and a method of regulating this or the supply to each bed. The method of regulation commonly employed is to alter the available head by raising the surface of water at the outlet, rather than to regulate at the inlet to the filter. Thus the water-surface on the filter remains at a constant height, while by raising or lowering a weir in the effluent chamber the head and consequent velocity are decreased or increased. An automatic appliance has been used in which the weir, being attached to a float in the effluent chamber, is kept at a constant distance below the surface of the effluent water, which falls and thus increases the head on the filter as this becomes more choked. The regulating appliance must be protected from freezing.

The walls and bottom of a filter should be water-tight to prevent either the loss of water or the access of impure ground-water. These have been made of all substances of which reservoirs have been constructed. In fact, a filter is very similar to a distributing-reservoir whose bottom is covered with a system of drain-pipes, and these in turn with gravel and sand. The best filter-walls, however, are vertical and formed of rough stone masonry.

Where the mean temperature for any month of the year falls below 32° more than occasionally, or where thick ice forms and remains for a week or more at a time, filters should be covered; since ice upon a filter is almost sure to diminish its efficiency, and has been known to cause typhoid fever. The expense of removing ice from a filter, also, may be considerable; this being $2323 for the Lawrence filter in 1897 (2½ acres, uncovered). On an average the cost of constructing covered filters in England has been 50% greater than that of uncovered ones of the same capacity. The expense of operating open filters is increased not only by ice, but by algæ growth, which rapidly clogs the filter. For roofing the filter, masonry, covered with earth, is preferable as excluding both cold and heat. The form of roof may be the same as for reservoirs (see Art. 74).

The first filter-beds in the United States to be covered with vaulting are thought to have been built at Ashland, Wis., in 1895–6, and cover an area of about one half acre, the roof being of groined elliptical arches of about 15¾ feet span and 3½ feet rise. At Somerworth, N. H., another filter-plant with arched masonry roof was constructed in 1897–8, the area being 148 × 150 feet, in two beds. At Grand Forks, N. Dak., in 1894, a sand filter 175 × 102 feet was constructed with a timber roof, 2-inch plank with tarred roofing-felt, pitch, and gravel.

The sand removed from a filter may be used for filling low land, or may be washed and used again, according to the relative value of filling material and cost of new sand as compared with that of washing the sand. The washing is done in various ways; in some plants a rotary washer like a pug mill is used; in others the sand is simply spread out and washed with a hose. An unsuccessful attempt was made at Hudson, N. Y., to wash the sand while on the bed by forcing a current of water up through it. At Albany it is intended to wash the sand in hoppers through which jets of water pass upward, carrying the sand with it, which water, overflowing into the drain, removes the dirt. From 0.5 to 1 per cent of the water filtered is generally required for washing the sand. The use of a mechanical agitator of some kind has been found to reduce the amount of water which must be used and the cost.

The sand on the top of a scraped bed is partially clogged, and when clean sand is placed on this there is a tendency for sediment to collect at the surface of the old sand where it cannot be readily removed and thus the entire filter to be clogged. For this reason the sand is renewed at as long intervals as possible, and the last scraping before renewal is made unusually deep. The top of the old and a layer of the new should then be thoroughly mixed before the whole of the new is placed. In England it is the general custom to take out the old and, placing the new sand in the bottom, replace the old upon it.

Filters are in some cases placed so that water will flow upon them from a river, reservoir, or other body of water by gravity; but in a majority of cases the water is pumped onto the filter, and the effluent water pumped into the system. The two sets of pumps being generally in the same building and using the same boiler-plant, the efficiency of the plant is not greatly decreased by the double pumping.

The efficiency of English sand-filtration is thought to be exceeded by that of no other system. The London filters are reported as removing an average during three years of 97.6% of the bacteria. The Altona, Germany, filters during February 1893 removed 99.69% of the bacteria.

It is apparent that if very muddy water were to be handled an English filter would quickly become clogged, and it might be necessary to have two or three times the service area to permit of frequent cleaning. In fact the Cincinnati tests appeared to show that at times the Ohio River water could not be rendered satisfactory in appearance and character by these filters alone. "So far as the information goes it appears that an average of 125 parts per million is a conservative estimate of the amount of suspended matters in the unsubsided river-water, which could be regularly and fairly satisfactorily handled by English filters"; and this amount was exceeded during 230 days in 1898. While they can handle this amount of silt in untreated water, experience seems to show that in subsided water they can remove the suspended clay in amounts ranging only as high as from 30 to 70 parts, and averaging about 50 parts per million. This is because the total matter in suspension in the subsided water is fine clay, while in raw water about one half of the suspended matter is in larger particles. It was found at Cincinnati that if the water be first clarified by sedimentation for two or three days and 75% of the suspended matter removed, English filters would have worked satisfactorily but 231 days of the year, or 64% of the time. For such rivers, therefore, these filters are not applicable as now used. But it must be remembered that the Ohio at Cincinnati is exceptionally muddy at certain seasons, and these objections will not be found existing to such an extent in a majority of cases.

If a small quantity of alum, or sulphate of alumina, be added to a water, the alum is decomposed into its component sulphuric acid and alumina, and the sulphuric acid combines with lime, magnesia, or some other base in the water; the alum forms a hydrate of alumina, which settles through the water in a gelatinous mass, entangling with it any matters in suspension, and also causing chemical changes not well understood by which dissolved organic matter and color are removed from the water. This method is used in a large number of sewage-purification works under the name of chemical precipitation. It has been used in a few cases as a preliminary to treatment by English filters. It was found in the Louisville experiments that a part of the alum was not available as a clarifier, but was absorbed by the suspended matter. The exact amount so lost under different conditions was not accurately learned, but with 400 parts per million of suspended matter about one half grain of sulphate of alumina per gallon of water was so absorbed. It was also found that, by first permitting the larger particles to settle out by plain sedimentation for a day, not only was the amount of coagulant necessary for use reduced, but the results obtained were uniformly better. In some instances this threefold process may give the best results, sedimentation removing the larger particles, precipitation by the coagulant much of the finer matter in suspension, and finally filtration through sand removing most of the remaining matters in suspension, including bacteria, and much of the organic matter in solution. In many cases an English filter alone would work satisfactorily for three fourths of the time; and for the remaining season of muddy water, if preceded by sedimentation and precipitation; the use and expense of alum thus extending over but one fourth of the year.

The main objection to the use of alum is, that if there be an insufficient amount of lime or other carbonate in the water to serve as a base, the alum is not all dissolved and a part of the sulphuric acid remains free; but this can be corrected by the addition of soda-ash or lime to the water. If ordinary care be used, the amounts of acid or of alum in the effluent will be so insignificant as to be harmless, one part of alum in 100,000 to 10,000 of water being the ordinary range of use;

or $\frac{1}{2}$ to 5 grains per gallon. By each grain of sulphate of alumina approximately half a grain of carbonate of lime is converted into sulphate of lime, 0.2 grain of carbonic acid being liberated, and these pass off with the filtered water. Perhaps the greatest objection to the sulphate of lime is its tendency to form a hard scale in boilers; while the carbonic acid facilitates the corrosion of unprotected iron.

Art. 78. Mechanical Filters.

The descent through water of the hydrate of alumina and the matters coagulated by it is extremely slow, and to hasten the process of purification the plan has been adopted of filtering out the coagulated matter, and also passing the water through it and causing it to act as a filter. Some support for the coagulant must be provided, and a bed of sand is used for this purpose, the coagulant forming a layer upon and in the top of this. This process is similar to English filtration, the alumina hydrate taking the place of the naturally formed Schmutzdecke. By increasing the amount of alum the coagulation and also the thickness of this coating and its filtering power can be increased; and it has been found possible to filter water at fifty to one hundred times the rate considered possible for English filters, or from 50 to 300 million gallons per acre daily. Since the amount of suspended matter removed is proportional to the amount of water purified, it follows that the clogging of such filters must be fifty to one hundred times as rapid as in the case of English filters. Moreover, the high velocity carries the sediment further into the sand-bed, and after running for a comparatively short time it will be carried entirely through into the effluent. It is therefore necessary not only to clean the filter at short intervals, but the entire bed of sand must be cleaned each time and not the top layer only. Practically the only material difference between the various mechanical filters—as these high-rate filters are called—are the methods of cleaning the sand and of applying the coagulant. Isaiah S. Hyatt, on Feb. 19, 1884, patented the process of purification by the use of alum or other coagulants which would "obviate the necessity of employing settling-basins." After nine or more years of litigation by various companies concerning the validity and scope of this patent the most of these have now (August 1899) combined in the New York-Jewell Filter Company. The general design of American filters as now manufactured is shown in Fig. 42. The water, the coagulant having first been added, is ordinarily admitted at the top and forced by gravity or pump pressure through the filtering-coke and sand and into the outlet-pipe. When the filtering material is to be cleaned, water, either purified or raw (preferably the former), is admitted through the outlet-pipe and forced up through the sand, agitating this thoroughly and washing out the collected sediment through a waste-pipe. This washing is continued until the effluent runs clear. In some filters the sand is agitated by revolving arms while the washing continues. When the filtering material is clean, water is again admitted from the top and filtering renewed. The head necessary to force water through the filter at the desired speed varies from 6 inches to 6 feet, depending upon the rate, the fineness of the sand used, and the state of the filter as to clogging. This head is obtained either by gravity, or by pumping into closed cylinders which contain the filtering material. The latter plan is more frequently adopted, and is that shown in the illustration, page 306.

Tests of the efficiency, best methods of operating, and cost of mechanical filtration have been made at Providence, R. I., in 1893, at Louisville, Ky., in 1895–6, at Lorain, O., in 1897, at Pittsburg, Pa., and Cincinnati in 1898, and at East Providence, R. I., in 1899. Allen Hazen was chemist and bacteriologist for the Lorain and Pittsburg tests, Geo. W. Fuller for those at Cincinnati and Louisville, and Edmund R. Weston for those at Providence and East Providence. From

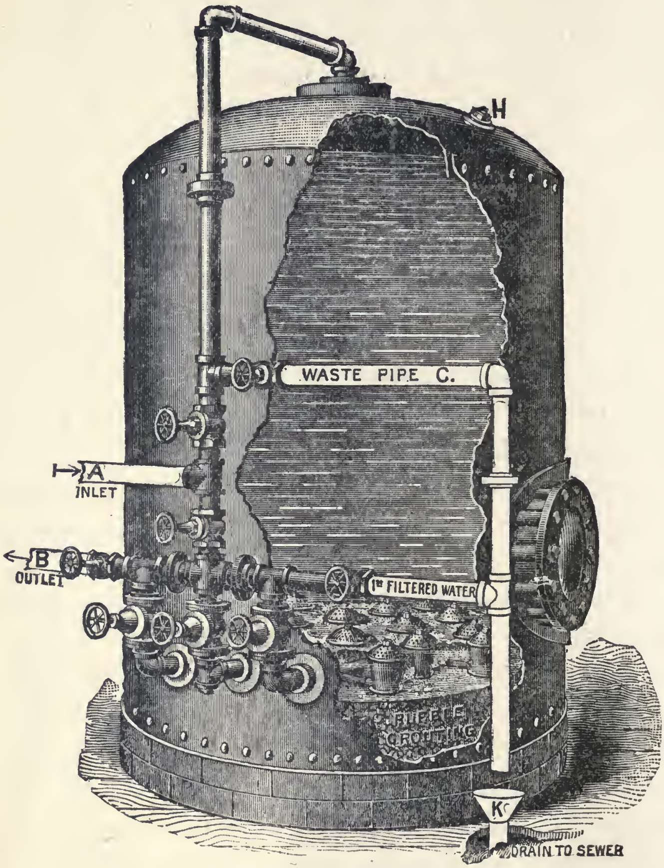


FIG. 42.—AUTOMATIC PRESSURE-FILTER.

these a large amount of valuable information was obtained, the more important points of which are summarized below.

In the Providence test 92 to 99 per cent of the bacteria were removed when the filtering was at a rate of 128 million gallons per acre daily; in the East Providence test 96.56 to 100 per cent were removed, the average for 3½ months being

99.24%, when filtering at the rate of 125 million gallons per acre daily and using one grain of alum per gallon as a coagulant.

In the Lorain test of five weeks' duration the bacteria removed when using 2.45 grains of alum per gallon were from 97.5 to 98.9 per cent of those in the raw water; with 1.07 grains of alum per gallon the bacterial efficiency fell to 90.9%; and with 0.89 grain the efficiency was but 86.3%; the rate of filtration being from 67 to 80 million gallons per acre daily.

In the Pittsburg test the bacterial efficiency varied from 93.23% when 0.56 grains of alum per gallon were used and the rate of filtration was 98 million gallons, to 98.96% when using 1.36 grains of alum per gallon and filtering at the rate of 103 million gallons per acre daily; the average being 97.85% when using 1.18 grains of alum per gallon and filtering at the rate of 109 million gallons.

In the Louisville experiments filtration at the rate of 33 to 155 million gallons per acre daily, and the use of ½ to 12 grains of alum, gave widely varying results, there being at times 200 or even 300 bacteria per cubic centimeter in the effluent; but it was concluded from final tests that a bacterial efficiency of 99% was obtainable with careful management; and this opinion has been confirmed by the Cincinnati tests.

The clarification of the water in the Providence experiments averaged 80%, in the East Providence experiments 83%. In the Pittsburg experiments the average reduction in color was 88½%, and in turbidity 98½%. At Lorain the water had little color, turbidity, or sediment, all of which were removed by filtration.

In the East Providence experiments the "total solids" were reduced 6%; free ammonia, 29%; albuminoid ammonia, 63%; and the hardness was increased 20%.

At Pittsburg the solids were reduced 37½%; free ammonia, 6.6%; albuminoid ammonia, 48%; hardness, 5½%.

TABLE No. 65.

RESULTS OF TESTS OF MECHANICAL FILTERS AT PITTSBURG, LORAIN, AND EAST PROVIDENCE.

Bacterial Results.

LORAIN O.

Week ending	Rate of Filtration, Gallons per Sq. Ft. per Min.	Alum, Grains per Gallon.	Bacteria in Lake-water.	Bacteria in Effluent.	Bacterial Efficiency, Per Cent.
June 19 1897	1.06	2.58	1,441	16	98.9
" 26	1.10	2.50	385	6	98.4
July 3	1.11	2.27	367	9	97.5
" 10	1.28	1.07	154	14	90.9
" 17	1.14	0.94	189	26	86.3
Average	1.14	1.83	507	14	96.4

EAST PROVIDENCE, R. I.

Rate of filtration, 125 million gallons per acre daily. One grain of sulphate of alumina per gallon used.

Month of *	Bacteria per c.c. in		Bacterial Efficiency, Per Cent.		
	Raw Water.	Filtered Water.	Max.	Mean.	Min.
March 13 to 31	897	5.0	99.82	99.42	99.08
April 1 to 30	730	5.5	99.98	99.24	97.87
May 8 to 31	345	3.4	100.00	99.01	96.56
June 8 to 30	453	3.6	99.92	99.20	96.56
Average	570	4.3	100.00	99.24	96.56

* Special experiments on March 20 to 30, May 1 to 6, and June 1 to 7 and 22 to 28 are not included in the above. No tests were made on Sundays.

PITTSBURG PA.

Months of 1898.	Rate of Filtration, Millions of Gallons per Acre Daily.	Alum, Grains per Gallon.	Bacteria in River-water.	Bacteria in Effluent.	Bacterial Efficiency, Per Cent.
WARREN FILTER.					
January	115				
February	104	0.70	9,430	238	97.48
March	108	1.81	11,747	164	98.60
April	122	0.86	5,010	78	98.44
May *	115	1.55	10,800	630	94.20
June	103	1.36	11,100	115	98.96
July	119	1.46	16,800	320	98.10
August	137	1.76	15,100	290	98.08
Average	116	1.36	11,427	262	97.70
Average excluding May	......	... .	11,531	201	98.26
JEWELL FILTER.					
January	85				
February	98	0.56	9,430	638	93.23
March	106	1.07	11,747	208	98.23
April	106	0.54	5,010	159	96.83
May	106	1.01	10,800	150	98.61
June †	106	1.18	11,100	1450	86.90
July	103	1.31	16,800	345	97.95
August	103	1.35	15,100	260	98.28
Average	102	1.00	11,427	459	95.99
Excluding June	......	... .	11,481	293	97.45

* Special experiments were made during May.
† " " " " June.

TABLE No. 65.—(*Continued.*)

Chemical and Physical Results.

	Lorain, O.		East Providence, R. I.	Pittsburg, Pa.						
									Sand Filters.	
	Lake-water.	Ef-fluent.	Reduction by Filtration, Per Cent.	River.	Gate-chamber.	Warren Ef-fluent.	Jewell Ef-fluent.	Settling-Basin.	No. 1.	No. 2.
Color	0.05	None	83	0.26	0.26	0.03	0.03	0.24	0.07	0.07
Turbidity	Distinct	None		....	0.20	0.004	0.002	0.14	0.010	0.012
Free ammonia	0.0015	0.0014	29	0.0020	0.0022	0.0019	0.0018	0.0020	0.0018	0.0018
Albuminoid ammonia	0.0146	0.0076	63	0.0101	0.0100	0.0047	0.0044	0.0092	0.0054	0.0053
Nitrogen as nitrates	0.0170	0.0170		0.0568	0.0611	0.0512	0.0491	0.0581	0.0642	0.0549
" " nitrites	0.0003	Trace		0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Total solids	27.3000	23.0000	6	15.4	15.0	9.3	9.5	11.9	10.8	10.6
Volatile solids	12.0000	8.0000								
Chlorine	0.5800	0.6000	1	1.87	1.73	1.71	1.71	1.80	1.84	1.77
Oxygen consumed	0.2300	0.1300								
Temporary hardness	9.4000	7.4000								
Permanent "	1.6000	3.0000								
Total hardness	11.0000	10.4000	—20	3.21	3.31	2.92	3.03	3.27	4.31	4.33
Alkalinity				2.44	2.56	1.61	1.72	2.49	3.53	3.56

The results of the Pittsburg, Lorain, and Providence experiments are given more in detail in Table No. 65.

Concerning the amount of sulphate of alumina required, Hazen concludes from his Pittsburg experiments that the amounts given in the table on page 310 will be necessary for different percentages of bacterial efficiency and of turbidity.

Fuller, from his Cincinnati experiments, concludes that the desired purification can be obtained at that place by precipitation for 48 hours, followed by treatment with 1.6 grains per gallon of alum, filtration being at the rate of 125 million gallons per acre. He also fixes 10 feet as the maximum head on the filter desirable; and a depth of sand of 30 inches, having an effective size of .35 mm.

The amount of water required for washing the filter both Hazen and Fuller find to be about 5% of that filtered; and Hazen finds that, although the efficiency of the filter is reduced for about 20 minutes after each washing, the effect upon the filtrate is insignificant and this water need not be wasted.

SULPHATE OF ALUMINA REQUIRED FOR DIFFERENT EFFICIENCIES AND TURBIDITIES.

Bacterial Efficiency, Per Cent Removed.	Required Amount of Sulphate of Alumina, Grains per Gallon.			
	Least Turbid Waters.	Preceding Amount sufficient for Turbidities up to *	Extra Quantity for Higher Turbidities throughout the Year.	Average Quantity throughout the Year.
WARREN FILTER. (Effective size of sand, 0.63 mm.; uniformity coefficient, 1.1.)				
95	0.37	0.03	0.33	0.70
96	0.44	0.06	0.28	0.72
97	0.56	0.11	0.22	0.78
98	0.84	0.22	0.14	0.98
98.5	1.12	0.34	0.10	1.22
99.0	1.60	0.53	0.06	1.66
JEWELL FILTER. (Effective size of sand, 0.46 mm.; uniformity coefficient, 1.4.)				
95	0.42	0.07	0.18	0.60
96	0.49	0.12	0.15	0.64
97	0.65	0.21	0.10	0.75
98	0.96	0.39	0.06	1.02
98.5	1.48	0.70	0.02	1.50

* The coefficient of turbidity is the reciprocal of the depth in inches beneath the surface of the water at which a platinum wire .04 in. thick is just ready to disappear from view. See page 19.

It was found at Pittsburg that if the filtering material be occasionally washed by placing upon it 1 lb. of soda ash per square foot of surface and boiling the whole by applying steam through the bottom pipe, the filter is given new life, and the length between washings possible and quantitative efficiency are both increased.

The largest mechanical-filter plant is that at Wilkesbarre, Pa.—twenty tanks with a combined capacity of 10,000,000 gals. per day when run at a rate of 193 million gallons per acre per day.

Art. 79. Other Filtering Methods.

A few water-supply systems have combined intakes and filtration-plants, one such arrangement being that of "filter-galleries" described in Art. 42; while another, used mostly in the Allegheny River, consists of a crib placed in a river or lake and covered with sand. A section of such a filter-crib used at Kensington, Pa., is shown in Fig. 43. This is essen-

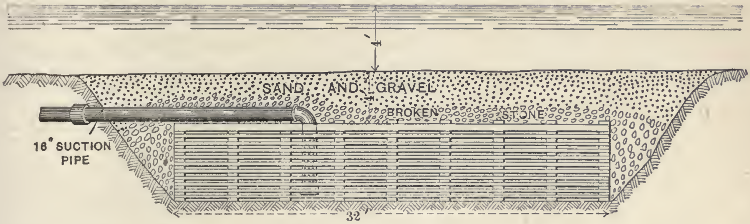


FIG. 43.—FILTER-CRIB; KENSINGTON, PA.

tially a sand filter, which is used at the rate of 16 million gallons per acre per day in the Kensington plant. No coagulant is used, of course, nor is its use possible, and it is doubtful if the Schmutzdecke forms on such a filter to any extent. A crib through which the Pennsylvania Water Company pumps Allegheny River water was found in 1897 to reduce the number of bacteria from an average of 3542 to one of 266 per cubic centimeter, and to generally decrease the albuminoid ammonia and nitrates; while the total solids were decreased from a maximum of 448 parts per million to 140 parts in the corresponding effluent. The main object, however, for which these cribs are used has been clarification rather than purification, and for this they seem fairly well adapted. Arrangement is usually made for pumping water into the crib and thus out through the sand for the purpose of cleansing it of sediment.

A method of mechanical filtration which was adopted about seven years ago at Worms, Germany, and has been recently advocated in this country, is by the use of hollow slabs or flat boxes of artificial porous sandstone. This stone is formed of a mixture of clean river-sand and silicate of lime and soda, which is baked in ovens. These slabs, which are

$3\frac{1}{2}$ feet square and 4 inches thick, rest on end, in the unfiltered water, which percolates through to their hollow interiors, these being connected with a clear-water pipe beneath them, into which pipe the water passes. The slabs are treated like English filters, and the same film forms upon their surfaces, the only advantage claimed being economy of space. At Worms the plates were placed in a covered filter which had previously been used for English filtration, and the quantitative efficiency of this area was increased eightfold. The filter-plates are cleaned by reversing the current or by scrubbing. Experiments made on these at Pittsburg "did not indicate that the requisite amount of pure water could be secured under all conditions"; and the first cost and operating expenses were considered excessive. The plates broke under a wash-water pressure of 20 feet.

About 1865 a similar plan for filtering through slabs of charcoal was devised, but found impracticable.

The removal of iron from ground-water has been effected in several ways. At Atlantic Highlands, Asbury Park, and Keyport, N. J., Reading, Mass., and other places mechanical filters are used, preceded by aeration by which the iron is oxidized into an insoluble compound. At Keyport and Reading lime is used as a coagulant, but no chemical is used at the other places named. The result seems satisfactory, although 8 to 10 per cent of the filtered water was required for washing the filters at Atlantic Highlands and Asbury Park in 1896.

Experiments with coke breeze (screenings from commercial coke) in 1896 at Provincetown, Mass., showed an efficiency of 93 to 98 per cent of iron removed when the water was passed through the breeze, arranged as a filter $3\frac{1}{2}$ feet deep, at a rate of one million gallons per acre daily. Aeration and filtration of the same water through sand removed but 20% of the iron, of which the water contained 0.495 parts per 100,000.

The Anderson process has been used in a number of places in Europe and in this country. It consists essentially in bringing the water to be purified in contact for three or four minutes with iron, preferably cast-iron borings or turnings, then aerating, settling, and filtering it. Contact with the iron is secured by passing the water through revolving cylinders containing particles of iron, the cylinders having shelves or ledges so arranged as to continually shower the iron through the water. The water is aerated just before entering the cylinders. Before leaving them it is said to be charged with a proto-salt of iron, which, by a second aeration, is changed into an insoluble ferric oxide. The water is then filtered. The ferric oxide in this process acts as a coagulant, but is not thought to be so effective as sulphate of alumina. From 0.1 to 0.2 grain of iron per gallon are claimed by the Anderson Company to be taken up by the water.

A modification of this method has been used at Wilmington, Del., since the latter part of 1894, in which bundles of revolving iron rods in a channel traversed by the water take the place of the Anderson cylinder, and the filtration is upward through sand-beds. As the iron yielded by the rods is less than .01 grain per gallon it is evident that this has little effect upon the result, which is due to aeration and sand-filtration alone.

So-called electrical purification of water is really but an electrolytical production of cogulant in the water to be treated; but the cost by any method yet adopted seems to be excessive, and the regular formation of coagulant not under control. On the other hand, aluminum and iron hydrates do not decrease the hardness, corroding or encrusting qualities of the water, as does sulphate of alumina; and it is possible that the electrical production of coagulents will yet be made a practical and commercial success.

The softening of hard water (see Art. 6) is a very important factor in preparing it for use in the laundry and for boilers, as well as in rendering it more potable. Temporary hardness can be removed (1) by boiling, when it forms a compact sediment; (2) by adding carbonate of soda, which combines with the bicarbonate of lime in the water, and bicarbonate of soda and carbonate of lime result. The former remains in solution but does not render the water hard, and the latter is precipitated as a fine powder. (3) By lime-water (a solution of freshly burned lime), which unites with a part of the carbonic acid in the carbonates of lime or magnesia, and reduces these and itself to insoluble monocarbonates which form a fine precipitate; which process is the least expensive. Permanent hardness can be removed by adding carbonate of soda, which, uniting with the sulphate of lime or of magnesia, forms an insoluble carbonate of these bases, together with sulphate of soda, which remains in solution but does not harden the water. When water is both permanently and temporarily hard, as is usually the case, a mixture of lime and carbonate of soda generally gives the best results.

At Southampton, Eng., of 25.7 parts of carbonate of lime in 100,000 parts of water, 18.7 parts are removed by adding 22 parts, by weight, of lime, which is first slaked and mixed with water to form a creamy fluid. After receiving the lime-water the water stands for an hour, and the insoluble lime is then filtered out; this being called the Clark process. The filtering has been rendered unnecessary by a recent improvement of Clark's method, by which the precipitate of a previously softened water, which has become aggregated into coarse flakes, is mixed with that just softened, and in settling carries down the fine particles of fresh precipitate much more rapidly than they would otherwise settle. Thus rapid precipitation takes the place of filtration at less expense and in less time. This modification of the Clark process, with other minor changes in application of the chemical and in the precipitation-tanks, is known as the Archbutt-Deeley process. (See *Engineering News*, vol. XL. page 403.)

In any process where chemicals are used it is necessary that some arrangement be made for admitting at all times the proper amount of chemical, which will vary with both the quantity and character of the water. Many methods of accomplishing this are in use, but few are satisfactory. In using lime, alum, and other solid chemicals, these are first mixed with or dissolved in a small amount of water, and this concentrated solution is then admitted to the water to be treated. If the character of the water is constant, the solution may be kept at a uniform strength, and the amount of this proportioned to the amount of water treated, by automatic connection with the pump delivering it or in some other way. If the quality of water vary, either the strength or quantity of the chemical solution may be correspondingly changed; the former being the plan generally adopted as the most reliable and least troublesome.

Distillation as a method of purification is not adapted to city supplies, because of both the great cost and the peculiar flat taste of distilled water which renders it objectionable to most persons. The United States Navy is probably the most extensive user of distilled water, this being the only supply on practically all its vessels; but it is there an alternative of stored water, which it would often be necessary to obtain from sources of doubtful purity.

Household filters are manufactured of many designs, some using chemicals, but most of them straining the water only. From what has been said it is evident that a chemical filter in particular needs careful attention to be efficient, and this it seldom receives in a household. Of the straining-filters the Pasteur is one of the best, porcelain being the filtering medium. But through even this bacteria have been known to pass, probably by a process of growth rather than by passing bodily through it. Hence these filters should be cleaned and boiled at least once a week. Many other filters become foul in much less time, and after use for six to twelve hours render the water passing through them vastly more polluted than if not "filtered." This result might be anticipated from the high rates at which they pass the water. It can be safely said that any filter which delivers water directly from the faucet, rather than filtering continuously and storing the filtrate, is much worse than no filter at all; and that, for a household of six, a continuously flowing filter without chemicals should have a surface area of at least 10 square feet, and with chemicals of 28 square inches.

Art. 80. Summary.

For softening water the Clark process forms the basis of all satisfactory methods in use.

For removing iron from water the method of aeration, and removing the resulting insoluble ferric oxide by filtration, or sedimentation, or both, is practicable and satisfactory if the details of the operation be properly conducted.

For removing clay and other inorganic matters, sedimentation followed by filtration gives good results, and when there is little matter to be removed the sedimentation may be omitted. For this purpose mechanical filters using a coagulant are better than slow sand (English) filters; but not so good as these if no coagulant be used.

For removing bacteria and other organic matter English filters give the best results; although mechanical filters with coagulation have given an efficiency only ½ to 2 per cent less. But the possibility of occasional serious lapses in full efficiency is greater with mechanical than with English filters.

When it is desired to remove two or more of the above classes of impurities, the choice of method requires more careful consideration. If the suspended matter never exceeds 100 to 125 parts per million, and seldom 75 parts, English filters are probably to be preferred for removing this and organic matter; but if more than this be present, sedimentation, followed in more extreme cases by coagulation and a short rest, are necessary before filtering with the English filter; or a mechanical filter may be used with coagulants. Which of these is preferable will depend on various local conditions, and characteristics of the water in question.

If the water contains very much fine clay, mechanical filtration preceded by sedimentation is probably the best solution.

If iron is to be removed in addition to suspended and organic matter, aeration may precede the other processes and the iron thus become part of the suspended matter to be removed.

Softening is generally best accomplished after other purification has been completed.

The following table gives some data concerning the filtration of water-supplies in the United States. It was compiled from the "Manual of American Water-works" for 1897 by Benj. H. Flynn, University of Ohio, class of '98, for a graduating thesis. (From *Engineering News*, July 7, 1898.)

Table No. 66.

FILTRATION AND SOURCES OF SUPPLY IN THE UNITED STATES.

		Number of Water-filtration Plants of Various Types.					Ground-water.							
		Filters.		Miscellaneous.*	Filter-galleries.†	Total.	Wells.	Springs.	Galleries and Tunnels.	Wells and Springs.	Wells and Galleries.	Springs and Galleries.	Wells, Springs, and Galleries.	Total.
		Mech.	Sand.											
Northeastern	Unfiltered ..						167	293	8	25	4	......	3	500
	Filtered	46	22	32	20	120	2	7	1	1	0	......	0	11
	Total						169	300	9	26	4	......	3	511
Southeastern	Unfiltered...						100	71	......	8	......	......	....	179
	Filtered	21	2	3	4	30	0	1	......	0	......	......	....	1
	Total						100	72	......	8	......	......	....	180
North-central	Unfiltered...						407	25	11	13	5	1	....	462
	Filtered	24	6	12	20	62	1	2	1	0	3	0	....	7
	Total						408	27	12	13	8	1	....	469
Western......	Unfiltered...						521	103	13	11	8	1	1	658
	Filtered	35	9	19	16	79	4	0	0	0	0	0	0	4
	Total						525	103	13	11	8	1	1	662
Grand total ..	Unfiltered...						1195	492	32	57	17	2	4	1799
	Filtered	126	39	66	60	291	7	10	2	1	3	0	0	23
	Total						1202	502	34	58	20	2	4	1822

* Miscellaneous includes gravel screens and embankments and all supplies reported as filtered without stating the method.
† Filter-galleries are not included among the filtered supplies in the rest of the table.

TABLE No. 66—(*Continued.*)

		Surface-water.						Surface- and Ground-water.												
		Rivers.	Lakes.	Impounding-reservoirs.	Rivers and Lakes.	Rivers and Impounding-reservoirs.	Total.	Rivers and Wells.	Rivers and Springs.	Rivers and Galleries.	Rivers, Wells, and Impounding-reservoirs.	Rivers, Wells, and Springs.	Rivers, Wells, and Galleries.	Lakes and Wells.	Lakes and Springs.	Lakes, Wells, and Springs.	Impounding-reservoirs and Wells.	Impounding-reservoirs and Springs.	Total.	Grand Total.
Northeastern	Unfiltered ..	276	122	128	12	3	541	19	53	2	1	3	2	4	4	1	6	1	96	1137
	Filtered.....	60	7	7	0	0	74	8	4	0	0	0	0	0	2	0	2	0	16	101
	Total	336	129	135	12	3	615	27	57	2	1	3	2	4	6	1	8	1	112	1238
Southeastern	Unfiltered...	97	5	16	....	1	119	7	7	....	....	1	....	....	....	....	....	1	16	314
	Filtered.....	19	1	4	....	0	24	1	0	....	....	0	....	....	....	....	....	0	1	26
	Total	116	6	20	....	1	143	8	7	....	....	1	....	....	....	....	....	1	17	340
North-central	Unfiltered...	91	58	9	2	....	160	28	4	....	....	3	1	11	....	4	....	....	51	673
	Filtered.....	26	4	3	0	....	33	1	0	....	....	0	0	1	....	0	....	....	2	42
	Total	117	62	12	2	....	193	29	4	....	....	3	1	12	....	4	....	....	53	715
Western......	Unfiltered...	220	22	37	0	2	281	32	16	2	....	2	1	3	....	....	4	2	62	1001
	Filtered	36	2	9	1	0	48	6	0	1	....	1	0	1	....	....	1	0	10	62
	Total	256	24	46	1	2	329	38	16	3	....	3	1	4	...	....	5	2	72	1063
Grand total ..	Unfiltered...	684	207	190	14	6	1101	86	80	4	1	9	4	18	4	5	10	4	225	3125
	Filtered	141	14	23	1	0	179	16	4	1	0	1	0	2	2	0	3	0	29	231
	Total	825	221	213	15	6	1280	102	84	5	1	10	4	20	6	5	13	4	254	3356

QUERIES.

24. What cross-section and length of basin would be required for sedimentation by continuous flow if the consumption were one million gallons per day and the water were required to remain 48 hours in the basin? What capacity for perfect-rest basins?

25. If the law of frictional resistance of sand to water holds good for clay sediment, what is the effective size of clay particles on a certain filter-bed if the clay sediment, when formed 2 inches deep on and in the top of the bed, requires an additional head of 4 feet to force 2,000,000 gals. per day per acre through the filter?

26. Judging from the Pittsburg experiments, which of the two mechanical filters tested would be best for only slightly turbid waters? which for water having a coefficient of turbidity of 0.50?

CHAPTER XIV.

PUMPING AND PUMPING-ENGINES.

ART. 81. PUMPS.

THE pumps in common use are either reciprocating or rotary. The former consists essentially of a water-chamber in which a closely fitting plate or a cylinder moves back and forth. In Fig. 44, B represents such a plate (called a piston)

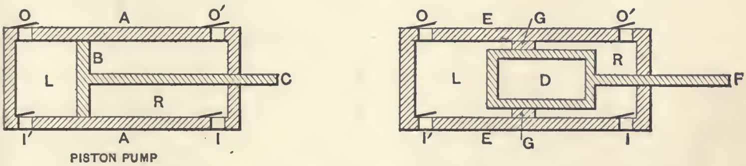


FIG. 44.—PISTON- AND PLUNGER-PUMPS.

moved by a piston-rod C in a water-cylinder A; and D a plunger (made hollow to reduce the weight) moving through a closely fitting ring GG in the cylinder E. Each cylinder is provided with openings for admitting the water (I, I') and others for its exit (O, O'), which are opened and closed by valves. As B or D advances to the left the water in L is driven out through the discharge-valve O and sucked in through the suction-valve I, O' and I' being closed by water pressure; when B or R moves to the right water enters through I' into L and is driven from R through O'. I and I' connect with a suction-pipe, O and O' with a discharge-pipe. If the surface of the water to be pumped is below the cylinder, a vacuum is created in this by the piston or plunger and the water is forced in through the suction-pipe and valves by atmospheric pressure. A pump so placed is called a suction-pump. But if the head above the cylinder be sufficient to force the water into it by gravity and the pump merely raises the water higher than itself, it is called a lift- or force-pump. Most pumps are combined suction and lift. If the pump has two sets of valves, as shown in Fig. 44, so that water is discharged during motion in each direction, it is called a double-acting pump. If the water is discharged while the piston or plunger moves in one direction only, it is called single-acting. The ring *GG* is called the bushing or plunger-sleeve. *C* is the piston- and *F* the plunger-rod, which pass through stuffing-boxes in the cylinder-ends by which leakage around the rod is prevented.

If a piston-pump cylinder stand upright, and a valve be placed upon the top of the piston opening upward, as in Fig. 45, this is called a bucket-pump. In this, water enters *I* and is discharged through *O*, *O* on the up-stroke; on the down-stroke *I* closes and water passes through the valve *V*, which closes on the up-stroke and raises the water which is above it. If the piston-rod *C* be of considerable size, water will flow from *O*, *O* during both down- and up-stroke and the pump will be a continuous-discharge pump. Bucket-pumps are generally used for deep-well pumping.

Upright cylinders on large engines frequently take their water through the bottom of the cylinders only, since these are 6 feet or more in height, and the upper suction-head would be greater than the lower by a large part of this amount. These are made double-acting by supplying the water-cylinder with a differential plunger, Fig. 46. In this the plunger area, *d*, is about half that of the cylinder, *a*. When the piston descends the water in *c* is forced into *b*, and half passes out into the mains while the other half enters *a*. On the up-stroke this latter is forced through *b* into the main, and *c* is at the same time filled from the suction-pipe. The discharge at each stroke is hence equal to that from a double-acting pump-cylinder of half the area.

The force required to pump is that necessary to raise the water by suction, to raise it by pressure or lifting, to overcome the inertia of all the water moved, to overcome the friction in the conduits, and to overcome the friction and

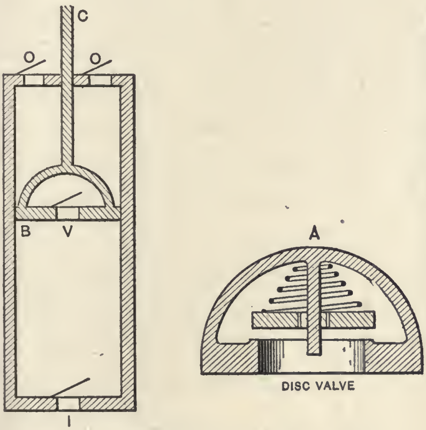


FIG. 45.—BUCKET-PUMP AND DISC-VALVE.

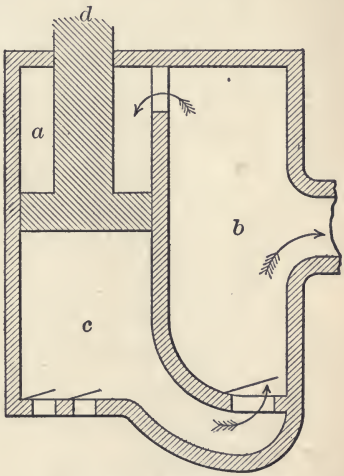


FIG. 46.—DIFFERENTIAL PLUNGER.

inertia of the moving parts of the pump and of water in passing through the pump. That required in suction and lifting is the product of the weight of water lifted by the distance through which it is lifted, and cannot be altered by any mechanical device. The inertia of the moving water depends upon the mass of water to which motion must be imparted and the velocity of such motion. The latter can be decreased by enlarging the size of the pipe, pump-cylinder, and other passages which the water must traverse. Friction is decreased also by decreasing this velocity. If the pump so acts that at the end of each stroke all the water in the mains comes to rest, and must be given motion again by the next stroke, the energy required to overcome the inertia is considerable. If the discharge of water could be continuous, there would be after the first few strokes no inertia to overcome except that of the water being delivered by the pump. Various arrangements are used to obtain this end, the most common being to so couple two or three double-acting pumps together that one is discharging its maximum when the other is at or near the end of its stroke (such an arrangement being called a duplex or triplex pump); to place an air-cylinder on the discharge-pipe near the pump, in which the air is compressed during the time of greatest piston-motion, and expands during that of least, thus equalizing the pressure and consequent velocity in the mains; or by the use of a fly-wheel, in which the energy is stored during mid-stroke to be given up by its inertia at the ends of the stroke.

To reduce the power required to overcome inertia in the moving parts of the pump these must be light in weight, and their motion slow. Their friction is a matter of detail in design, methods, and materials of construction, and of careful maintenance. The friction of water in passing through the pump may be considerable, but can be largely reduced by making the connections between the cylinder and the suction- and force-pipes of ample size and easy curves, the valve-openings large and with no angles and well cleared by the valves, and by introducing in the path of the water as few obstructions as possible. Another loss, called slip, exists to a certain extent in all pumps and is the water which escapes back through the valves while they are closing and around the piston or plunger, and hence is practically lost after being pumped. To reduce this, which sometimes equals 5% or more of the water pumped, the valves should close rapidly, and the piston must fit the cylinder, or the plunger its ring, as closely as is possible with little friction. (In the illustration the valves are shown as flap-valves; but in high-class pumps disc-valves are used, or discs of metal or hard rubber which rise bodily, and are generally closed quickly by springs; as at *A*, Fig. 45.)

The pump should of course be substantially built in all its parts; should be so constructed that contraction and expansion by heat will not affect its working; should be durable, all its moving contact-surfaces being automatically lubricated; and should be provided with hand-holes and facilities for reaching all parts and renewing them when worn or injured.

Next to reciprocating pumps, rotary pumps are used most extensively for raising water, the most common of these being the centrifugal pump. The principle used in this pump, as the name implies, is that of the centrifugal force in a revolving body. Its essential parts are a circular hollow casing or "shell," in which revolve a set of curved arms which fit closely the inner space; the shell being open in the centre on one side, as at *I*, Fig. 47, and on the circumference at one

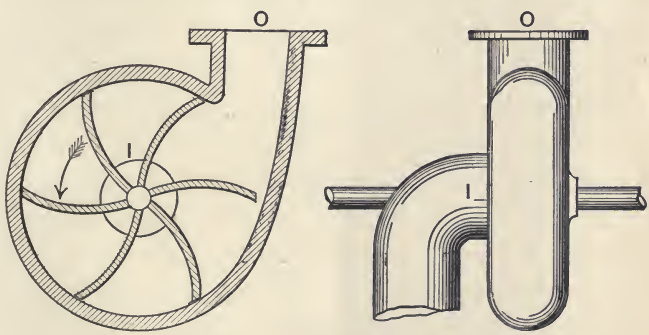


FIG. 47.—CENTRIFUGAL PUMP.

point, as *O*, each opening forming the end, *I* of a suction-, *O* of a discharge-pipe. The pump being full of water, the vanes (or piston) are revolved; and the water, seeking the circumference, is forced out through *O*, and other water to take its place enters through *I*, and this operation is continuous as long as the piston revolves. The flow here being continuous, there is no inertia of water in the pipe or of moving parts to be overcome after the pump is in motion; the friction in the pump, also, need be but slight; but the slip is considerable and may become almost or entirely 100% if the velocity of revolution be too low, the head to be pumped against too great, or the vanes, or wings, and the casing too much worn. Such pumps are subject to little wear, however, unless the water be very dirty. One of their disadvantages is the limited range of rate at which they can pump, since the velocity must be considerable to prevent slip. They are particularly adapted to raising large volumes of water short distances, and for sediment-bearing water. Under favorable conditions they are capable of high efficiency.

Other pumps have been used acting upon the principle of the screw of Archimedes, or consisting of chains of buckets, and several other devices; but it is not thought necessary to consider these in this general discussion of pumping machinery, nor to go into a consideration of the details of pump construction. To such a high state of efficiency has the design and construction of pumps been brought by American manufacturers that, except for the most important engines or for unusual conditions, they may be trusted to meet any stated requirements most satisfactorily. It is desirable, however, that the engineer be able to ascertain whether these requirements have been met. The principal ones are the general solidity and durability of the pump, and its efficiency. The solidity requires a proper adjustment of the strength of each part to the work it is to perform; and that the general weight of the engine and its foundation, and the smoothness of running, be such that no vibration be peceptible. Durability demands these conditions and also that the material be of the best—castings of the best gray iron with no flaws, steel tough and fibrous; and rubbing-surfaces made of metals giving little friction, generally Tobin bronze or gun-metal.

The efficiency of a pump is the relation between the power applied to it and the work performed by it in pumping water, or $\frac{\text{work done in pumping}}{\text{energy applied}}$. Each is generally expressed in foot-pounds, and the efficiency is hence an abstract number. The work done is ascertained by multiplying the weight of water by the entire distance through which it is raised, including both suction and lift, and adding the friction in all pipes or that due to any other causes outside of the pump. To allow for this outside friction the head is generally taken as that indicated by the sum of the pressure in the discharge-pipe and vacuum in the suction-pipe, both referred to the same level. That is, if the vacuum-gauge on the suction-pipe is three feet below the pressure-gauge on the delivery-pipe, and these two read 20 feet and 100 feet respectively, the total head of 123 feet is used in the calculation. The energy applied is measured by a transmission-dynamometer, or some other contrivance whose character is adapted to the method of applying the power.

ART. 82. PUMPING-ENGINES.

Rotary pumps are generally driven by belting or gearing from a steam or other engine, or by water or electric motors mounted upon the shaft of the pump.

Reciprocating pumps may be driven either by belting, gearing, or motor, when they are called power-pumps; or they may be direct-acting, that is, the piston-rod of the pump may have upon its other end another piston working in a cylinder and actuated by steam or other medium; or the power may be applied by an engine through the medium of a crank-shaft supplied with a fly-wheel.

Water is a practically non-elastic fluid, and "while this is met in the direct-acting type of steam-pump by the elastic steam-cushion, the load in the power-pump is received fully on the crank-pin as it passes the centre. Therefore in the power-pump the bed-plate must be so constructed as to withstand this excessive strain and vibration. The foundations must also be of the most substantial character, all working parts must be of unusual strength, and suitable means of adjustment to compensate for wear must be provided." (Chas. L. Newcomb.)

In the direct-acting pump, *B*, any shock communicated by the water, stoppage in the water-end, or break in the

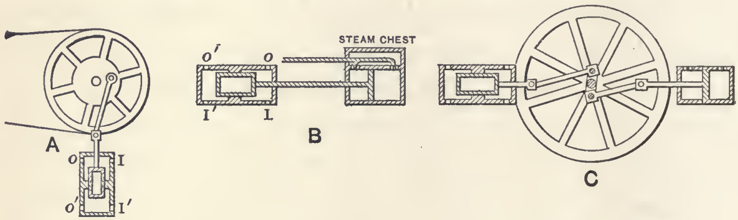


FIG. 48.—POWER, DIRECT-ACTING, AND FLY-WHEEL PUMPS.

delivery-pipe and consequent release of pressure, is largely taken up by the steam acting as a cushion in the steam-cylinder. In the case of the fly-wheel pump, the wheel takes the shock and little is felt by the motive engine; but the length of stroke of the steam-end is fixed and this is for some reasons an objection. In the power-pump, *A*, when gearing is used the engine and intermediate gearing are both apt to be wrecked by a sudden stoppage in the water-end. For this reason a belt instead of gearing is often used, its disadvantage being the larger amount of space necessary, 10 to 20 feet at least being required by the belting alone. "A higher piston-speed can be had with a crank and fly-wheel pump than if the pump were direct-acting, for the reason that in the latter type the termination of each stroke is defined and secured by steam acting as a cushion to counteract the force of the moving parts of the water. In large steam-pumps 100 feet per minute may be considered as the limit to safe piston-speed. With pumping-engines having cranks, connecting-rods with fly-wheels to terminate and define the stroke of the piston, any piston-speed possible to the pumps can be secured with safety. The power stored in the moving mass of the fly-wheel at the termination of the stroke is carried to the beginning of the next stroke without any loss but that due to the friction of the moving parts and the resistance of the air to the motion of the fly-wheel. Then the practically uniform speed of the rim of the fly-wheel secures the desired motion for the piston through the connecting-rod and crank of the pump by gradually retarding the motion until the point of rest is reached, and accelerating it after the piston has passed that point." (H. P. M. Birkenbine.) The definite length and termination of stroke are obtained by the power-pump also.

If we consider the two cylinders and pistons in *B*, Fig. 48, it is evident that the total pressure on the steam-piston is practically the same as that upon the water-piston. Hence the pressure per square inch on each is inversely as the areas of the pistons. Thus, if the diameter of the water-piston be 15 inches and that of the steam-piston be 12, and the pressure per square inch in the water end be 100 lbs., that upon the steam end must be $\frac{\overline{15}^2}{12^2} \times 100 = 156 +$ lbs. per square inch, and somewhat more than this steam-pressure must be maintained in the boilers. When the piston has reached the end of the stroke the steam-pressure must still be 1.56 times the water-pressure. There is still, however, energy remaining in the steam which could be utilized in expansion under less pressure. To effect this, the exhaust-steam (that escaping from the cylinder at the end of the stroke) is conducted to another cylinder whose piston has three to five times the area of the first and on which consequently the pressure is but one third to one fifth of 156 lbs. per square inch, and the steam can here expand until exerting only this pressure, when it is

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allowed to escape. In some cases still a third cylinder larger than the second is used to utilize further expansion. Such an engine is called a compound if there be two cylinders, a triple-expansion if there be three. The first cylinder is called the high-pressure, the last the low-pressure; if there be three, the middle is called the intermediate cylinder (see Fig. 50, page 343). The exhaust-steam from the low-pressure cylinder may be condensed by spraying cold water through it, or by passing it over pipes kept cool by water flowing through them, and a vacuum is thus produced in the low-pressure cylinder which adds still more energy. Compounding an engine adds about 20% to the energy derived from a given amount of steam; and condensing, about 20% more; and the third condensing-cylinder still further increases the efficiency of the engine in its use of steam. A high-duty engine—that is, one of high efficiency—must, to be so considered at the present time, be compound or triple-expansion, condensing, and the pump-end duplex or triplex; in addition to which a fly-wheel or an equivalent "high-duty attachment" is generally required. The latter is a method of storing energy at one part of the stroke and using it at another by the compression of water in a strong cylinder instead of by a fly-wheel.

A small engine is usually constructed with horizontal cylinders, since it can thus be made more solid at less expense. But if the pistons and rods are very large and heavy the friction upon their lower side causes unequal wear, and the rods may sag; larger foundations are required; and for other structural reasons it is preferable to place the cylinders vertical, the steam-cylinders being placed above the water-cylinders. These are called vertical, as distinguished from horizontal, engines. An additional advantage in their use occurs when the pump is placed beneath the ground-surface, since the steam-cylinders are then nearer the surface and the boilers, and the pump-pit may be made comparatively small.

So intimately are the pump and its motor generally connected that the whole is called a pumping-engine, is generally furnished by the same builder, and its efficiency as a whole is that ordinarily required and determined.

A great majority of the pumping-plants at present in service use steam as a motive power, and these will be first considered.

ART. 83. DUTY OF PUMPING-ENGINES.

In a steam-engine the applied energy is the amount of heat contained in the steam which reaches the engine. The efficiency of an engine is stated as its duty, which equals the amount of work done in pumping divided by $\frac{1}{1000000}$ of the heat-units furnished, or $\frac{1}{1000}$ of the pounds of dry steam, whichever is specified. It is assumed that one pound of steam furnishes 1000 heat-units, but this amount varies, and the most accurate and reliable method is the use of heat-units, the "British thermal unit" being that generally used. The duty is described as foot-pounds of work per 1000 lbs. of moist steam, per 1000 lbs. of dry steam, or per 1,000,000 B.T.U. (A British thermal unit is the amount of heat necessary to raise one pound of water at its maximum density through one degree Fahr.)

That these methods do not all give the same result is seen by the following table giving the result of a test made of a 16-million-gallon pumping-engine designed by E. D. Leavitt and built by the I. P. Morris Co.

In this case one pound of dry steam furnished about 1075 heat-units. (If water is carried over into the steam-pipes by the steam from the boiler, or is formed in them by condensation, this furnishes no energy and should be deducted from the weight of the steam. This reduced weight is referred to in the following table under the head of "dry steam.")

DUTY OF PUMPING-PLANT.

Day.	Ft.-lbs. per 100 lbs. Dry Pittsburg Coal.	Per 100 lbs. Dry Pocahontas Coal.	Per 100 lbs. Pittsburg Combustible.	Per 100 lbs. Pocahontas Combustible.
First	117,192,000	....	120,504,000	....
Second...............	130,170,000	....	133,895,000	....
Third................	129,854,000	....	134,464,000	....
Fourth	....	132,208,000	....	138,516,000
Fifth................	....	140,747,000	....	147,015,000
Sixth	....	144,676,000	....	152,370,000

DUTY OF PUMPING-ENGINE.

Day.	Per 1,000,000 Heat-units.	Per 1000 lbs. Moist Steam.	Per 1000 lbs. Dry Steam.
First	138,008,000	148,285,000	149,104,000
Second	137,525,000	147,892,000	148,710,000
Third....................	137,948,000	148,195,000	149,014,000
Fourth	137,387,000	147,667,000	148,483,000
Fifth....................	138,274,000	148,538,000	149,360,000
Sixth	136,260,000	146,459,000	147,269,000

The total number of B.T.U. in steam is represented by the equation

$$H = 1092 + 0.3(T - 32^\circ) = 1146 + 0.3(T - 212^\circ),$$

in which T is the temperature of the steam in degrees Fahr. The available energy is that due to $H - h$, when h is the number of B.T.U. in the steam which leaves the cylinder. The latent heat of evaporation of steam = 966 B.T.U. when under one atmosphere of pressure, or when the pressure is 14.7 lbs. per square inch. If the pressure is greater the evaporating-point is higher, as is shown in Table No. 67, the first and second columns.

Table No. 67.

FACTORS OF EVAPORATION.

Pressure per Sq. In.	Boiling Point ("at")	Initial Temperature of Feed-water ("from")										
		32°	50°	68°	86°	104°	122°	140°	158°	176°	194°	212°
14.7	212°	1.19	1.17	1.15	1.13	1.11	1.10	1.08	1.06	1.04	1.02	1.00
20.8	230	1.20	1.18	1.16	1.14	1.12	1.10	1.08	1.06	1.04	1.02	1.01
28.83	248	1.20	1.18	1.16	1.14	1.13	1.11	1.09	1.07	1.05	1.03	1.01
39.25	266	1.21	1.19	1.17	1.15	1.13	1.11	1.09	1.07	1.06	1.04	1.02
52.52	284	1.21	1.20	1.18	1.16	1.14	1.12	1.10	1.08	1.06	1.04	1.02
69.21	302	1.22	1.20	1.18	1.16	1.14	1.12	1.11	1.09	1.07	1.05	1.03
89.86	320	1.22	1.21	1.19	1.17	1.15	1.13	1.11	1.09	1.07	1.05	1.03
115.1	338	1.23	1.21	1.19	1.17	1.15	1.14	1.12	1.10	1.08	1.06	1.04
145.8	356	1.23	1.22	1.20	1.18	1.16	1.14	1.12	1.10	1.08	1.06	1.04
182.4	374	1.24	1.22	1.20	1.18	1.17	1.15	1.13	1.11	1.09	1.07	1.05
225.9	392	1.24	1.23	1.21	1.19	1.17	1.15	1.13	1.11	1.09	1.07	1.06
276.9	410	1.25	1.23	1.22	1.20	1.18	1.16	1.14	1.12	1.10	1.08	1.06
336.3	428	1.25	1.24	1.22	1.20	1.18	1.16	1.14	1.12	1.11	1.09	1.07

The higher the evaporating-point the greater the amount of latent heat in the steam; it being 4% greater when the pressure is 115 lbs., for instance, than when it is 14.7 lbs., as is shown by the above table. It takes a certain amount of heat, or number of heat-units, to raise the temperature of water each degree. Hence the number of pounds of water which can be evaporated by a given amount of heat depends upon the original temperature and that of the steam, which latter varies with the boiler-pressure. In order to compare different tests, the actual duty performed by coal in a given test is reduced to that it would have performed had the water been at a temperature of 212° Fahr. when reaching the boiler. If the temperature of the water be lower than 212°, or the pressure more than 14.7, more heat-units will be required. As an illustration of the use of Table No. 67: if one pound of coal evaporate 8 lbs. of water whose original temperature was 50°, the steam-pressure being 182.4 lbs. per square inch, the work done was equivalent to evaporating 1.22 times 8 lbs., or 9.76 lbs., from and at 212°.

The efficiency of the boiler which generates the steam is of nearly as great importance as that of the pumping-engine; and a high efficiency of the entire plant—boiler, steam-pipes, engine, and pump—acting together is the end sought. In a majority of cases this total efficiency is guaranteed by one firm, which furnishes the entire plant in running order; and this efficiency is that determined, although generally that of the separate portions is ascertained also. This total efficiency or duty is expressed as foot-pounds of work per 100 lbs. either of coal or of combustible matter in the coal, or per 1,000,000 heat-units in the coal. The efficiency of the boiler alone is expressed in the number of pounds of steam formed for each pound of coal burned, or for each pound of combustible matter in such coal. The former is found by weighing the steam and the coal used in producing it; the latter by deducting from the weight of coal that of the ashes remaining. In the test tabulated on page 332 the ashes on the first day formed

$$\frac{120,504,000 - 117,192,000}{120,504,000} \text{ or } 2.7 + \text{ per cent of the Pitts-}$$

burg coal, and on the fourth day 4.7 + per cent of the Pocahontas coal. One pound of Pittsburg coal was found to supply about 9250 heat-units and to evaporate 8.60 lbs. of water; while one pound of Pocahontas coal supplied 10,294 heat-units and evaporated 9.57 lbs. of water. Different coals may vary 30% in their "calorific effect," or available heat-units. Eureka Clearfield bituminous coal has been found to contain 15,174 B.T.U. by calorimeter tests.

It is desirable to supply water to the boiler as warm as may be without increased expense, and this is effected by a feed-water heater or economizer, in which a part of the heat escaping from the fire into the flue and otherwise wasted is utilized to heat the water which is being supplied to the boiler. A portion of the heat in the exhaust-steam is used for the same purpose, also. To prevent radiation from and condensation in the steam-cylinders of high-duty pumps, steam is passed continually around them in "jackets," and from these it passes either directly to the boiler, or to a "hot well" from which it is pumped into the boiler by the feed-water pump. Other minor appliances are used for decreasing loss of heat by radiation from boiler, steam-pipes, pumps, etc., or for utilizing such heat.

To understand and study the energy furnished, utilized, and lost in a steam-plant, it should be borne in mind that steam is but an agent for utilizing heat, and that all the heat furnished by the fire can and should be accounted for in a test. The following details of the test above referred to are here given, accompanied by a graphic representation of the same facts, prepared from the data by Capt. H. R. Sankey and presented by him in a report to the Institution of Civil Engineers. It is thought that a careful study of these, and particularly of the diagram, will be of great value in getting clearly in mind the losses in, and efficiency of, the various parts of a steam pumping-plant.

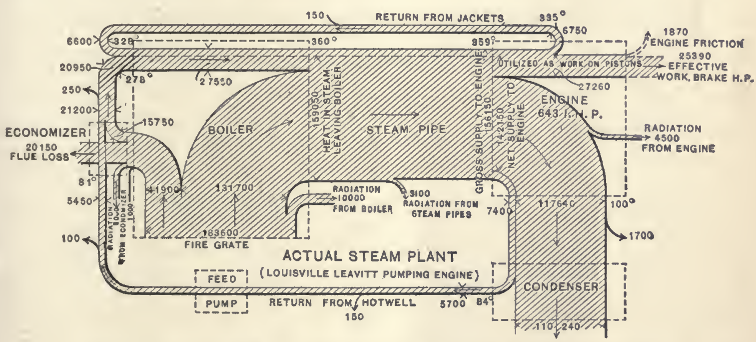

FIG. 49.—LOSSES OF HEAT IN A STEAM PUMPING-PLANT.

"Starting at the fire-grate, it is shown that 183,600 B.T.U. are produced per minute by the combustion of the coal, and that 131,700 of these go direct into the water of the boiler, 10,000 are lost by boiler-radiation and leakage, and the remainder, viz., 41,900, pass away with the flue-gases. On the way to the economizer 1000 B.T.U. are lost by radiation, but in the economizer itself 15,750 B.T.U. are diverted into the feed-water, 5000 B.T.U. are dissipated by radiation, and finally 20,150 B.T.U. pass out of the economizer and into the chimney, and are lost to the steam-plant. The heat entering the economizer with the feed-water is 5450 B.T.U., which is added to the 15,750 B.T.U. diverted from the flue-gases, thus giving a flow of 21,200 B.T.U. in the feed out of the economizer. Radiation, however, reduces this flow to 20,950 B.T.U. per minute at the entry to the boiler, where a further addition is made of 6600 B.T.U. returned by the jacket-water.

"The steam produced by the boiler is thus seen to derive its heat from three streams, as shown in the diagram; the steam finally leaves the boiler with 159,250 B.T.U. per minute. Before this heat gets to the engine, however, 3100 B.T.U. are lost by radiation and leakage from the steam-pipes, so that the flow of heat is reduced to 156,150 B.T.U. per minute, which is the gross supply of heat to the engine; the net supply is less, because there are certain returns of heat to the boiler to be deducted. In the first place credit has to be given to the engine for the heat which could be imparted by means of the exhaust-steam to the feed-water, inasmuch as the exhaust is theoretically, and very nearly practically, capable of raising the temperature of the feed to the exhaust temperature. On this basis 7400 B.T.U. should be credited to the engine. Although the actual return to the boiler, or rather to the economizer, is only 5450 B.T.U." (H. R. Sankey.)

Of the 6750 "return from jackets" the 150 lost by radiation should be charged against the engine, since the jackets are supplied to correct a fault in the engine, viz., cylinder-condensation. This leaves 142,150 B.T.U. as the net supply to the engine. Of this 117,640 departs in the exhaust without yielding any of its energy, and the remainder, or 27,260 B.T.U., goes into work in the engine. Of this, 1870 is consumed in the internal friction, leaving the actual work done in pumping due to 25,390 B.T.U. The thermal efficiency of any part of the plant can now be readily determined, and is the ratio of the number of B.T.U. applied to those delivered as heat or work. Thus that of the engine is $\frac{27,260}{142,150} = 0.19+$; or, taking the actual work done, $\frac{25,390}{142,150} = .18-$, the per cent of applied energy utilized. The thermal efficiency of the boiler, economizer, and grate is $\frac{159,250}{183,600 + 5450 + 6600} = .81\frac{1}{2}$; and of the entire plant $\frac{25,390}{183,600} = .14-$. The mechanical efficiency of the pump is $\frac{25,390}{27,260}$, or $.93+$.

The following are some of the data obtained in the test of this engine:

STEAM USED BY ENGINE AND FEED-PUMP; ENTIRE TEST OF 144 HRS. 10 MIN.

(1) Weighed feed-water	968,128 lbs.
(2) Feed-pump steam, condensed	23,390 "
(3) Total water pumped into boilers	991,518 "
(4) Total water returned to boilers from jackets and reheaters	189,795 "
(5) Sum of (3) and (4)	1,181,313 "
(6) Total steam used by calorimeter	727 "
(7) Total water drained from separator	23,428 "
(8) Total moist steam used by engine and feed-pump	1,157,158 "
(9) Percentage of moisture in steam after leaving separator	0.55%
(10) Total dry steam used in engine and feed-pump (= 99.45% of (8))	1,150,792 lbs.

STEAM USED BY ENGINE.

(11) Total moist steam used by engine only	1,133,768 lbs.
(12) Total dry steam used by engine only	1,127,533 lbs.

(13) Total moist steam passing through cylinders...	943,973 lbs.
(14) Total moist steam passing through jackets and reheaters..............................	189,795 lbs.
(15) Percentage of moist steam used by jackets and reheaters..............................	16.74%
(16) Moist steam used per hour per I.H.P. (indicated horse-power) *......................	12.223 lbs.
(17) Dry steam used per hour per I.H.P.*..........	12.156 lbs.
(18) Dry steam passing through cylinders per hour per I.H.P..............................	10.120 lbs.
(19) Dry steam used per hour per pump H.P.*.......	13.050 lbs.

B.T.U. SUPPLIED BY BOILERS.

(20) Heat of vaporization, steam 154.6 lbs., absolute †	859.4 B.T.U.
(21) Heat of liquid, steam 154.6 lbs.†...............	332.5 B.T.U.
(22) Heat of liquid feed, 143.30....................	111.5 B.T.U.
(23) Per pound of moist steam supplied by boilers $(859.4 \times .9945 + 332.5 - 111.5 =)$............	1,075.7 B.T.U.
(24) Total supplied by boilers, ((5)) $1,181,313 \times 1075.7 =$	1,270,738,600 B.T.U.
(25) Total supplied by boilers per minute...........	146,903 B.T.U.

B.T.U. USED BY THE ENGINE.

(26) Per pound of moist steam used by the cylinders	1134.5 B.T.U.
(27) " " " " " " in jackets and reheaters..................................	880.8 B.T.U.
(28) Used by engine during trial....................	1,238,108,959 B.T.U.
(29) " " " per minute....................	143,134 B.T.U.
(30) " " " " " per I.H.P..........	222.46 B.T.U.
(31) Average mean effective pressure in high-pressure cylinder..............................	43.53 lbs.
(32) Average mean effective pressure in low-pressure cylinder..............................	14.155 lbs.
(33) H.P. developed in H.P. cylinder...............	279.00
(34) H.P. " " L.P. cylinder...............	364.40
(35) H.P. lost in friction..........................	44.30
(36) Efficiency of mechanism........................	93.12 per cent.

FUEL.

	Coal.	
	Pittsburg.	Pocahontas.
(37) Moist coal consumed, lbs......................	67,917	63,591
(38) Wood consumed at 50 per cent weight, lbs......	772	25
(39) Moisture in coal, per cent.....................	0.7	2.6
(40) Dry coal consumed with wood equivalent, lbs.	67,995	61,692

* I. H. P., the energy utilized as work on the pistons, represented in the diagram by 27,260 B.T.U. Pump H.P., the energy delivered by the pump in work, represented as 25,390 B.T.U. per minute.

† Note that the sum of (20) and (21), or total heat units, gives the same result as the formula for H; the temperature of the steam being 365°.

(41) Total ash, dry, lbs	2,025	2,849
(42) Total combustible, lbs	65,970	58.843
(43) Calorific value of one pound of coal by analysis B.T.U	13,226	14,924
(44) Water actually evaporated per pound of dry coal, lbs	8.60	9.57
(45) Equivalent water evaporated per pound of dry coal, from and at 212°, lbs	9.63	10.70
(46) Equivalent total heat derived from a pound of dry coal, B.T.U	9,250	10,294
(47) Water actually evaporated per pound of combustible, lbs	8.86	10.03
(48) Equivalent water evaporated per pound combustible, from and at 212°, lbs	9.92	11.22
(49) Efficiency of boilers, per cent $\left(=\frac{(46)}{(43)}\right)$	70.0	69.0
(50) Dry coal burned per sq. ft. of grate per hour, lbs.	12.70	11.50
(51) Coal used per I.H.P. per hour, lbs	1.47	1.33
(52) Coal used per pump H. P. per hour, lbs	1.58	1.43

This test, conducted by F. W. Dean and Dexter Brackett in April 1894, was continuous for 144 hours and 10 minutes. The engine was regarded by Mr. Dean as "the most economical compound engine that has ever been tested." Its duty of 137,565,000 ft.-lbs. per 1,000,000 B.T.U. has been exceeded by triple-expansion engines, however; and Geo. H. Barrus considers 155 or even 160 million foot-pounds as possible of attainment. The table on next page was prepared by him, and brings records of high-duty engines up to December 1898.

From the definition of duty it is evident that theoretically an engine having a duty of 100,000,000 will consume but half as much coal as one having a duty of 50,000,000. And this will be practically the case if there be in charge experienced engineers and firemen who can and will constantly keep both boiler and engine working at their best. If such men are not placed in charge it is doubtful whether a high-duty pump should be used, as it is more liable than a low-duty to be ruined by careless treatment, and under such treatment

TABLE NO. 68.—DATA OF HIGH-DUTY PUMPING-ENGINES. (G. H. BARRUS.)

1. Name of designer or builder	E. P. Allis & Co.	E. D. Leavitt, Jr.	E. P. Allis & Co.	Lake Erie Eng. Wks.	Snow Steam-pump Works
2. Locality	Milwaukee, Wis.	Chestnut Hill, Mass.	Detroit, Mich.	Buffalo, N. Y.	Indianapolis, Ind.
3. Type	Trip. Expan.	Trip. Expan.	Trip. Expan.	Trip. Expan.	Trip. Expan.
4. Extent of jacketing	Barrels and receivers	Barrels, heads, and receivers	Barrels and receivers	Barrels and receivers	Barrels, heads, and receivers
5. Name of expert conducting test	Prof. R.C.Carpenter	Prof. E. F. Miller	Geo. H. Barrus	Geo H. Barrus and Newcomb Carlton	Prof. W. F. M. Goss
6. Capacity—million gallons in 24 hours	18	20	24	30	20
7. Size of steam-cylinders, inches	28, 48 74 × 60	13.7, 24.37, 39 × 72	28, 48, 74 × 60	37, 63, 94 × 60	29, 52, 80 × 60
8. Size of water-plungers, inches	32 × 60	Dbl. act'g 17.5 × 48	36 × 60	42 × 60	33 × 60
9. Total head, pounds	70.4	59.4	53.4	86.1	88.7
10. Piston-speed, feet per minute	203.1	607	209.9	207.7	214.6
11. Ratio of vol. of L.P. cylinder to vol. of H.P. cylinder	7.1	8.3	7.1	6.5	7.7
12. Pressure near throttle (above atmosphere), lbs	121.4	175.7	125.2	167.1	155.6
13. Cut-off pressure (above atmosphere), lbs	118.6	151.5	119.4	152.2	153
14. Release pressure, L.P. cylinder (above zero), lbs	5.3	6.9	5.8	7.4	6.4
15. Back " " " " " "	1.6	1.5	2.8	2.2	2.5
16. Cut-off, H.P. cylinder	.337	.384	.338	.323	.315
17. Clearance, H.P. cylinder	.014	.015	.014	.014	.018
18. Ratio of expansion	20.4	21	20.3	19.6	23.8
19. Ratio of expansion referred to pressure near throttle	20.8	23.9	21.2	21.3	24.3
20. $\frac{\text{Absolute pressure near throttle}}{\text{Ratio of expansion}}$, lbs	6.6	9.1	6.9	9.3	6.8
21. Indicated horse-power (I.H.P.)	573.9	575.7	573.7	1185.5	775.5
22. Friction, per cent	9.2	10.5	10.2	5.1	4.6
23. Dry steam per I.H.P. per hour, including jacket and reheater steam, pounds	11.68	11.22	12.52	12.39	11.26
24. Per cent of steam condensed in jackets and reheaters	9.2	17.1	12.7	13.7	10.5
25. Dry steam per I.H.P. per hour, exclusive of steam used in jackets and reheaters, lbs	10.61	9.3	10.93	10.7	10.08
26. Steam accounted for by indicator, H.P. cylin. cut-off, lbs.	9.05	8.5	9.5	9.1	8.7
27. Steam accounted for by indicator. L P. cylin. cut-off, lbs.	8.7	9.6	9.5	9.7	8.7
28. Steam accounted for by indicator, L.P. cylin. release, lbs.	9.04	9.1	9.9	9.7	9.8
29. Cylinder condensation and leakage, including jacket and reheater condensation, at cut-off, H.P. cylinder	.225	.242	.241	.266	.227
30. Mean effective pressure referred to L.P. cylinder, lbs.	21.77	26.36	21.03	27.19	23.65
31. Theoretical mean effective pres. referred to pres. near throttle-valve and ratio of expansion at same point.	26.42	33.23	26.76	34.63	29.66
32. Line 30 ÷ line 31, or "diagram factor"	.824	.794	.792	.786	.797
33. Duty based on 1,000,000 heat-units, expressed in million foot-pounds	137	141.9	129.7	135.4	150.1
34. Duty based on 1000 lbs. of dry steam, expressed in million foot-pounds	154	154.9	142.4	152	167.8

may not realize more than 50 to 75 per cent of its full efficiency. In any particular case the interest on the cost and the depreciation of a low-, medium-, and high-duty pump, with the cost of coal, engineer's and fireman's salaries, and other expenses of each should be compared with those of the other, and that which shows the least annual cost will be the most economical. With pumps of a capacity less than 300,000 to 500,000 gals. per day, single-expansion, or simple, engines alone are ordinarily made, and the duty is seldom greater than 38,000 or 40,000. Above this size compound duplex pumps are generally used; and horizontal triple-expansion pumps (see Fig. 50, page 343) are made of a capacity of 750,000 to 5,000,000 gals. daily. Vertical engines are made of all sizes, but horizontal are now seldom made larger than 5,000,000 gals. A compound condensing direct-acting engine can be obtained with a duty of 50 to 125 million foot-pounds, depending upon its size; and 75 to 160 million is obtained with triple-expansion engines of 1 to 30 million gallons capacity.

The duty possible of attainment depends in some degree upon the pressure of steam in the boiler. Charles A. Hague states that the different steam-pressures will cause the following differences of duty in the same engine.

Steam-pressure.	Duty per 1000 lbs. of Steam.	
	Compound Engine.	Triple-expansion Engine.
75	104,000,000	....
80	106,000,000	....
90	108,000,000	....
100	110,000,000	125,000,000
110	112,000,000	126,000,000
120	114,000,000	128,000,000
129	....	130,000,000
141	....	134,000,000

The duty of an engine is also reduced by reducing its work. It is hence inexpedient to call for an engine of much greater capacity than that which is actually required. "A properly designed pumping-engine will work with water-pressures from 10 to 25 per cent higher than the nominal pressure, without appreciably falling off in economy; but if the water-pressures are from 10 to 25 per cent less than the engine was designed for, the falling off in economy becomes very marked." (I. H. Reynolds.) It is hence desirable that the specified duty be required at ordinary working pressure and speed, but that the pump be guaranteed to safely work under maximum conditions.

Engines under working conditions seldom attain the duty found by the test, as might be expected. But they can be kept up to 90% of this when in good condition, by careful, intelligent management. Boilers of the Brooklyn Water-works evaporated 10.94 lbs. of water, at and from 212°, per pound of Eureka coal, in 1895. The pumping-plant at Newton, Mass., worked with an average annual efficiency of 110,000,000 ft.-lbs. per 100 lbs. of coal. On the other hand, the effect of wear and necessity of renewing pumping-plants which have been outgrown is shown by that at Taunton, Mass., which attained an average duty of 53,406,265 ft.-lbs. in 1893, 37,837,519 in 1895, and but 34,096,561 in 1896.

ART. 84. ARRANGEMENT OF PUMPING-ENGINES.

Horizontal compound engines are made with the low-pressure cylinder either between the high-pressure and the water end, or in the reverse order. In the former case the smaller high-pressure cylinder is generally made to overhang the base, and thus reduce the length of foundation necessary. In fact, the reduction of cost made possible by this is about the only advantage of this arrangement; and

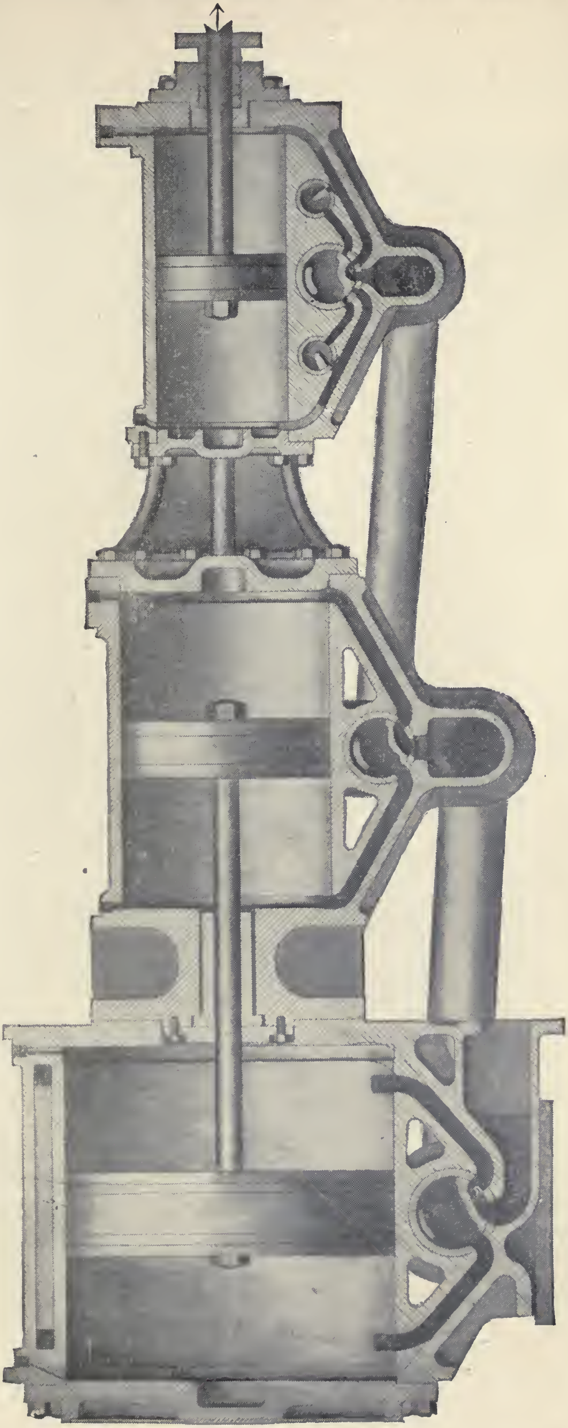

FIG. 50.--TRIPLE-EXPANSION HORIZONTAL ENGINE.
(Worthington Type, steam-end only.)

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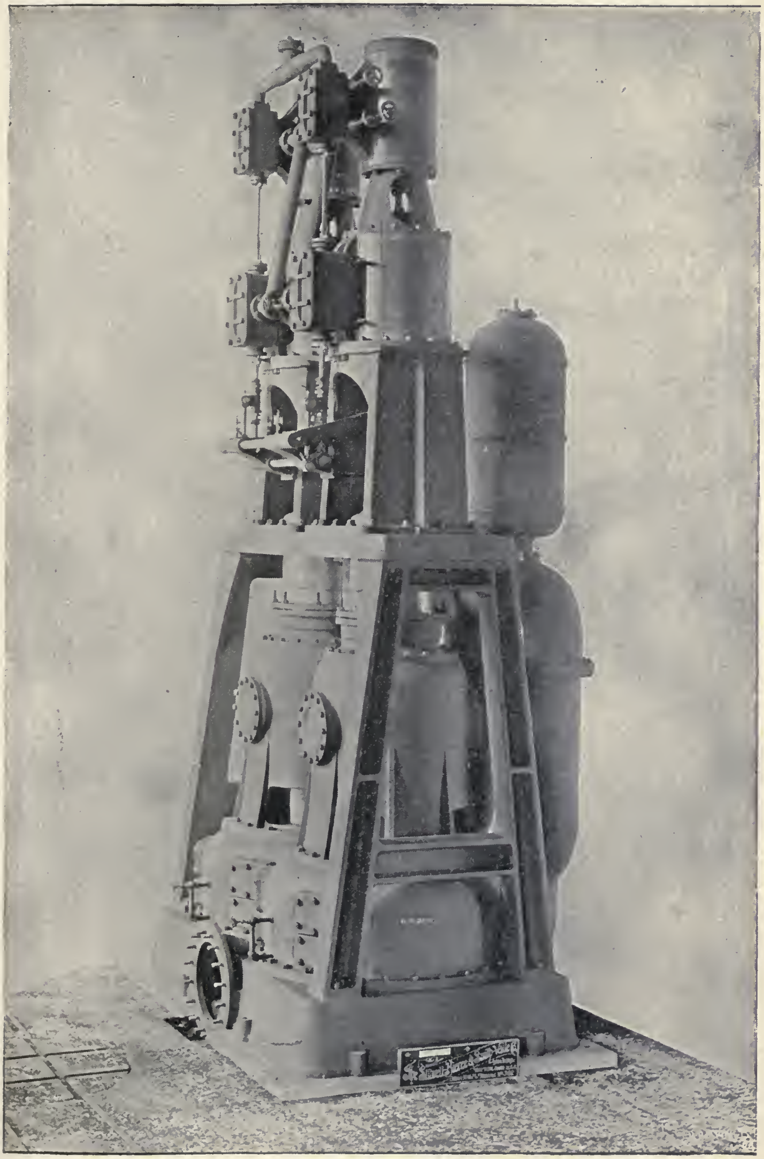

FIG. 51.—VERTICAL COMPOUND PUMPING-ENGINE.
(Differential Plunger Type, Smith-Vaile.)

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it has the disadvantage of causing the L.-P. cylinder to be less accessible for adjustment and repairs, and makes a less rigid construction. It is not often employed for engines of more than 750,000 or 1,000,000 gals. capacity, and is not recommended for any. In vertical engines, either compound or triple-expansion, the steam-cylinders may be tandem (one above the other, see Fig. 51), but are more often placed side by side; the latter arrangement not only being more rigid, but permitting two or three cranks or direct-acting pump-cylinders to be used and thus securing a more continuous flow.

The suction-lift of a pump may be made 25 or 26 feet, but it is better to make it not more than 15 feet if possible. In the case of pumping from rivers subject to great variations in height, the pump must be raised and lowered to suit the stages of the river; or, better, it should be placed in a water-tight pit which extends above the highest water. This construction is necessary on the Ohio and other rivers of our "central basin." In the new Cincinnati Water-works now under construction the pump-pit is 95 feet deep and 98 feet in diameter, in which four 30,000,000-gallon pumps are to be placed (see Fig. 52).

Pumps in such a pit may be vertical, direct-acting, or crank-and-fly-wheel; or they may be power-pumps, the engine being upon the surface and thus more accessible. An excellent illustration of the latter arrangement is afforded by the Rockford, Ill., plant, recently completed. In this the pump is placed 79 feet below the engine, to render available a large flow of ground-water. The pumps are centrifugal, 3.5 feet diameter, designed for 300 to 350 revolutions per minute and one million gallons per day each, with a suction-lift of 26 feet and discharge head of 60 feet. They are driven by rope transmission from vertical compound condensing-engines of a mechanical efficiency of about 93%; the plant showing a duty of 58,000,000 per 1000 lbs. of dry steam.

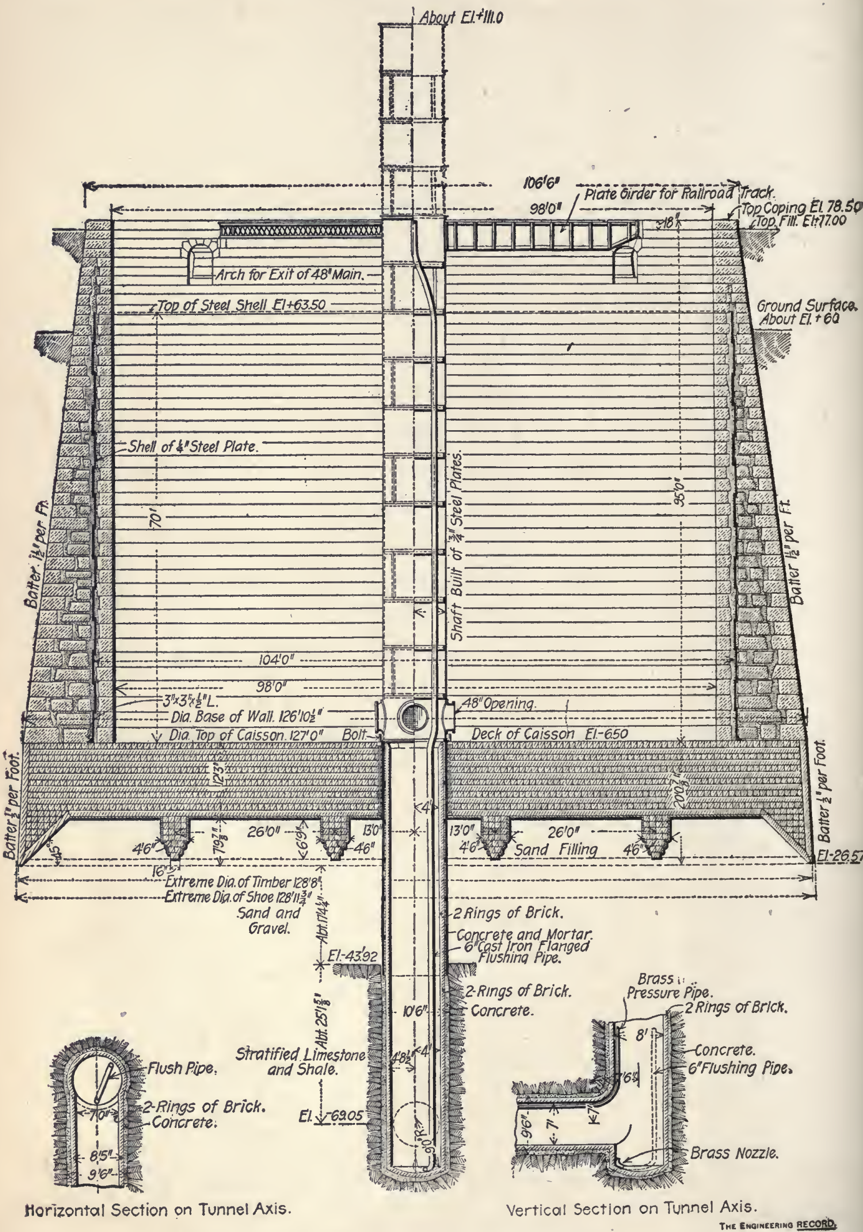


FIG. 52.—PUMP-PIT, CINCINNATI WATER-WORKS.

When water is to be taken from a deep driven well the entire pump must be of such size as can be lowered into the tube, and those at present in use are driven by a long pump-

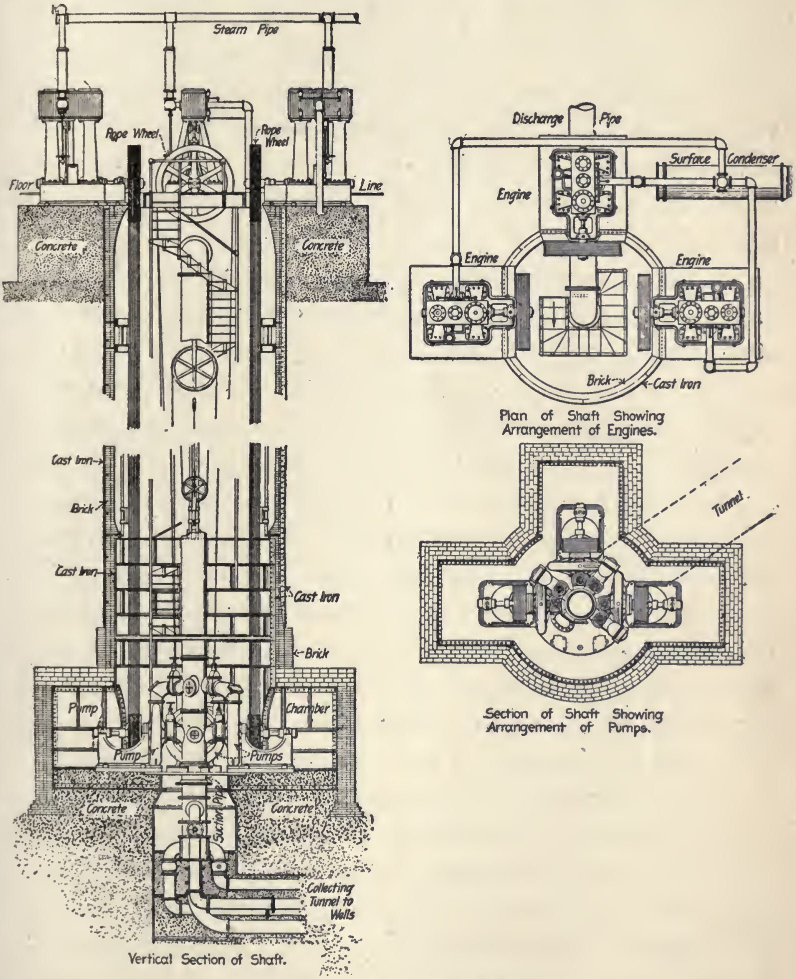


FIG. 53.—CENTRIFUGAL-PUMP PLANT, ROCKFORD, ILL.

rod extending to the surface. In general the pump is single-acting, its cylinder made of brass tubing, the principle being that shown in Fig. 45, page 323. The water-cylinder is suspended from a drop-pipe of slightly larger diameter, or is locked into the well-casing, which acts as a discharge-pipe. The steam-cylinder is placed above the well-opening, and the engine is direct-acting (see Plate XVII, page 397). The strain on the pump-rod should always be tensile, owing to its great length; hence, to render this pump continuous in flow two water-cylinders are furnished with independent piston-rods. The great weight of the rod and column of water above the cylinder in a deep well renders great velocity of motion impossible.

If the pumping-engine is direct-acting, it is evident that any repairs to or inspection of the pump are possible only after removing the engine and drawing the pump-rod and cylinder out of the well. To facilitate this operation a working-head is often employed; the greater advantage in its use, however, being the higher efficiency attainable. The working-head is a contrivance for utilizing the power of an ordinary slide-valve engine, converting rotary into reciprocating motion, and may be driven by belt or gearing. The working-head is placed over the well and drives the pump-rod; the engine is placed at a greater or less distance away from the well. A modification of this gives a more nearly continuous flow to either single- or double-cylinder pumps by making the down-stroke much more rapid than the up.

A pump has recently been introduced which utilizes the principle of the screw to obtain continuous flow. It consists of a central torsion-shaft, on which a pair of "runners" is placed every 3 to 5 feet. The runners are shaped similar to the propellers of a vessel, and fit closely to the sides of the well-casing. A lift of over 200 feet has been made with such a pump. Its efficiency is not known.

Every pumping-engine should be protected by a building from rust-producing moisture, from dust and dirt, and from curious or malicious tampering with. To keep from it dust and grit from the coal-house and boiler-room these should be entirely separated from the pumping-engine, which should have a room entirely to itself. Near it should be placed gauges showing steam-pressure in the boilers, pressure in the discharge-pipe, and vacuum in the suction. Also a counter, registering the number of revolutions or strokes of the engine. The foundation of the pump should be perfectly rigid and unyielding, resting upon a firm soil, and substantial and strong to resist vibrations.

The pump-room should be light, dry, and easily ventilated; and the pump readily accessible in all its parts. The boiler-room should be close to the pump-room, that the length of steam-piping and consequent loss of heat may be as small as possible. The boilers should always be of such number and capacity that any one can be put out of service without rendering impossible the maximum work at any time desirable. They should be placed as near each other as possible to reduce loss of heat by radiation. The steam-piping should be so arranged that any boiler may be used for any engine. The coal-house should be as near the fire-doors as possible, to reduce the handling of coal. A good plan is to place a long coal-room just in front of the boilers, with only sufficient room between to permit of using the rake, flue-cleaners, and other boiler tools. The boiler-room should be well built-in to prevent unnecessary radiation of heat in winter. Boilers have been known to have their efficiency decreased 5% or more by cold weather. Facilities for delivering coal to the station should not be neglected. It should be near a railroad, canal, or navigable river if possible. If the coal come by rail it may be economical to furnish a siding into the coal-room or along its outer side; and if by water, to provide a dock and hoisting-derrick for filling the bins.

ART. 85. BOILER-PLANTS.

Boilers in common use in water-works plants may be classified as water-tube, and fire-tube or flue. In the former the water passes through a coil of tubes so arranged that a large number of coils are contained inside a single casing, the fire passing through the casing and around the tubes. In horizontal return-tubular (flue) boilers the water is contained in the casing, and the heated air passes under this and back through the tubes, which extend through the boiler from end to end and are not connected with each other. Flue boilers other than horizontal return-tubular are seldom used in water-works.

The capacity of a boiler is expressed in horse-powers. Theoretically this is the amount of work, in H.P., which an engine fed by this boiler could perform if its efficiency were 100%; but boilers are commercially designated by arbitrary standards of capacity. Fifteen square feet of heating-surface is considered equivalent to one H.P., and the grate-area is generally about $\frac{1}{40}$ the heating-surface. On page 353 are shown the average dimensions of ordinary return-tubular boilers. In the Louisville test above described the heating-surface was but 7 square feet per H.P. developed in the engine; but if only 70 million duty had been developed, the ratio would have been about 15 square feet.

In a flue-boiler the boiler-shell must be sufficiently strong to resist the maximum steam-pressure. In a water-tube boiler only the tubes and steam-drum must resist this pressure, the shell serving to confine the heat to them. For this reason, largely, water-tube boilers are frequently used for high steam-pressures. In the latter boiler the scale, being inside the tubes, is difficult to remove; while in the fire-tube boilers the removal of soot and other fine deposits from the flues is similarly difficult.

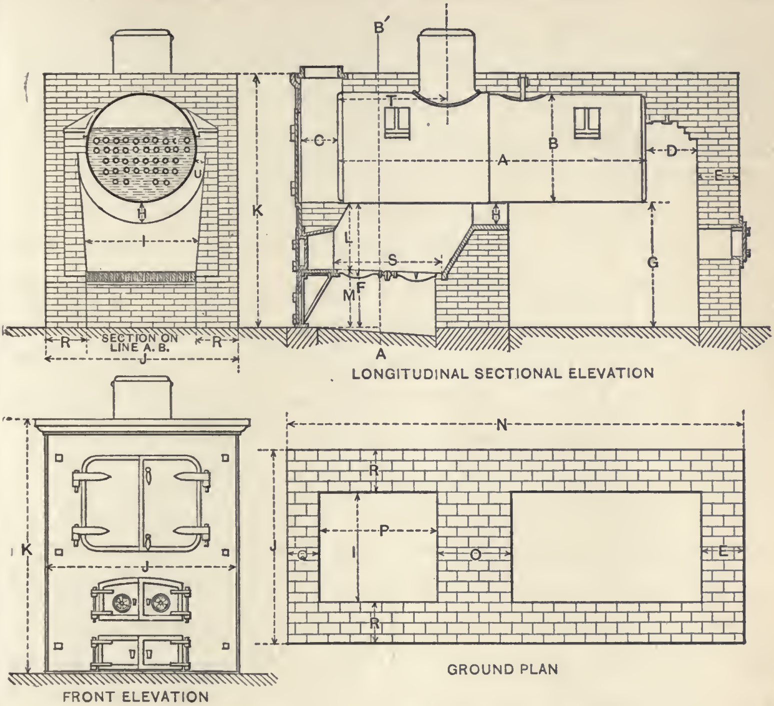


FIG. 54.—SETTING OF RETURN-TUBULAR BOILER.

MEASUREMENTS FOR SETTING TUBULAR BOILERS.

No.	Reference Letters on Diagrams.												
	A Feet.	*B* Ins.	*C* Ins.	*D* Ins.	*E* Ins.	*F* Ins.	*G* Ins.	*H* Ins.	*I* Ins.	*J* Ins.	*K* Ins.	*L* Ins.	*M* Ins.
1	7	30	12	20	16	45	44	7	30	62	83	26	19
2	8	36	12	20	16	48	47	8	36	68	92	26	22
3	10	42	14	20	16	48	47	8	42	74	98	27	21
4	12	44	14	24	16	48	46½	10	44	76	100	27	21
5	14	48	16	24	16	47	45½	10	48	88	103	26	21
6	12	60	18	24	20	50	48½	12	60	108	118	26	24
7	16	60	18	26	20	50	48	12	60	108	118	26	24
8	16	66	18	28	20	50	48	12	66	114	124	26	24
9	18	72	20	30	20	50	48½	12	72	120	130	26	24
10	18	84	20	30	20	50	48	12	84	132	142	26	24

No.	*N* Ft.	Ins	*O* Ins.	*P* Ins.	*Q* Ins.	*R* Ins.	*S* Ins.	*T* Ins.	*U* Ins.	Number of Fire-brick.	Number of Common Brick above Floor-level.
1	11	6	20	40	12	16	36	34	4	500	4.800
2	12	6	24	40	12	16	36	34	4	600	6,000
3	14	8	28	46	12	16	42	42	4	750	8,000
4	17	0	32	52	12	16	48	49	4	900	10.400
5	19	2	36	58	12	20	54	84	4	1000	13,800
6	17	10	32	50	16	24	48	49	4	1300	17,000
7	22	0	40	56	16	24	54	96	4	1400	21,000
8	22	2	40	56	16	24	54	96	4	1550	23,000
9	24	6	40	62	16	24	60	108	4	1800	26,500
10	24	6	40	62	16	24	60	108	4	2000	31,000

The matter of height and area of chimneys has not been reduced to an exact science, and in a large number of cases is fixed by guesswork. The empirical formula of Abendroth & Root, which is perhaps as satisfactory as any, is

$$c = \frac{201}{\sqrt{h} - 8},$$

in which c is the area of the chimney in square inches per square foot of grate-surface, and h is the height of the chimney above the grate. The height of chimney depends upon the draught desired. Anthracite screenings probably require the most draught, and wood the least. Probably no chimney should be less than 40 feet in height, and the higher it is the greater the draught which can be obtained; although this is dependent also upon the force and direction of wind, the outside temperature, and that of the air in the chimney. Chimneys have been built over 200 feet in height, but probably 75 to 125 are the ordinary limits of chimneys for water-works plants.

Art. 86. Other Motive Powers for Pumps.

Next to steam, hydraulic power has probably been used more than any other for driving pumps. The near future may, however, and probably will, see electricity used quite widely for small plants, and perhaps for large ones under some conditions. In some cases gas-, gasoline-, oil-, hot-air-, and wind-engines are used for small plants. (The use of oil or gas under steam-boilers is of course but a detail of a steam-plant.)

Hydraulic power is conveniently used where water is taken from a river whose dry-weather flow sufficiently exceeds the amount to be pumped and which can be dammed at this point. Philadelphia's Schuylkill supply was for years wholly, and is still partly, pumped in this way. The motive power is the ordinary water-motor, generally a high-grade turbine. The rotary motion of the turbine may be communicated directly to a rotary pump, or by crank-shaft to a reciprocating pump. The pump and turbine should generally be near together, and the head-race of one may serve to feed the other also. A turbine with an efficiency of .80 may readily be obtained, and, given a pumping-engine with a 94% efficiency upon the same shaft, the total efficiency will be about 75%. If gearing is employed between turbine and pump, the efficiency may be reduced to 60%. Assuming an efficiency of .70, the proportion of total flow which can be pumped is $\frac{.70H}{h + .70H}$, in which H is the available head in the river and h is the total head against which the pumps must work. This method of furnishing power for water-supplies is subject to intermission by droughts in a majority of cities where it is possible at all, and in such cases an auxiliary steam-plant or other ever-available source of power should be provided for occasional use.

It sometimes happens that different parts of a city lie at such greatly different elevations that to furnish water to all through the same pipe system would necessitate providing a very low pressure in the higher part or an excessive one in the lower part of the city; and to avoid this two distribution systems are furnished, with a reservoir for each, or a stand-pipe taking the place of the upper reservoir. The maintenance of separate pumping-mains would be expensive, as would that of a separate pumping-station at the low-service reservoir. This difficulty has been met in some plants by placing a hydraulic motor on the pumping-main to the low-service reservoir, which motor is run by the water on its way to this reservoir. At Burlington, Vt., a motor 5 to 10 feet below the water-surface in a low-service reservoir raises water 60 feet to a high-service reservoir. If the efficiency of this motor were .90, it could lift .15 to .075 of the water flowing through it.

Such a motor may be of the general form of a reciprocating steam-pump; or some form of ram may be employed. The older style of ram was capable of an efficiency of about 65 to 70 per cent under practical conditions. Recent improvements are said to bring this up to 82%. Electric motors are used in a number of small water-plants, the current being generally purchased from an electric-power company. The power is transmitted from the motor either by gearing on the motor-shaft or by belt; the latter being generally preferable. The efficiency of the combined plant—steam-engine, line, and dynamo—may easily be 60 to 65 per cent. If the engine and boiler have a duty of 100,000,000 foot-pounds—which, being of considerable power, it may easily have—the efficiency of the two plants combined would be 60 to 65 million foot-pounds; a greater efficiency than a small pumping-plant could expect to attain, but less than a large one should be designed for. Under these conditions a number of small pumps run from a central steam-plant, or one such pump from a general electric-power plant, would be an economical arrangement. In connection with this comparison it should be remembered that such a plant will require no more space than a steam pumping-engine alone, no boiler- or coal-room being required. A saving may thus be made in the building, and in superintendence also—the latter a considerable item in many cases. It may also be that a cheaper source of power—as a hydraulic plant—or a saving in the hauling of coal will render economical the separation of engine and pump by a considerable distance and electrical transmission of power between the two. One of the largest electric pumping-plants in the country is at San Antonio, Tex., where a 2-million-gallon pump is run by a 30-H.P. motor with electricity furnished by the city electric-light plant.

Gas-, gasoline-, and oil-engines have been used in a number of small plants, but are not economical for large ones. The economy over steam lies in the smaller first cost of engine and building, the small expense for running and maintenance, and the ability to run for a few hours a day only without waste of fuel. One oil-engine in a certain small plant is started by a plumber each morning, visited at noon, and stopped at night, the plumber receiving $350 per year for his services. At Midvale, N. J., a triplex pump lifting 65 feet is run by a 4-H.P. engine on 0.312 gals. of gasoline per H.P. per hour. At Greensburg, Ind., a triplex pump, lift 81.3 feet, is run by a 6-H.P. engine on 0.47 gals. of crude oil per H.P. hour. At Winchester, Mass., a triplex pump, 500,000 gals. capacity, 143 feet lift, is run by a 20-H.P. Hornsby-Akroyd oil-engine on 0.17 gals. of oil (150° test kerosene) per H.P. hour, or 11,516,518 foot-pounds of work per gallon of oil.

Freeman C. Coffin considers that the table on page 356 gives a fair comparison of the total annual costs of pumping with different types of engines; this estimate including supplies, repairs, attendance, interest on cost, and depreciation (at 4 and 3 per cent respectively), and fuel.

This cost will of course vary with the character of fuel and efficiency of engine of each kind, and this table can only be taken as an approximation, which may be useful in preliminary study of what is desirable for a given small plant.

A description of these engines and their advantages are thus briefly summed up by Mr. Coffin in the Journal of the N. E. W. W. Assn. for March 1899:

" Gas- and gasoline-engines are practically alike in construction and operation; the same engine can be used with either fuel, a few alterations being required in the arrangement when the fuel is changed. The oil-engine is similar in general appearance and operation, but the treatment of the fuel in the engine differs from that of gasoline.

TABLE GIVING COMPARATIVE ANNUAL COST OF PUMPING WITH DIFFERENT TYPES OF POWER.

(Journal N. E. W. W. Assn.)

Average Daily Pumping, in Gallons.	Oil-engine; Oil at 9 cts. per Gallon.	Gasoline-engine; Gasoline at 9 cts. per Gallon.	Gas-engine.		Steam-pump.		
			Gas at \$1 per 1000.	Gas at \$0.50 per 1000.	Coal at \$5 per Ton.	Coal at \$4 per Ton.	Coal at \$3 per Ton.
50,000	\$ 770	\$ 735	\$ 920	\$ 675	\$1230	\$1160	\$1090
100,000	1275	1200	1580	1035	1740	1600	1460
200,000	2200	2050	2815	1820	2525	2300	2075
300,000	3085	2875	4000	2510	3130	2850	2570
400,000	3920	3640	5140	3150	3700	3350	3000
500,000	4745	4400	6270	3780	4200	3790	3380

"In a gas-engine the gas is introduced into the cylinder and mixed with air which is drawn through a valve into the cylinder by the outward stroke of the piston, the mixture is compressed by the return stroke and fired by an electric spark or an ignition-tube.

"The cylinder is open on one end. The explosion of the air and gas behind the piston drives it forward and imparts the energy to the fly-wheel, which is very heavy. There is but one explosion in two complete revolutions, the return of the piston forcing the gases formed by the combustion out at the exhaust, the next forward stroke drawing in the air and admitting the gas, and the return stroke completing the cycle of work and compressing the charge for the next explosion. The gasoline-engine works in precisely the same way, except that the fuel is forced in by a pump worked by the revolution of the engine and is turned to gas within the cylinder or combustion-chamber before mixing with the air.

"The principal difference in the working of the oil-engine is that the fuel, not being as volatile as gasoline, is introduced in a fine spray to a vaporizer, where it is turned into vapor by the heated walls and then mixed with air. The vaporizer must be heated by a special lamp before starting the engine; this requires from seven to ten minutes, and therefore the oil-engine requires that much more time to start it than the gas- or gasoline-engine, which simply requires the supply-cock to be opened, a few turns to be given to the fly-wheel and it is off, if there is no difficulty with the battery which provides the spark for firing. This seems to be the weak point in the gas-engine; at least the only difficulty that I have seen in starting and running them seems to be connected in some way with the battery. Some of the oil-engines require no battery, the charge being ignited after the engine is started by the heat of the walls of the vaporizer in combination with the pressure produced by the return of the piston. . . . Perhaps the most important feature in the operation of internal-combustion engines is the attendance. Any one with ordinary intelligence and no training as an engineer can be taught in a short time to run one. In a well-designed plant, properly supplied with large oil-cups, the necessary attendance is limited to starting the engine, providing a sufficient supply of fuel in the tank and oil in the cups, and stopping it at the proper time. Starting under ordinary conditions requires from a minimum of one minute with gas- or gasoline-engines to a maximum of fifteen with oil-engines.

"The speed of these engines seems to be most perfectly regulated by the governor which controls the supply of fuel, a sudden variation in load making but slight change in speed. If a main should break in front of the pump the speed would hardly vary 10%. On the other hand, there is very little adjustment of speed possible. About 15% from the rated speed either way is as much variation as can be obtained. If a pumping-plant were required in which the running capacity could be reduced 50%, it would be necessary to use two small pumps, one each side of the engine, with friction-clutches, when either or both pumps could be run. As the engine so readily adapts itself to a change of load, a pump could be thrown on or off at any time. These engines run very economically with a small load."

For very small plants windmills are sometimes used, in connection with a tank for storage during calm weather. It is very desirable, however, to provide a gasoline- or similar engine to supplement the wind-engine. The whole may be combined in one structure, the windmill above the tank, the gasoline-engine housed in beneath it. Wind-engines may be obtained of 40 H.P. easily capable of pumping 500,000 gals. per day 100 feet high; but dependence probably should not be placed upon an average of more than 200,000 gals. per day from such an engine, owing to the uncertainty of the wind.

Hot-air-engines have been used for small domestic plants with some success, but not for village or city plants to the author's knowledge. The Diesel Motor, which uses petroleum, has just been introduced into this country from Germany and is said to give a thermal efficiency 50% greater than the most perfect steam-engine; but this has not yet been demonstrated by use.

A contrivance for raising water called the "air-lift," which is not strictly a pumping-machine, has been brought into practical use within the past five or ten years and gives excellent service under certain conditions. Its principle of action is the lessened specific gravity of water in which air is contained in considerable quantity, or through which it is rising in bubbles. The air-lift consists essentially of an air-pump, and an air-pipe leading from this to the bottom of a water-tube, into which tube air is discharged. The air, rising into and through the water in the tube, causes the surface of this to rise, since the atmospheric or other pressure at the base remains the same. By forcing in sufficient air the water and air combined rise to the top of the tube and overflow. This lift has shown itself particularly adapted to increasing the flow from deep wells, but is uneconomical for any kind of surface pumping. Different manufacturers vary the details somewhat,—some, for instance, placing the air-tube within and others without the water-tube (see Fig. 55).

Little accurate scientific knowledge has yet been obtained concerning the working of the air-lift, and it is probable that its efficiency will be increased within the next few years. Efficiencies of from 2 to 50 per cent have been obtained from air-compression and lift combined. Probably 25 to 28 feet is the maximum height to which the water can be raised with any success. It is generally considered necessary to immerse the water-tube one to one-and-a-half times as deep below the ground-water surface as is the desired lift above it. Some tests have seemed to indicate better results from such an arrangement as *b*, Fig. 55, than from *a*; the air-pipe in *b* being simply lowered into the well-casing, 12 to 18 inches of its lower end being perforated with a large number of holes. One of the chief advantages of the air-lift is the flow attainable by it from small wells, as compared with a deep-well pump. With a double-acting or continuous-flow deep-well pump probably the greatest flow obtainable is 120,000 gals. per day from a 6-inch and 350,000 from a 10-inch tube-well; while from a 10-inch well at Indianapolis an air-lift pump raised about 1,500,000 gals. per day to a height of 25.7 to 27.3 feet. If the tube had been 18 or 20 inches in

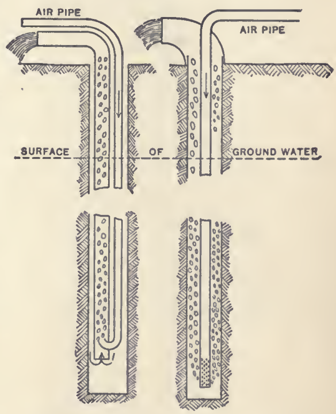


FIG. 55.—AIR-LIFT PUMPS.

diameter for 30 feet below the surface, however, a deep-well pump of the same capacity could have been used. An advantage of the air-lift pump is that the moving parts are all above ground and such as to require little attention; and the pipe below the surface needs little or no attention and can easily be removed if necessary.

The air-lift is most useful where the quantity of water is the chief concern, and the first cost is of more importance than the cost of operation. It is the best pumping appliance on the market for obtaining a large quantity of water from a small hole. The facility with which an air-lift plant can be applied to a well and altered as to depth of air-pipe and other conditions, and its freedom from injury or wear by muddy or gritty water, causes it to be especially adapted to testing the flow of deep wells.

As in the case of steam-pumps, air-lifts—the chief details of which are patented—are generally put in by contract. This should stipulate not only the delivery per day, but the efficiency of the entire plant. Probably 5 or 6 per cent thermal efficiency is the greatest which can be obtained by present methods; or 25 to 35 per cent mechanical efficiency for combined air-compressor and lift.

QUERIES.

27. A direct-acting pump has a water-cylinder and piston 18 inches diameter, a steam-cylinder and piston 14 inches diameter. From it a 12-inch pumping-main one mile long leads to a reservoir 150 feet elevation above the pump; and a 16-inch suction-pipe connects the pump with the water in the suction-well, which is 100 feet distant from and at an elevation 25 feet lower than the pump. If the efficiency of the pumping-engine is 90%, what pressure of steam must be exerted on the steam-piston to start pumping at the rate of two million gallons per day? the pump attaining this rate from a state of rest in one minute. New cast-iron pipe; no valves or other obstructions; mains filled with water.

28. From the table on page 332, records of the sixth day, find

the amount of ash in the coal; the amount of dry steam created by one pound of coal; and the percentage of water in the steam. How many thermal units did a pound of dry steam contain on each of the six days?

29. Suggest other methods than those mentioned in Art. 83 for utilizing more of the heat obtained from the coal burned under a boiler.

30. Referring to the table on page 339, from (44) and (45) determine the average heat of the feed-water, the temperature of steam in the boiler being 365°.

CHAPTER XV.

DESIGNING.

ART. 87. COLLECTING THE DATA.

THE problem presented to the designing engineer is generally: Given a certain city to be supplied or territory to be irrigated; to decide upon a source of supply which will meet the requirements as to both quality and quantity, to properly develop this source, conduct the water to the point of utilization, and arrange for its distribution to meet the various requirements demanded of the system.

If a city supply is to be obtained, the first problem presented is the population to be provided for; and the solving of this requires that all past records of population be obtained. These can generally be furnished by the city officers or gleaned from reports of the Boards of Health, School, and Police, and State and U. S. census reports.

A map of the city should be obtained showing curb as well as property lines, and the location as far as possible of all underground structures within the street limits, gas-pipes, fire-wells, subcellars or vaults, etc. A convenient scale for such map is 200 feet to 1 inch, but if the size of the city is such that this scale would necessitate the use of paper more than 3 feet wide it may be better to use a scale of 250 or even 300 feet to 1 inch. It is inadvisable to use a smaller scale than this, and if the resulting map is still too large for the paper it may be necessary to spread it over two or more sheets, when the use of the 200-foot scale is advisable. Extreme accuracy in this map is not necessary for the planning of the system, but for preparing the estimate of cost it is desirable that the lengths of pipe may be scaled from the map with tolerable accuracy. It is advisable to limit the possible error in lineal dimensions to 0.5 per cent.

A topographical survey of the city can be used to advantage, but the benefits accruing do not ordinarily warrant incurring the expense for this purpose alone; and a few levels taken at the lowest and highest points of each section of the city will generally be sufficient.

The problem of fixing upon the source of supply generally involves the comparative consideration of all possible sources. The water of streams or lakes should be submitted to a chemist and bacteriologist for examination, as should samples of water from test-wells if there is a probability of using ground-water. The banks of streams and lakes, and of their tributaries, should be examined for sewers, drains, outhouses, slaughter-houses, or other sources of pollution. If a surface-supply is in question, the best location for the reservoir will require a survey of the limiting line of the catchment area; a line of levels along the valley giving elevations relative to the city; test-borings at proposed reservoir-sites to determine the character of foundation obtainable; and an accurate topographical survey of the dam-site and of the reservoir-location above it, that for the dam generally locating 1- to 5-foot contours, while 5- to 25-foot contours may be sufficient for the reservoir-site. A few test-pits should be dug in the reservoir-site to determine the depth reached by organic matter in the soil; and the location of all swamps, ponds, stables or cow-houses, buildings, roads, and other natural and artificial features should be determined. It will be necessary also to learn who are the owners of the property in question, and in fact of all which is in the catchment-basin.

If ground-water is considered, the geology of the surrounding country should be carefully studied and the location and history of all wells in the neighborhood—their depth, character of strata which they pierce, and quantity and quality of flow from year to year. If the upper outcrop of the water-bearing stratum is within 25 or 30 miles, especially if the formation be glacial, the probability of contaminated water entering it should be investigated.

If the supply be from a storage-reservoir, the best conduit line between this and the city should be determined by survey. If an open conduit be employed, the survey will assume much the character of a railroad survey for a road with its grade and terminals fixed.

The location of pumping-station, standpipe, distributing-reservoir, and some other features of the system will call for special investigations and surveys, the latter mainly consisting of measurements of the distances and relative elevations between these and the city.

Estimation of the quantity of water flowing in rivers is made by obtaining with either floats or current-meter the velocity in a cross-section whose area has been measured (see Art. 65, (63)). The measurement may be taken with a graduated rod if the water be nowhere more than 8 or 10 feet deep, or with a plumb-line if deeper than this. The best rod for the purpose is a round one about 2 inches in diameter throughout, weighted at the lower end, and graduated to feet and tenths. The plumb-line should be braided of linen or silk, and graduated after having been soaked, and while suspended with the plumb immersed in water. The boat should, while measurements are being taken, be kept in line between range-poles on the opposite banks on the line of cross section (which should be normal to the stream-flow), and the location of the boat determined by triangulation from two transits in known positions, angles being taken by each simultaneously with the soundings. The depths obtained are referred to the water-surface, whose elevation is known.

The yield of a small stream may be obtained by use of a weir (see Art. 65); that of an artesian well, by catching the flow in casks of known capacity, and noting the number of times they are filled in a given period, or by receiving the flow in a flume and measuring it by a weir. The natural flow of wells is generally increased, both during the test and for practical use, by a pump, either surface, deep-well, or air-lift.

The test of wells should extend over at least a week, with several gaugings daily, if an estimate at all accurate is desired; and tests are sometimes continued for a month or more. River gaugings should be made during the lowest water if possible, and compared with calculations of the probable minimum yield to be expected from the drainage area. Gaugings of certain rivers extending over long periods of time can be obtained from government reports or the records of water-power companies, canals, or others; but such rivers are unfortunately few.

All obtainable rainfall data should be collected; and one or more rain-gauges should be established in the catchment areas under consideration as soon as the investigation is begun, and maintained throughout the time of the investigation, designing, and construction; and it will be desirable to continue their use on the watershed adopted, after the plant is in service.

The above applies largely to irrigation systems also. But the quality of supply does not demand such careful investigation, and the quantity depends upon the area to be irrigated rather than population. In most cases the aim is to obtain all the water possible, more land than it will irrigate being generally available for farms.

ART. 88. SELECTING THE SUPPLY.

If the city is upon or near a river or lake, this forms the most obvious source of supply. Unfortunately there are few in the settled parts of the country which are not more or less polluted; and there is a probability that settlements will before long be established along the rivers in sections not now populated. Many cities which originally drew their supply from their own river-fronts have been forced to abandon this supply on account of the fatal epidemics of disease clearly traceable to it; and many others are eagerly searching for purer supplies for the same reason. A clearer foresight and shrewder economy would in many cases have led to the acquiring of drainage-basins and reservoir-sites when they could have been obtained at a much lower cost than must now be paid for them. Whatever the opinions of the citizens, it would seem to be the engineer's duty to place the river as the last rather than the first alternative source, and to plainly and forcibly state to the public his reasons therefor. If river-water is to be used, purification, either immediate or in the future, should be provided for.

The present quality of a river supply may be learned approximately by chemical and bacterial analyses, in connection with a searching investigation for all causes of contamination (Chapters II and VII). It is desirable to obtain analyses not only during ordinary periods of flow, but during low and high water also. In determining the quantity of flow in a river it must be remembered that low water generally lasts for several days or weeks, while the maximum rate generally continues but a few hours. If no storage is to be supplied, particular care must be taken to learn the very lowest rate of flow attained by the river, and a margin of safety should be used in connection with this.

In testing the quality of a ground-water, analyses should be taken during the quantity test; and particularly at the end of this, since the quality often changes as the flow continues, owing to differences in the composition of the near and distant parts of the water-yielding stratum.

Before or while making exhaustive tests of the quality and quantity of water available from different sources, an estimate of that required should be prepared. The estimate of population can be approximated as suggested in Art. 12; a considerable extra allowance for growth being made for small cities. The decision as to quantity of consumption per capita is a most difficult one. If meters are to be used, and every effort made to keep the consumption within reasonable limits, 60 gals. for residential and 80 to 100 for factory towns should be sufficient. If no efforts in this direction are made, however, the consumption may reach any figure short of a thousand gallons per capita, and the only logical conclusion would be to furnish all the water available.

In case a water-supply of good quality can be obtained in sufficient quantity for domestic consumption only, but water of a more polluted supply from a river is also available, the latter may be used for an auxiliary supply for factories, street- and sewer-cleaning, public fountains, and similar purposes; although great care should be taken that it be excluded from all faucets or other contrivances by which it might be obtained for drinking-water by careless citizens. Such a supply, for fires only, has been introduced at Milwaukee, Detroit, Buffalo, Boston, and Cleveland; and for manufacturing purposes at New London, Conn. Salt water has been used for street-sprinkling and sewer-flushing in a number of English cities, where it is said to prove very satisfactory; one gallon of sea-water laying the dust as effectively as three or four of fresh. New York City has discussed the advisability of introducing an auxiliary salt-water fire system for the business section. The chief reason for introducing an auxiliary fire system is not, however, scarcity of supply, but the possibility of thus obtaining greater pressure and rate of discharge; and the use of such a system in no way affects the selection of a source of supply for domestic consumption.

In searching for a watershed to supply surface-water, it is desirable to find one from which the water can be led by gravity, and this will generally be at the head-waters of a stream passing through or near the city, or of one of its tributaries. This shed should be comparatively free from occupants, and preferably wooded and without swamps. The drainage area necessary may be calculated by dividing the maximum annual supply desired plus evaporation from the reservoir by the expected yield per square mile. This yield may be the average annual yield, but not unless a very large storage-reservoir is provided. It would be better to use the minimum average yield of three or four years. Great care and judgment should be used in estimating the rate of yield, all rainfall, evaporation, and stream-flow data available being used as outlined in Art. 31. The most reliable data are those obtained by careful run-off or stream-flow gaugings on watersheds in the same section of country and general topographical location, correcting these for different areas of water-surface if necessary. If no such data have been obtained, use may be made of the general rainfall and run-off records for that section of the country, as given in Chapters V and VI or obtained from government records. The less reliable and specific the data, the greater the amount of surplus catchment area which should be provided above that estimated.

The size of catchment area having been determined, a point should be looked for in the drainage valley which offers a favorable location for a dam and reservoir and above which the extent of drainage area is at least that found necessary. This point should also be at such elevation above the city or irrigation-fields that water can be delivered there by gravity and with the desired pressure. If one area cannot be found sufficiently large to furnish the entire supply, several may be used, the nearer they are to each other and to the point of utilization the better. A large city may be compelled to go a long distance for the necessary supply; as did New York City; and Liverpool, whose supply is impounded 68 miles distant. Philadelphia, also, is considering the advisability of going to the head-waters of the Delaware River for her supply.

The best reservoir-location is one at which the desired quantity can be stored on the least surface area, and where the shores of the reservoir will be fairly steep; also where the impounding-dam may be short, have a solid foundation, and be constructed largely of material found near at hand. A bowl-shaped basin among the hills or mountains, having a narrow gorge for its outlet, best fulfils these conditions. Reservoirs have been built in a rolling country, however, where the length of the dam is as great as that of the reservoir, and the maximum depth but a few feet; but such reservoirs lose much of their stored water through evaporation and seepage, and are liable to acquire undesirable qualities.

The geological conditions of a reservoir-site should be such that there is little danger of leakage; the strata being synclinal rather than anticlinal, and containing no fissures or open faults.

In searching for a ground-water supply, if shallow wells or filter-galleries are proposed no data concerning the flow to be expected can be obtained from points more than a very few miles away; except as these have thrown light on the general laws of ground-flow. Test-wells and pumping should form the chief basis of estimating the flow. In this, as in tests of deep wells, pumping should be continued for a considerable time; and it is desirable to sink another observation-well at some distance—say 1000 feet—away, and note the fluctuations of ground-water level in it. If this constantly falls while pumping at a uniform rate is continuing, it indicates that not only is all of the regular ground-flow being pumped, but the ground-storage is being drawn upon. The water-surface in the observation-well may be lowered during the first day or two of pumping, even if this be less than the rate of ground-flow, but after the conditions of flow to the well become established it should remain stationary. If the pumping could be continued at the maximum rate which it is possible to maintain without drawing upon the storage, this would equal the ground-flow at that time. But it must be remembered that shallow wells are much more affected by droughts than are deep ones.

Of deep wells also the only sure knowledge can be obtained by actual test; but if there are others in the vicinity, much may be judged from their performance. If these tap water-bearing sandstones or other rock strata, the probabilities are that wells of similar capacity and characteristics may be driven to the same rock over a considerable section of country; as in the case of the Dakota, St. Paul, and other sandstones in the North-central States. If the wells are in glacial deposits, the extent of the strata which they tap is very uncertain and can be known only by actual investigation.

If the amount of water desired is small, the possibility of obtaining it from springs should not be overlooked, and suitable ones should be searched for. On the other hand, the limit of area to be investigated for supplies is generally limited by the small amount of money available for constructing conduits in a small plant.

It will generally be desirable to make comparative estimates of the cost of using each of the available sources; never forgetting that quality should outweigh any financial considerations where the water is for domestic use.

ART. 89. THE GENERAL DESIGN.

The location of the source of supply will generally determine whether the system will be a gravity or a pumping one; and the two may be combined, either when there are two sources of supply, or when the territory to be supplied is at different elevations. In the latter case there will generally be a high-service distributing-reservoir into which the water is pumped; in the former the pumping and gravity supplies may be distributed by practically two separate systems, or, as is more often the case, pumping is resorted to to supplement the gravity supply when this becomes deficient. If the storage-reservoir is at a great distance from or elevation above the city, a smaller distributing-reservoir is desirable. The top of a hill or ridge immediately above the city and 150 to 300 feet higher forms the best location for such a reservoir; or a flat slope on a side hill may be used. If such a location is not obtainable and the storage-reservoir is more than 300 to 350 feet higher than the city, in which case a distributing-reservoir is desirable to prevent a pressure head dangerous to ordinary plumbing, this may sometimes be located lower down the valley of the impounded stream and formed by a small dam across this valley, the old stream-bed serving as a conduit to connect the two reservoirs. But if the watershed above the distributing-reservoir is liable to pollute the supply, the distributing-reservoir should be placed out of the channel of flow.

If pumping is employed, a reservoir or standpipe is extremely desirable, and should be omitted only when it is impossible to obtain the money to pay for it; and even then its future construction should be provided for, and carried out as soon as possible. Whether reservoir or standpipe is provided will generally depend upon the local topography. A reservoir is to be preferred if a hill sufficiently high for one is near the city; but if there is no such hill, a standpipe will be necessary to give the desired elevation and head of water. A standpipe may be placed anywhere in a city, as it occupies but little ground-space, and is more efficient the nearer it is to the centre of the district most urgently requiring fire-protection—generally the business or manufacturing district.

The elevation of the reservoir or standpipe will determine the volume and range of fire-streams in the city, since the pressure at the nozzle, plus the friction loss in the hose and pipes between this and the reservoir or standpipe, must equal the difference in level between the nozzle and the surface of the stored water. (The velocity-head will be so small as to be negligible.) Knowing the volume and range of fire-streams and the number of simultaneous streams desired, and the size and length of pipes between the nozzle and supply, the velocity in these last can be calculated and from this the friction-head; and friction-head plus nozzle pressure-head plus elevation of the principal fire district will give the desired elevation of the water in reservoir or standpipe. A standpipe should be carried at least 15 to 20 feet above this elevation, since the water-level falls rapidly during fires.

In the case of a large city in a level country such a condition is practically impossible unless a number of standpipes be scattered throughout the city. (In such a location the supply will in every case, probably, be by pumping.) It then becomes necessary to obtain additional head by pumping during fires. This may be effected either by the use of steam fire-engines, in which case the head in the water-mains must be sufficient to supply water to the fire-hydrant at the maximum rate at which the fire-engines can use it; or the pressure and volume given by the water-works engines may be increased when a fire breaks out, the reservoir or standpipe being at the same time shut off from the system. The objections to the latter plan are, that a longer time is generally consumed in raising the pressure at the pumping-station than in starting a fire-engine; and pipes and fixtures which showed no sign of weakness during ordinary service-pressure are apt to be broken by the excessive fire-pressure. Both these objections are found to be most seriously applicable to four out of five of the smaller cities and towns.

If the supply for a pumping system be from ground-water or other source not subject to occasional pollution with considerable suspended matter, or if it be filtered, an excellent arrangement is to place the pump and the reservoir or stand-pipe on opposite sides of the city, and use the direct-indirect system. If the main break near either pump or reservoir, the other of the two can then supply the entire city; and with a proper arrangement of piping it will be impossible for any one break to deprive more than one city block of its supply. Also a more generally uniform fire-pressure can be obtained thus than if pump and reservoir were on the same side of the distribution system.

If the supply is from a river, however, and is not filtered, the pumps should preferably discharge directly into reservoirs, that the water may have some opportunity for clarification when muddy before being delivered for consumption. At least two reservoirs, or one double reservoir, should be provided, and these should always be kept full while the river-water is clear, to furnish the supply while it is muddiest; being if possible of such size as to furnish the supply until the river-water again becomes clear. The further a river-valley extends above the intake the longer will this period of turbidity continue. The period will vary widely with the character of soil and slope of the watershed, but for average conditions the turbidity will probably continue, after a heavy rain ceases, one day for each 30 to 50 miles of river above.

Where there is no location for distribution-reservoirs in a river pumping-system, it is advisable to place settling-reservoirs near the river at the pumping-station, and pump into these, and from them to a standpipe by the direct-indirect system. The same pumps could be used alternately for pumping from river to reservoir and from reservoir to standpipe; but greater economy could generally be obtained by using for the former centrifugal pumps of high efficiency, and reciprocating pumps for the service-pumping, both being run by the same steam-plant.

A by-pass should be provided around the distributing- and settling-reservoirs from the pumping to the distribution main, to be used for direct pumping if it is desired to clear or repair the reservoirs, or to increase the pressure during fires.

An uncontaminated supply is more satisfactory and reliable than a purified one, but the latter is infinitely preferable to a contaminated one unpurified. If purification is necessary, the works may be immediately below a storage-reservoir, but would generally be more accessible just above a distributing one. In the case of a pumping supply, an excellent location for a gravity purification-plant is between the sedimentation-basin and the main pumps; or, if there be no sedimentation-basins, the purification-plant should be between the river and the pumps, secondary pumps raising the water to it from the river if necessary. If pressure mechanical filters be used, these may be placed near the pumps on the pumping-main, water being pumped through them directly from the river. But the life and efficiency of both pumps and filters will generally be increased by first clarifying the water in a sedimentation-basin.

Art. 90. Details of Gravity Head-works.

The investigation of a watershed calls for accurate data concerning its area and form. These may sometimes be obtained from State or government works; but in most cases it is necessary to make the required surveys. The most important of these is a careful running of a transit-line around the boundary of the catchment area, and a line of levels (referred to the city elevation) and transit-line up the bottom of the drainage valley. From a map prepared from these data the catchment area above any proposed dam-site can be scaled off, and its total yield estimated; and the head available at the city or irrigation-fields due to the elevation of the dam may be learned.

At the dam-site selected, borings or test-pits should be made to determine the character of the underlying strata. These should be located at several points over the site to be covered by the dam, and also over the reservoir area. An impervious stratum should not be pierced by the test-holes; but if found to exist under the dam- and reservoir-site, other test-holes a short distance below the dam-site should be carried through or several feet into the impervious stratum to learn if its thickness be sufficient. If this be more than one third the height of the dam and the material be solid, it can probably be relied upon as a foundation. If less than this be found, with a soft or porous material underlying it, further investigation should be made before finally selecting the site; although an earth dam would probably be safe upon a considerably thinner rock-foundation if free from seams or cracks. The danger from a thin impervious stratum is twofold: its strength may not be sufficient to support the dam, or the head of water created may cause a leakage through it.

A thorough investigation should be made, if necessary, to render certain and exact the information concerning the geological formation at the dam-site. Such conditions as are shown in Fig. 56 are inadmissible. In *a* leakage would almost surely occur under the dam. In *b* it would penetrate the rock and the under-stratum in consequence of the head created behind the dam, and either a leak result or the under-stratum become saturated and soft; in either of which cases the rock would be likely to yield under the weight of the dam. In *b* the dam should be carried down to a thick stratum of unseamed rock or moved to another location, as below the point *c*. In *a* the core-wall may be carried to the lower rock-stratum, unless this also is underlaid by a porous stratum which outcrops within the reservoir area.

Not only the bottom, but the sides also of the valley should be carefully examined for seams, porous strata, or

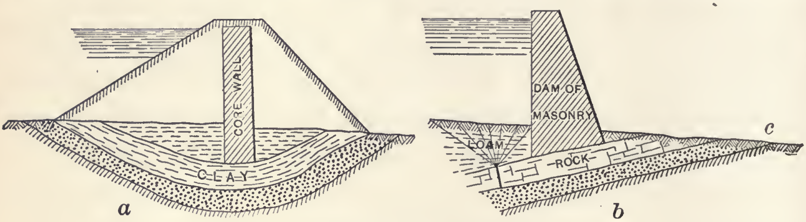


FIG. 56.—UNSUITABLE DAM-SITES.

other conditions which would cause leakage under or around the dam.

From the size of the catchment area and the estimated yield, the amount of storage required can be estimated (see Art. 33). This capacity is then used to learn the height of dam necessary. A carefully prepared contour-map of the reservoir-site is made, the contour interval being from 1 to 5 feet, depending upon the steepness of the ground-slope. The dam is approximately located on this. Beginning at the lowest contour, the area included by this above the dam is measured by a planimeter or otherwise; and the same is done with each successive contour; the volumes embraced between successive contour-planes is calculated, and when the sum of all these volumes below a certain plane equals the amount of storage desired, this contour-plane will be the elevation of the reservoir spillway. The water will rise such a distance higher than this as is necessary to discharge the maximum floods over the spillway.

If a storage-reservoir of the desired capacity would require a very high dam, it may in some cases be cheaper to construct two reservoirs, either one below the other in the same valley, or in different valleys.

If the back-water from the reservoir will cause shallow ponding of water along the banks of the tributary stream, the channel of this should be walled in and the banks raised above the water-level of the reservoir; and the same treatment, or excavation and paving, or more often both combined as at *b*, Fig. 57, should be applied to all shallow water within

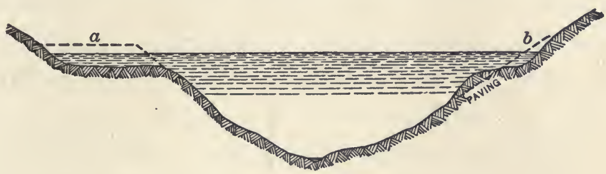


FIG. 57.—RESERVOIR CROSS-SECTION.

the limits of the reservoir. If a shallow pond is formed, as at *a*, it will generally be better to fill this entirely, as shown by the dotted lines. Sufficient material for this will in most cases be obtained in clearing off the reservoir-site and grading the banks. The last few feet nearest the slope of this fill, however, should be of gravel or earth free from organic matter. Embankment-slopes should be paved with dry slope-wall; and it is desirable to treat the entire shore of the reservoir in this manner.

The decision as to the material of which a dam should be constructed will, to a large extent, depend upon the size of the dam, character of the foundation, the material at hand, the accessibility, and the money available. If bed-rock is near the surface and can be quarried for construction near by, a masonry or rock-fill dam is easily possible. If hardpan or clay is near the surface and good embankment materials near at hand, an earth dam is practicable. If rock exists for a foundation and for masonry, and also soil adapted to embankments, either construction is possible. Where labor, cement, and sand are reasonably cheap an earthen embankment more than 70 or 80 feet high will probably cost more than a masonry dam, owing to the great width of the base and volume of material in the former. If the dam is short and the spillway must be placed in it, this, which must generally be of masonry, may form so large a proportion of the dam that constructing the whole of masonry as a weir-dam would cost but little more, and would be much safer.

If no rock, hardpan, or clay exist within 40 or 50 feet of the surface to form a foundation and impervious bottom for the reservoir, it will probably be well to endeavor to find another site. If the site selected be not underlaid with such a stratum of considerable thickness it would be useless to place there a tight masonry or earth dam of considerable height; but one 10 or 15 feet high may be constructed on a timber foundation protected by sheet-piling (see Fig. 25, page 244), although this construction is to be avoided if possible. In such a location a rock-fill or timber dam may be used to divert the water if no storage be required.

The general form and cross-section of dams has been already considered in Chapter XII. The cubic contents of a given dam can be most readily ascertained by the use of a contour-map of the site, to a large scale. The method is illustrated in Fig. 58. On the prolonged axis, *AB*, of the dam a cross-section of the same, *CDEF*, is drawn to the same scale as the contour-map and with *AB* as its vertical axis. If the crest of the dam is to be at an elevation, say, of 125 feet above datum, horizontal lines are drawn at intervals of 5 feet (if this be the contour-interval) beneath this and numbered with their proper elevations. The points of intersection of the 120 line are projected down to the map, and where the projection lines cut the 120-foot contours, as at *e*, *f*, *g*, and *h* are points in the junction of the dam and the earth surface. *ef* and *gh* are connected; *a*, *b*, *d*, and *c*, are found and connected in the same way, and *a* is connected with *e*, *b* with *f*, etc., the outline *aekfbdglhc*, being thus formed. The areas *acdb*, *ehgf*, etc., are now measured and treated as parallel section of a prismoid in calculating the cubic contents of the dam. The contours *ac*, *bd*, *fg*, and *eh*

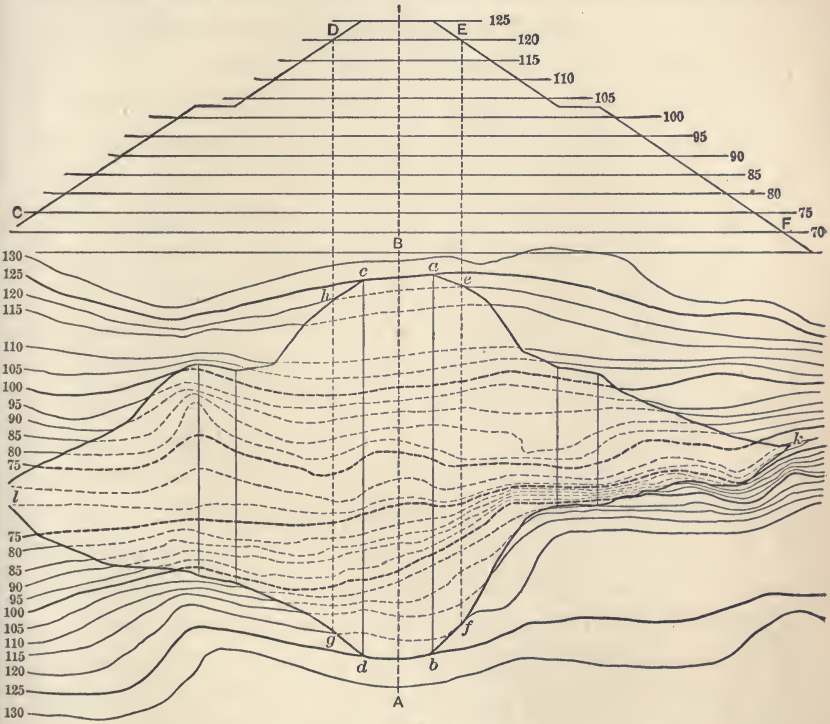


FIG. 58.—ESTIMATING CONTENTS OF EARTH DAM.

are taken as the ends of the horizontal sections. Instead of using surface-contours, those of the ground as cleared or excavated, or of the rock, if it be a masonry dam, should be used; or the surface-contours may be used, and to the resulting calculation may be added the amount to be excavated and then refilled as embankment or masonry.

The upper face of earth dams should be protected from wash, weeds, and ice by paving. This is generally in the form of a dry wall 12 to 24 inches thick, composed of a single layer of flat stone, carefully laid by hand on a bed of coarse gravel or broken stone 6 to 20 inches thick. A bed of concrete 4 to 12 inches is better than the broken stone; and when this is used brick is in many cases substituted for the dry stone wall.

The concrete may be made practically impervious if this is desirable; or asphalt may be used to obtain a tight lining in one of the ways described in Art. 73. The outer slope of a dam is generally sodded to prevent wash by rains; although in some cases this also has been paved. If there is a berme, a paved gutter should be placed along the inner edge of this, leading to a drain which discharges upon the natural surface below the dam.

The face of a masonry dam may be vertical, sloping, curved, or stepped. The first is applicable to very low dams only, the second to dams up to 20 to 30 feet in height if never overflowed, but only 10 or 15 feet if ever acting as weirs. Weir-dams from 10 or 15 to 20 or 30 feet high should be stepped in front. Dams more than 30 or 40 feet high should be given forms calculated for greatest economy; as the "economical" or the new Croton profile in Plate XV, page 258; and if to act as weir-dams should have an ogee or other curved face, similar to the Austin or Holyoke dam (Plates XIV and XV, pages 256 and 258). The face-stones may all rest on horizontal beds, but in weir-dams, at least, it is better to make the joints radial, as in Plate XIV.

The crest- or cap-stones should be heavy and substantial, and where there is great danger from heavy logs and ice they may be bound together as in the Holyoke dam. For spillways in storage-reservoirs, however, the construction shown in Fig. 59 will generally be sufficient, the steps being sometimes replaced by an ogee face for high spillways. If the main dam be of earth and a masonry core-wall be used, it should be joined to the masonry of the spillway; or if there be no core-wall, a spur-wall should be carried for some dis-

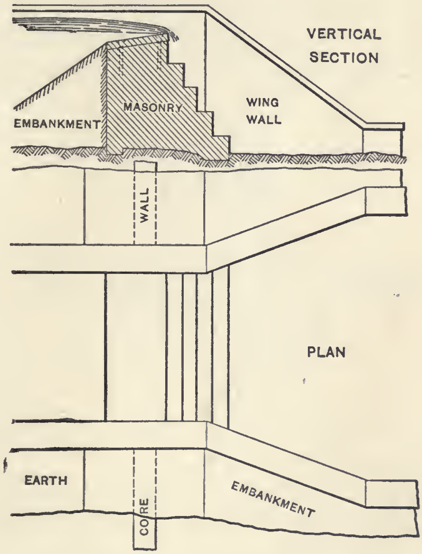


FIG. 59.—SPILLWAY IN EARTH EMBANKMENT.

tance into the centre of the embankment from each end of the spillway. A spillway over natural bed-rock at the head of the dam is generally preferable to a weir.

The cut-off flanges for pipe passing through an embankment or dam may be made as shown in Fig. 60. A thin sheet of lead or other yielding and durable material should be placed between the casting and the pipe, and between the flanges of the casting, so that no water may work its way between the pipe and cut-off flange.

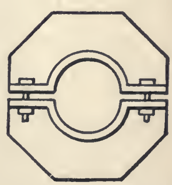


FIG. 60.—FLANGE FOR PIPE IN EMBANKMENT.

But one pipe, an outlet main, may be used in small reservoirs, a blow-off branch being placed on this just outside the reservoir and controlled by proper valves. The inner end of such pipe must be at the lowest point in the reservoir and flush with the bottom, in order that the water may all be drawn off. It should, however, be supplied with a removable extension rising at least a foot or two above the bottom, to exclude mud and other deposits from the outlet-main during service. This riser should terminate in a

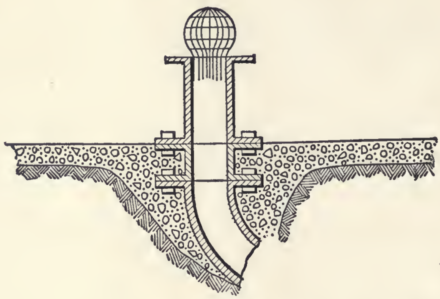
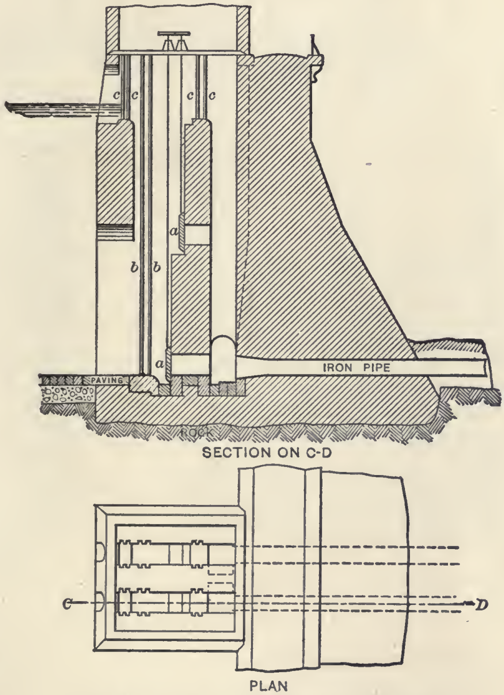


FIG. 61.—OUTLET-PIPE FOR SMALL RESERVOIRS.

screen to keep out fish and other large matters. The gates controlling the flow from the reservoir can be placed outside the embankment in a manhole or gate-house. This is the simplest arrangement practicable, and is applicable to the smallest reservoirs only; and even in these a separate "mud-" or "waste-" pipe is preferable.

Such a screen as is here referred to cannot well be cleaned without removing it entirely. Also if any break or leak occur in the outlet-pipe above the gate, the flow through it cannot be stopped except by emptying the reservoir, before which the bank would probably be destroyed. It is hence better to provide gates on the outlet-pipe *inside* the reservoir; and duplicate screens which can be removed for cleaning and replaced without leaving the outlet unprotected; and this is always done in large reservoirs. A vertical flat screen does not become obstructed so quickly as a horizontal one and is more easily removed and cleaned, and is consequently preferable. Such a screen may be made of, say, No. 10 copper wire, $\frac{1}{4}$-inch mesh, in a steel frame; but a copper plate, punched to form a screen, cleans more easily. Wire screens in frames of almost any size are carried in stock by companies dealing in valves and other water-works supplies.

The form of gate-house employed is ordinarily on the general plan of that shown in Fig. 62. Any one screen can



a,a, SLUICE GATES. b,b, SCREENS. c,c,c,c, FLASH-BOARDS

FIG. 62.—GATE HOUSE.

(East Branch Reservoir, New York.)

here be removed, a duplicate one meantime being in service. The discharge from the reservoir can be passed through any or all outlet-pipes, each of which is controlled by a gate worked from above by a stem and wheel. Water enters the gate-house from the reservoir through several openings in the wall *AB*, which are placed at different elevations and controlled by sluice-gates (Fig. 63) to permit of drawing water

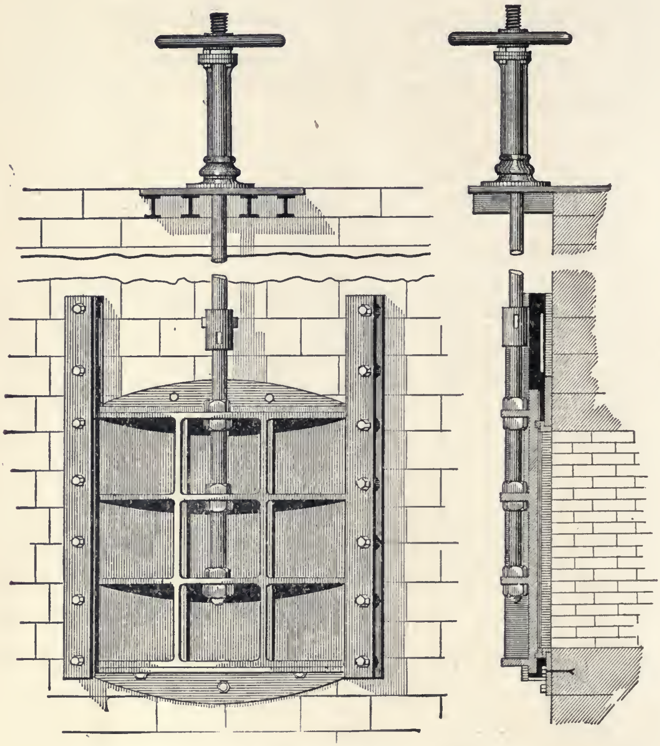


FIG. 63.—RESERVOIR SLUICE-GATE.

from any depth, and thus securing the purest and coolest. The lowest one should be a little lower than any part of the reservoir bottom, and is used for emptying the reservoir, or drawing off the foulest water before the "turn-over," through a mud- or waste-pipe, which discharges below the dam, generally in the old creek-bed. The outlet-pipes and gate-house demand the most careful attention, the firmest foundation, and the most substantial construction. The foundation of the gate-house should rest on the same material as does

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the dam or core-wall. The bottom should be absolutely water-tight; if not on rock, a broad thick bed of concrete forms probably the most desirable foundation. The wall of the gate-house should be of small uncoursed ashlar, in the best of Portland cement-mortar, with walls of such thickness that no stone reaches entirely through them, the idea being to make them water-tight and sufficiently strong to support the outside water-pressure when the gate-house is empty, or the pressure of ice. The construction of the gate-house should be such as to prevent its injury by ice if the water in the reservoir rises or falls while frozen over, or by the expansion of ice between it and the dam. To insure this it is generally better, if the dam be of masonry, to have the gate-house built against or as a part of it. In the case of earth dams, the gate-house is generally at the inside foot of the embankment, and communication with it is afforded by means of a foot-bridge from, and on a level with the top of, the embankment. In a few cases the gate-house has been formed by sinking a well in the rock at one end of the dam, and filling it through a tunnel or open cut carried from it into the reservoir; this plan avoiding the weakening of the dam by the outlet-pipes. In the Oak Ridge (East Jersey Water Company) Reservoir this plan was adopted, the gate-house and inlet- and discharge-channels being more than 40 feet deep in solid bed-rock. The gate-house is floored over, generally on a level with the top of the dam; and is covered with a small building to protect the gates and exclude meddlesome intruders. This building is frequently given an attractive appearance.

The outlet-pipes should make a water-tight connection with the gate-house, and be placed on a perfectly firm foundation, that they may not be broken by settlement. The bottom of the reservoir, for several feet in front of the gate-house inlet-opening, should be paved to prevent washing.

Where a storage-reservoir is not necessary and a diverting-dam only is provided, the water is generally drawn off from at or near the surface of the stream. For this purpose, instead of a gate-house and pipes, an opening is ordinarily made in one end of the dam, leading to an open conduit; this opening being controlled by sluice-gates. In addition to the gates it is generally desirable to place in front of them an upright grooved frame with flash-boards sliding in the same, and a contrivance by which an opening can be left between flash-boards at any elevation at which it is desired to draw off the water. This opening should generally be kept a foot or two below the water-surface, that both floating brush, leaves, etc., and the heavier matter carried in suspension may be excluded from the conduit. As sand and gravel will probably collect in front of the flash-board weir, another opening is generally provided in the bottom of the dam near this weir for flushing out the deposit at intervals.

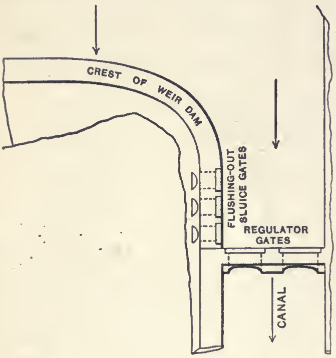

FIG. 64.—HEAD-GATES AND FLUSHING-OUT SLUICES.

ART. 91. PUMPING-STATION AND INLET DETAILS.

A pumping-station should generally be located as near as possible to the body of water from which it draws, to reduce both friction in the suction-pipe and the possibility of its leaking air and reducing the vacuum. The boilers should be above the reach of floods; and their foundation should be extremely firm, on account of the great weight to be supported, which may amount to 250,000 lbs. or more for each boiler. The pumps must also be protected from water, but it may be necessary for them to be below high water (see Art. 84), in which case they are generally placed in a water-tight pump-pit, as in Fig. 52, page 346. The walls of this may be of thick brick, concrete, or stone masonry, or of sheet iron or steel; or of iron and masonry, as in the illustration; and the well is usually circular in plan, this giving the greatest strength. To reduce the size and cost of the well the pumping-engines only are placed in it, and these are generally vertical engines. The stairway to the bottom of the well can be made to follow the wall in a large spiral. A water-tight joint must of course be made where the suction-pipe pierces the wall of this well.

If the pump is on the surface, the suction-pipe should be carried several feet beneath the ground-level before leaving the building, and be kept at this depth until reaching the water, to prevent its freezing in winter. The suction-pipe should be larger than the discharge; and if of considerable length should be of such size that, when all the pumps are in service, the velocity of flow through it will not exceed 2 to 4 feet per second for 12- to 36-inch suctions respectively. It should be laid with unusual care to obtain tight joints. It should not terminate in a river or lake wall nor near the bank or shore, as the water here is more liable to contain fresh sewage and floating matter than in mid-stream. The best location and design of the inlet will vary with the circumstances. It should be placed where the water is purest; in a lake especially this will generally be the furthest possible from the shore. It should be in deep water, as this is ordinarily cooler; but should not be near the bottom, as the most sediment is carried there. An excellent plan is that of terminating the inlet in a tower somewhat similar to a reservoir gate-house, there being several openings at different levels through which the water can be taken as desired. In the Nashville, Tenn., water-works this tower is hexagonal in plan, 10 feet interior diameter and 85 feet high, of stone masonry on solid rock. At Cincinnati is a masonry intake about 140 feet high, the range of river height being over 70 feet. The intake for the Chicago water-works is four miles from shore, formed of a circular double steel shell 70 feet inside diameter, the space between the shells being filled with concrete so as to form a wall 24 feet thick; the whole resting in 40 feet of water, and being 50 feet high to the bottom of a masonry superstructure. The St. Louis masonry inlet-tower is about 50 feet high, and is oval in plan, with an ice-breaker pointing up-stream. The Cleveland intake is of steel, somewhat similar to the Chicago one, 100 feet outside and 50 feet inside diameter; located in 49 feet of water, and connected with the shore by 26,000 feet of 9-foot tunnel lined with 12 inches of brickwork—the longest intake-tunnel in the world.

Such inlet-towers must be substantial and massive enough to resist water-currents, ice, or floating logs, and are consequently expensive. They are also an obstruction to the current in a river, and to shipping in a navigable water, in which situation they must be provided with a light-house.

Instead of a tower, a submerged crib is frequently used, especially in smaller plants. This should be placed where there is least danger of silting, and it is desirable to make its height about one third the depth of the water, with limits of 3 to 15 feet. This structure is essentially a wooden crib, weighted with stone and anchored to the bottom in some way to prevent movement by tides or currents, this being effected by surrounding it with coarse rip-rap, or piles, or bolting it to bed-rock. It may have openings on the sides; but it is in most cases preferable to make these tight and take water through the top only, which is provided with a coarse grating. The suction- or intake-pipe rises into the centre of this crib. Such cribs are used at Duluth, Minn., Erie, Pa., and many other places (see Fig. 65).

The total area of the inlet-openings in the crib should be considerable to prevent the formation of a vortex in the water

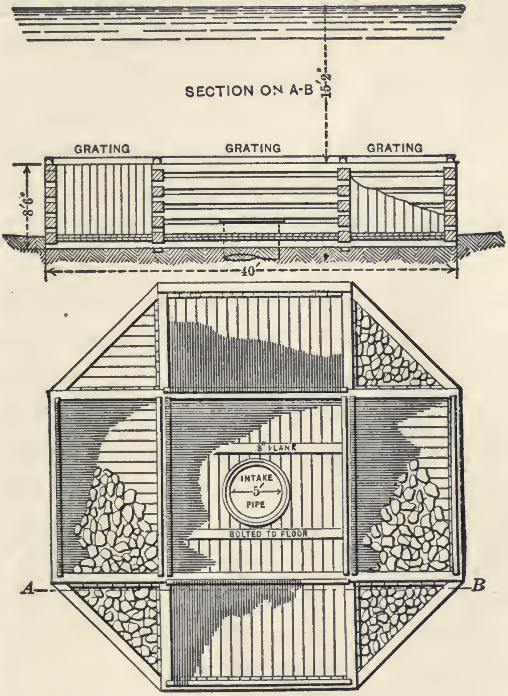

FIG. 65.—ERIE, PA., INTAKE-CRIB.

above, which will cause floating matter to be sucked into the main; or the entrance of "needle-" or "anchor"-ice, which closes the openings in the pier or crib, or in the intake-pipe, or may reach and stop the screens or even the pump. A very little motion will serve to carry the ice-needles down and into an intake-crib, and several cities have suffered water-famines for hours and days on this account.

Intakes for small plants in shallow water are frequently not provided with a crib, but are simply an upright extension of the suction-pipe, brought up to a short distance below the surface and surrounded with concrete or rip-rap, and in some cases with piles to protect it from boats or floating logs. A great number of small cities use such intakes, and they give fairly good service in many cases. In fact, if the water is quite shallow the use of a crib may be practically impossible in Northern streams, where the going-out of the ice would carry the crib with it. If the expense is not prohibitive it is, in such a case, desirable to place a low weir-dam just below the intake, to raise the water-surface, permit the use of a crib, and prevent damage to the intake from ice and floating timber.

To prevent the stoppage of an intake-pipe by ice several devices have been used. In one, compressed air is carried by a small pipe, which is laid in or fastened to the outside of the intake-pipe, and delivered under the horizontal screen which covers the intake-opening, the rising bubbles preventing the ice from collecting. In another, steam from the engine-room is applied in the same way. A "rotary strainer" has been used with success in some cases, this consisting of a revolving cylindrical screen at the mouth of the suction, which is caused to rotate by air forced to it from the shore, as above.

An intake-pipe or tunnel leads from the intake to a suction-well, from which the water is pumped; or the pipe may act as a suction-pipe and lead direct to the pump, this being the common arrangement for small plants. The Milwaukee water-works intake-conduit consists of a tunnel 3000 feet long, continued further into the lake by two lines of 60-inch pipe 5000 feet long laid in an 8-foot trench and terminating in 60 feet of water. Chicago and other large lake cities also take their supply through tunnels; but smaller supplies are generally drawn through pipes, either cast or wrought iron, laid in the bed of the stream or lake. These should be laid in trenches, to protect them from currents or shifting sands, and are sometimes covered with rip-rap as a further protection.

If filter-cribs are used (Art. 79), these take the place of intake-cribs (see Fig. 43). The intake-pipe is similar to that from an ordinary crib.

Art. 92. Details of Ground-water Plants.

Except where artesian wells flow under considerable head, it is generally desirable to be able to draw down the ground-water as low as possible in the well, thus increasing the flow. Since pumps can raise water by suction only 20 to 25 feet with any efficiency, it is frequently necessary to lower the pump to meet these conditions. If the water rises to within five or ten feet of the surface, a surface-pump may be used, placed in a pump-pit as described in the previous Article. The construction of this pit must be such as will exclude any ground-water.

If it is desired to draw the water down to a distance of more than 40 or 50 feet below the surface, requiring a pit more than 20 or 25 feet deep, such construction is not often employed, but deep-well pumps (Plate XVII, page 395) are lowered into the wells. An exception to this is the Rockford, Ill., plant (see Fig. 53, page 347), where centrifugal pumps are used in a pump-pit 80 feet deep. The air-lift is in some cases used in place of a deep-well pump; but the efficiency of both of these is so much lower than that of a good reciprocating pump, and the latter is so much more accessible for repairs, that its use is recommended wherever possible in any but very small plants, and may be the most economical in many of these. Where a large dug well is the means of supply, the pump may be placed in this, the boiler being on the surface, and the whole roofed over. This is likely to cause a pollution of the water, however, and a better plan in most cases is to place the pumping-station near the well, connecting the two by a suction-pipe, and roofing over the well.

If the wells are tube-wells, they are ordinarily coupled to a main collecting-pipe which passes by and near them, the short connecting branches being furnished with valve-gates, that any well may be put into or out of service. Collecting-pipe and branches should all be perfectly air-tight—a result often difficult of attainment.

In spite of all efforts some air is likely to leak into the collecting-pipe, and this should be removed in some way; especially if the collecting-pipe be connected directly to the pumps, as the presence of air in the pumps will cause them to "pound." The only practical method of accomplishing this seems to be to place an air-drum in the pipe at its highest point, and remove the air from this with a vacuum-pump. At a plant of forty-five wells and 5806 feet of suction-pipe in Lowell, Mass., two duplex vacuum-pumps, one $6 \times 8\frac{1}{2} \times 6$ inches, the other $7\frac{1}{2} \times 10\frac{1}{4} \times 10$ inches, were found necessary for this purpose in 1894.

The pump may be connected to the collecting-pipe and draw directly from the wells; in which case a sand-box or sand-interceptor should be placed between the pump and the wells to prevent sand from entering and cutting the pump-valves and plunger or piston. In a few cases the pump draws from a suction-well which has been dug to somewhat below the depth to which it is desired to lower the ground-water, the collecting-pipe being laid at about the level of the pump, and its vertical end carried down to near the bottom of the suction-well. By this plan the water is siphoned into the suction-well, which also acts as a small reservoir to permit of higher rates of pumping for short intervals than could be obtained directly from the wells. The suction-well also serves to intercept the sand. The collecting-pipe is apt to fill with air at its highest point, and this should be near the pumps and provided with an air-drum, and arrangements made for removing the air at intervals.

The distance by pipe from the pump to any well should be as short as possible, and the collecting-pipe of such size that the velocity of flow in it shall not be greater than 2 or 3 feet per second, that the friction loss between well and pump may be a minimum. The best arrangement for obtaining this result is to place the wells in a straight line at right angles to the direction of ground-flow, pass the collecting-pipe about 3 feet from each of these, and place the pump at the middle of the collecting-pipe. If two or more water-bearing strata are tapped by the wells, these may all be connected to one collecting-pipe; but if the water rises naturally much higher from one stratum than from another, it is better to provide a separate suction from each, and, if more than one pump is used, so arranged that either pump can draw wholly from either stratum or from both combined.

If deep-well pumps are used one must be placed in each well, and be driven by separate engines or working-heads. This will require a building of some kind over each well; and either an engine to each pump, or one engine transmitting its power by rope, belt, water- or air-pressure, or electricity to the various working-heads. For this reason deep non-artesian wells should be given large casings and powerful pumps, that their number may be reduced to the minimum.

The location of wells has already been referred to. It is generally desirable to drive them on low ground, as the suction-lift of the pumps is thus reduced at the least expense of pump-pit excavation. They should never be placed along the direction of ground-water flow, but as nearly as possible at right angles to it. The most desirable spacing will vary with the material, volume of flow, and depth to which pumps lower the water in the well, and can be learned for each case only by trial. It should generally be such that the ordinary pumping of one well will have little effect upon the simultaneous yield of its neighbors. This is particularly true of deep and expensive wells. At Galveston, Tex., thirty 7-inch and 9-inch wells, 750 to 850 feet deep, are located at intervals of 300 to 750 feet. The sinking of an additional well between any two others will, however, almost always increase the total supply, although it may be but slightly; and if the wells are shallow and cheaply driven they may be placed quite close together. At the Spring Creek Station of the Brooklyn water-works, one hundred 2-inch wells, 30 to 42 feet deep, are placed in two rows 14 feet apart, the interval along each row also being 14 feet; and the same spacing is used at the Jameco Station, where are one hundred and eighty-three 2-inch wells from 27 to 73 feet deep.

Each driven well is furnished with a strainer which is several feet in length; in deep wells this length may equal the thickness of the water-bearing stratum; in shallow wells it usually occupies the lowest 3 or 4 feet only. The strainer is a section of wrought-iron pipe pierced with holes or slits to admit the water, and generally covered with a gauze screen to exclude sand and gravel. The gauze should be protected by a heavy screen or thin tube of pierced metal surrounding it called a jacket. Probably the best strainer for sand is the Cook, which is formed by vertical or horizontal slits cut as shown in Plate XVII, *a*. At *b* is shown a drive-well "point," and a strainer before the gauze and jacket are put on; and at *c* is a point and strainer with which no gauze is used. Points are used when the well is driven into the ground by blows of a heavy hammer (called a drive-well). An open-foot well is "washed down" by forcing water into it and thus washing to the surface the soil beneath the pipe, which is forced down simultaneously.

In Plate XVII is shown a deep well with strainer and deep-well pump. At the top is shown the engine, to the

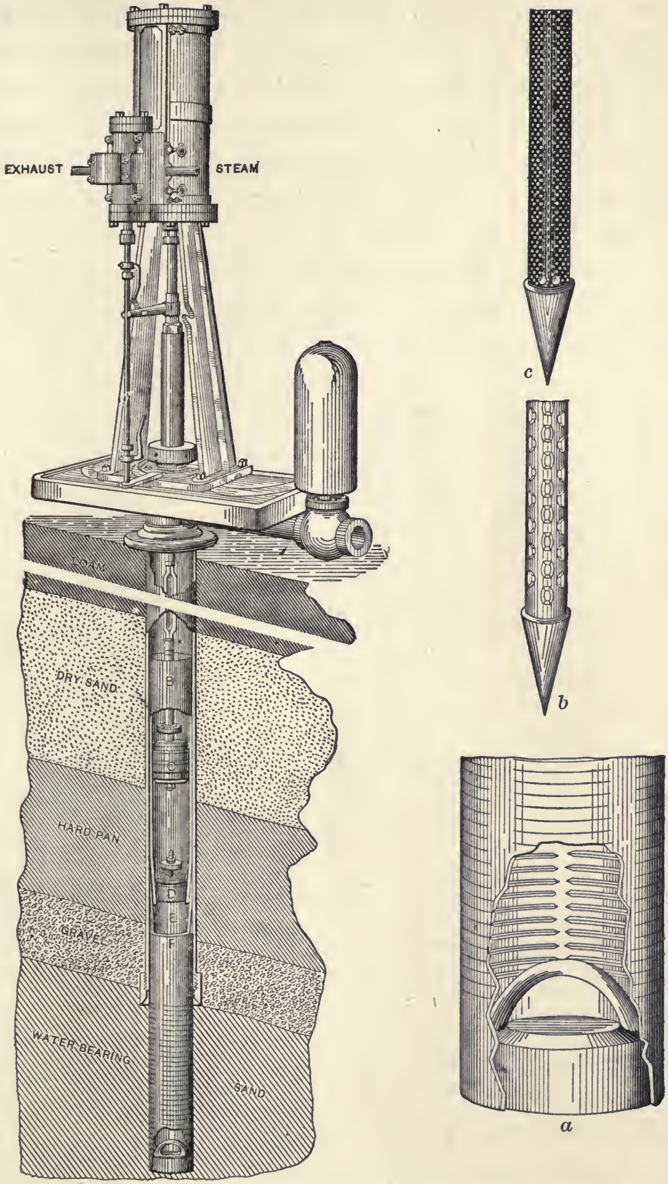

PLATE XVII.—DEEP WELL, WITH PUMP; AND STRAINERS.

right of which is an air-chamber on the pumping main for reducing the water-hammer due to the intermittent action of the single-acting pump. At *B* is the pump-barrel, which is fastened in the bottom of the well-tubing, and is provided at *D* with a valve. At *C* is a piston and valve, which is raised and lowered by the pump-rod connecting with the engine-piston.

A dug well is generally circular with brick or stone-masonry walls, as shown in section in Fig. 66; but a somewhat greater interception of ground-flow and ease of construction may be obtained with a long well of the same

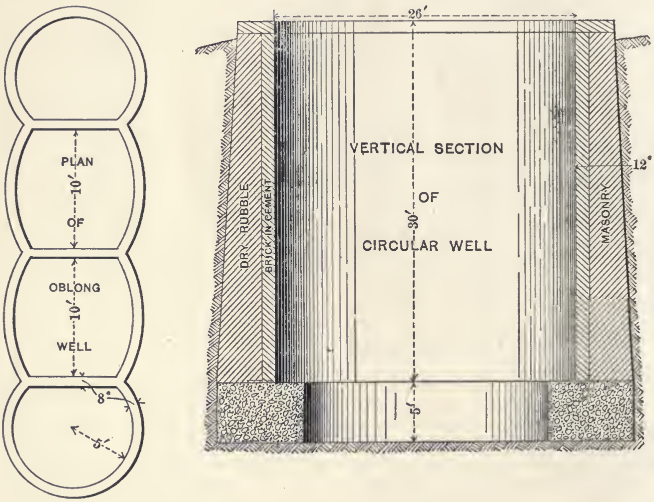

Fig. 66.—Deep Dug Wells.

storage-capacity; although more masonry will be required. Such a well, which was sunk to a depth of 32 feet, is shown in plan in Fig. 66.

Infiltration cribs and galleries are practically elongated deep wells roofed over below the surface. Timber ones are ordinarily made on the general plan shown in Fig. 67. They should be used only where the entire crib is always immersed, as otherwise they are liable to decay. A masonry gallery may be made in the form of a circular sewer, with a great number of openings left in the walls; or may have vertical walls and an arched top, the bottom being open to admit the water. Infiltration galleries and cribs generally lead directly

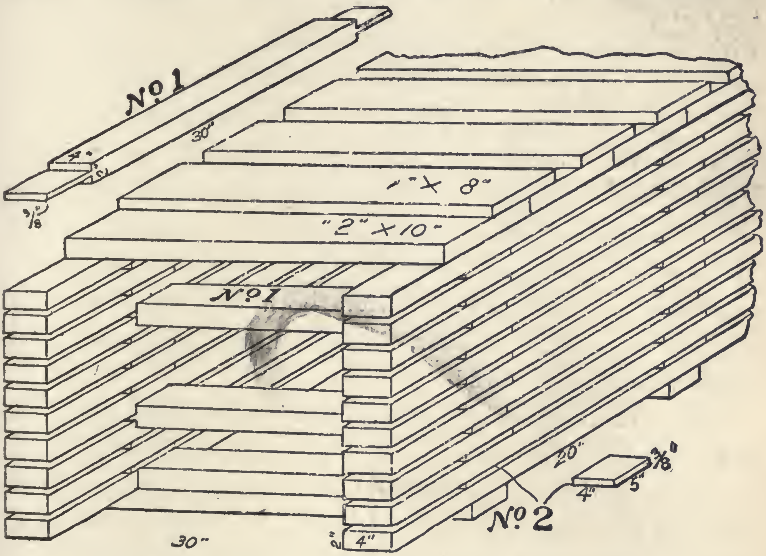

FIG. 67.—INFILTRATION-CRIB, DENVER WATER-WORKS.
(From Trans. Am. Soc. C. E., Vol. XXXI.)

to or are connected by pipe with a suction-well, from which the water is pumped. They may be extended from time to time as the demand increases, always crossing the line of ground-flow approximately at a right angle. A manhole should be provided to give access for repairs and cleaning.

ART. 93. DETAILS OF PURIFICATION PLANTS.

Mechanical filters, being patented articles, are furnished and set ready for service by the company contracting for them. The contract should stipulate the service to be performed, such as the maximum percentage or amount of mineral and organic matter and bacteria which shall pass the filter at any time, during a certain maximum rate of filtration. It is generally desirable to place mechanical filters in one room of the pumping-station, that the pump-engineer may keep them under his immediate supervision. Pressure filters are conveniently located near the pumps, being interposed in the pumping-main. Gravity filters may be placed on a level with or even a little above the main pumps, and should discharge into a suction-well. The water can in some cases be brought to the gravity filters by a flume or a short race, as from above a river-dam, but must generally be pumped. The lift will ordinarily be light, and as the rate of filtration, and consequently of pumping, should be uniform, and the water may be muddy, centrifugal pumps are well adapted to this service. For pressure filters both the pump-room and filter-room could be smaller, since no intake-pumps and fewer filters would be required.

All steam- and water-piping should be so arranged that any boiler could be used for running any pump; that any or all pumps could be put in or out of service instantly; and that any filter could be put in or out of service, or be cleaned, without interfering with the others; also a by-pass around the filters should be provided, to be used for the occasional imperative demands of the fire department. All the rooms may be upon the same level, the suction-well being, of course, lower. If sedimentation-basins are used, these may be so placed that water from the river (generally the only kind requiring sedimentation) can flow directly into them, and the pumps draw from these; or they may be at such an elevation that their effluent can flow directly into the filters, if these are the gravity type, the water being lifted from the river to them by intake-pumps, as above; or it may be necessary, in the case of gravity filters, to provide special pumps to raise water into the basins and from them into the filters. Probably the best plan, when sedimentation is necessary, is to use pressure filters, and to place the basins level with the river if possible.

A sedimentation-basin is practically but a small reservoir, and is constructed as such. When it is desired to provide a number of basins (see Art. 76) this is generally accomplished by dividing up the reservoir by a number of partition-walls. The dimensions of these are fixed by the rules for masonry dams, the vertical section being either rectangular or trapezoidal, and the top width two or three feet. It is desirable to arrange for conveniently cleaning the basins, and disposing of the sediment. For this purpose a traveller, spanning one basin and running upon tracks fixed on the partition-walls, from which buckets can be raised and lowered in any part of any basin, is often an economical contrivance when the basins are narrow. If there is little sediment, however, or if the basins are large, no fixed appliance is generally provided, the material being removed by wheelbarrows on temporary plank runways.

English filters have been formed by placing the filtering material in the bottom of old reservoirs; and new ones are commonly made similar to a reservoir 8 to 12 feet deep. The general construction of these filters has already been described in Art. 77. Particular pains should be taken to properly grade the materials from coarse to fine, and so compact them all that no considerable settlement may take place, and that the sand be not washed through into the drains. The sand must be of uniform size throughout each layer of each filter to insure a uniform rate of filtration in all parts of the same. Water should be so admitted to the filter as to prevent washing of the sand near the inlet; which requires a very low velocity in the inflowing water, such as a steady flow over a long weir. In the Albany plant this is effected by a weir in the form of a quarter-circle, shown at *a*, Fig. 1, Plate XVIII. In the same figure is shown the arrangement of drains and collectors, the right-hand half showing the manholes in the roof. The construction of the

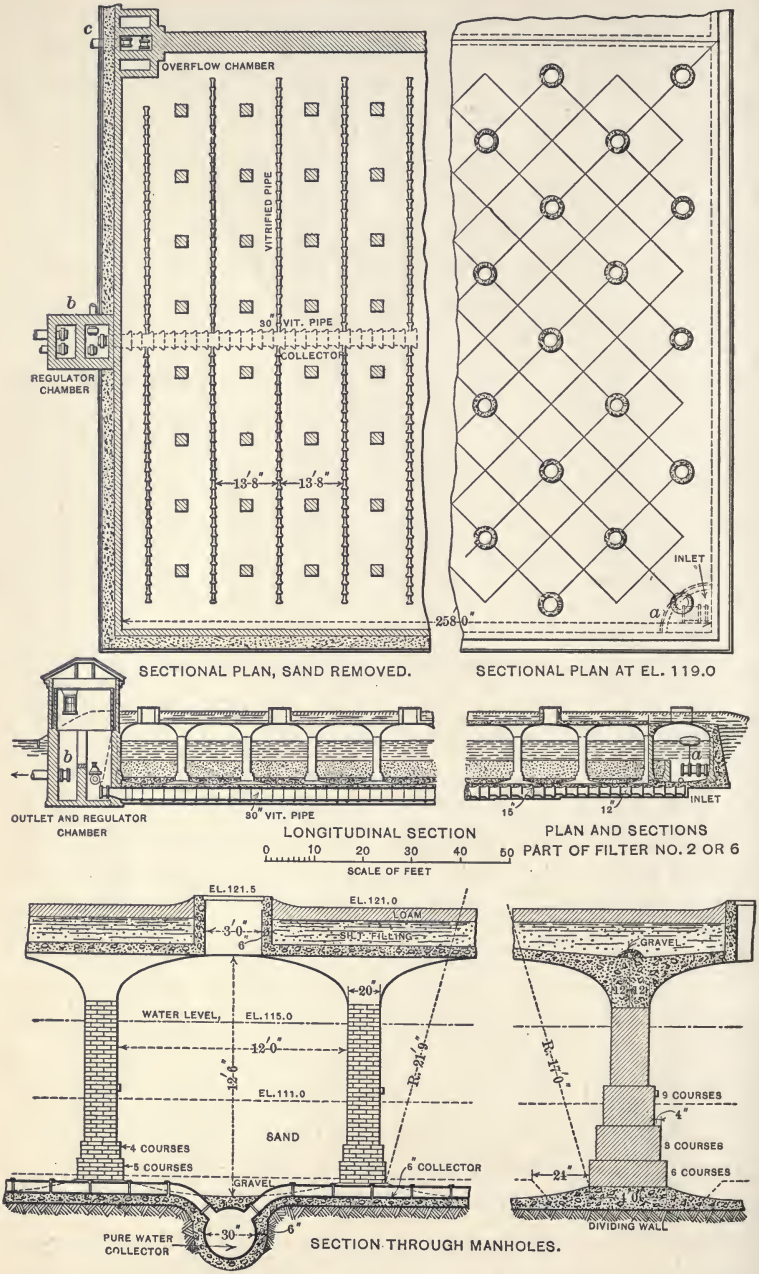

PLATE XVIII.—ALBANY FILTER-BEDS.

roof and drains is shown more plainly in Fig. 2, and the arrangement of filtering materials in Fig. 41, page 299. In Plate XIX, Fig. 1, is shown the general arrangement of the filter-beds and sedimentation-basin. Water from the river enters at *d*, and passes either to the right and into the basin through the inlets *e*, *e*, or directly to the beds through the 36-inch pipe at the left and lower side of the basin, entering the beds through the inlets *a*, *a*. Water from the sedimentation-basin passes through the outlets *f*, *f* to the 36-inch pipe just referred to, and thence to the beds.

The height of water on the beds is controlled by the float and valve at *a* (the same letters are used in both plates for corresponding parts), and is prevented from rising above a given height by overflows at *c*, *c*; at which point also the water may all be drawn off the beds, water from the overflow-chambers being wasted. The rate of filtration is controlled at the regulator-chambers *b*, *b*. Between the two sets of beds is a court in which is the laboratory and sand-washer, the sand being removed by a runway in each bed. The filter effluent flows to a pure-water reservoir.

For smaller plants the arrangements are generally more simple, but the general plan should be practically the same.

If it is desired to filter the water of a surface supply, similar filters, either slow or mechanical, may be placed between the storage- and distributing-reservoirs, gravity pressure only being used. A distributing-reservoir is necessary when a supply is filtered, since a filter should work at a uniform rate while the consumption is not uniform. It is desirable to cover this reservoir, to preserve the purity of the filtered water. The storage-reservoir will generally act as a sedimentation-basin, and a special basin for this purpose will be unnecessary.

A by-pass from river or storage-reservoir around the filter-basin should be provided, for occasional use. It may in some

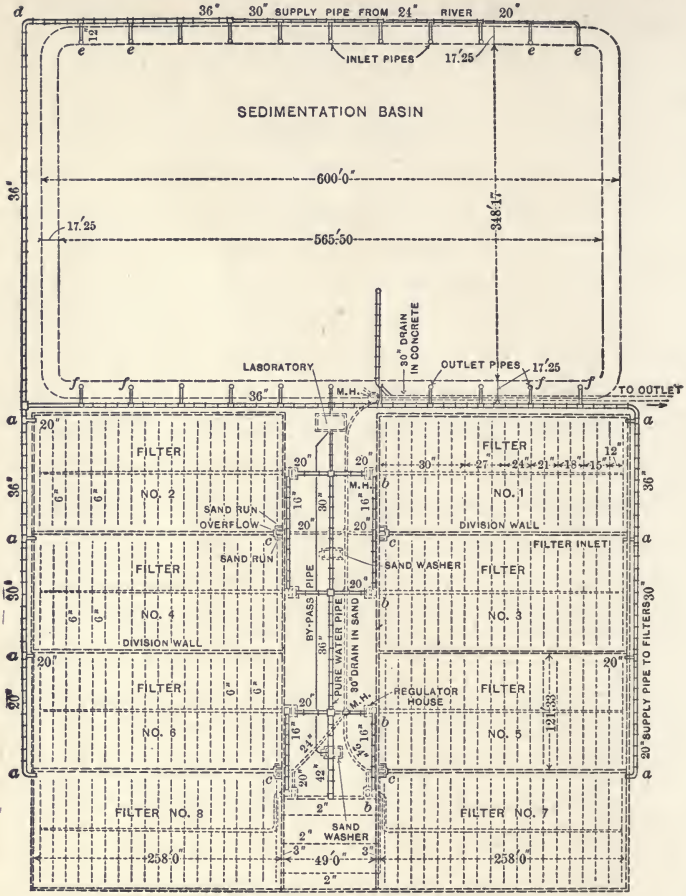

Fig. 1

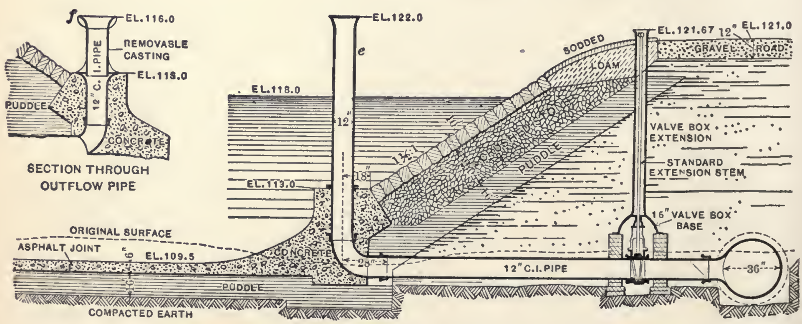

Fig. 2

PLATE XIX.—ALBANY FILTER-BEDS.

FIG. 1.—Sedimentation-basin and Pipe Systems.
FIG. 2.—Bank, and Inlet and Outlet Pipes of Sedimentation-basin.

cases be found that filtration is necessary only a part of the time, the water being used unfiltered during the remainder. This is often the case with surface-waters in which the only objectionable impurity is the occasional presence of algæ in large numbers, or clay or loam washed in by heavy rains.

Art. 94. Details of Conduits and Distribution Systems.

Canals embody the same principles of construction as do earth dams and reservoirs. The side slopes are generally $1\frac{1}{2}$ to 1, the tops of embankments 6 to 12 feet wide. The same precautions and means are taken to prevent leakage. Earth canals are seldom used for city supply; but for irrigation systems have been used both with and without lining.

On the Santa Ana Canal (California) one section is in clay, having a top width of $12\frac{1}{2}$ feet and a depth of $7\frac{1}{2}$ feet, the sides being sloped but $\frac{1}{2}$ to 1, the bottom being rounded and the whole lined with stone in cement-mortar, and given a coating of rich cement-mortar; the walls being about 6 inches thick at the top and 12 to 15 inches at the bottom. Plate VIII, page 175, shows a section of this canal.

Canals with masonry walls may be used with satisfaction where bed-rock is found at the surface. This will generally be on the side hills, where the canal will be formed on a bench. The rock will then form the bottom and usually one side, and the other side is formed by a masonry wall, the stone for which is obtained in the excavation. The wall should be at least two or three feet thick on top, and of trapezoidal section. It is better to use small "one-man" stones, laid in rich cement-mortar thoroughly filling all joints. If the work is at all seamy or porous, the canal should be lined with concrete and given a cement coating.

The most common form of open conduit is the flume, made wholly or partly of timber. The rectangular flume is illustrated in Plate X, page 179; its general construction is shown in Fig. 68. A tighter and more durable flume, known

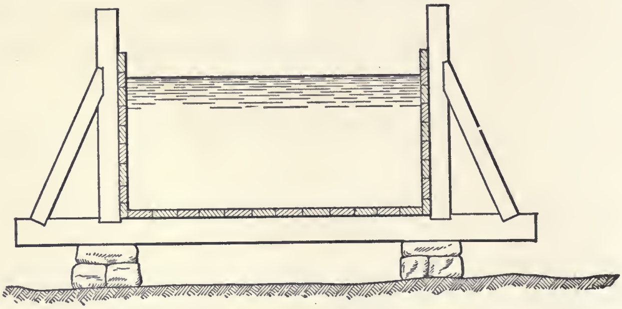

FIG 68.—BOX-FLUME CONSTRUCTION.

as the stave and binder flume, has been used recently in the West. Its general construction is shown in Fig. 69. In this the bottom is made like the lower half of a wood-stave pipe, but vertical sides take the place of a closed top; a binding-rod passes around the flume, its ends passing through the two ends of a cross-head and being provided with nuts by which the staves are forced together. This flume is supported by stiff U-shaped frames of T iron resting on wooden bolsters or sills and spaced 8 feet apart, each frame resting on concrete foot-blocks.

Flumes resting on benches should be supported wholly by the original soil, and in no place by filling. The sills should be raised above the ground a short distance and supported on solid stone, brick, or concrete foot-blocks. The excavation should be carried into the hillside three or four feet beyond the flume, to lessen the danger of this being damaged by stones or earth falling from above. The timber used should be that which is least liable to decay.

Creosoting will add considerably to the life of the timber, but is generally too expensive. If all surfaces intended to come in contact are first painted with hot asphaltum or tar, decay will be considerably delayed.

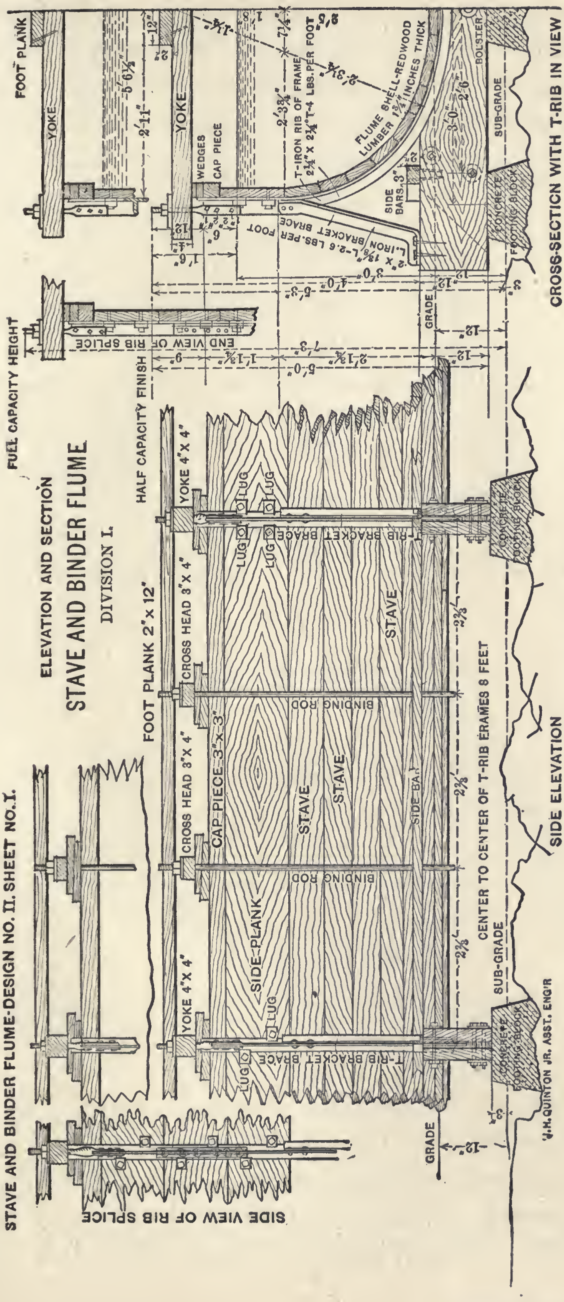

FIG. 69.—STAVE AND BINDER FLUME
(From Trans. Am. Soc. C. E., Vol. XXXIII.)

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A closed conduit may be made by covering a timber flume with plank, or may be constructed of masonry. The latter plan is generally adopted, and the conduit is covered with a few feet of earth to protect it from frost in winter and heat in summer, and from malicious damage. It is generally circular in section when in firm soil, this shape being the most economical. It may be made of brick, stone, or concrete. If of brick, two concentric rings are generally employed up to a diameter of about 5 feet; three rings up to about 10 feet. When on the surface, on a timber or other foundation, or in unstable soil, the "horseshoe" section is ordinarily employed, having an arched top, vertical side walls sufficiently heavy to receive the thrust of this, and a flat inverted arch at the bottom. Such a section is shown in Fig. 70, that of the old Croton Aqueduct. Vertical side walls of an aqueduct are generally made of stone masonry, lined with brick or concrete.

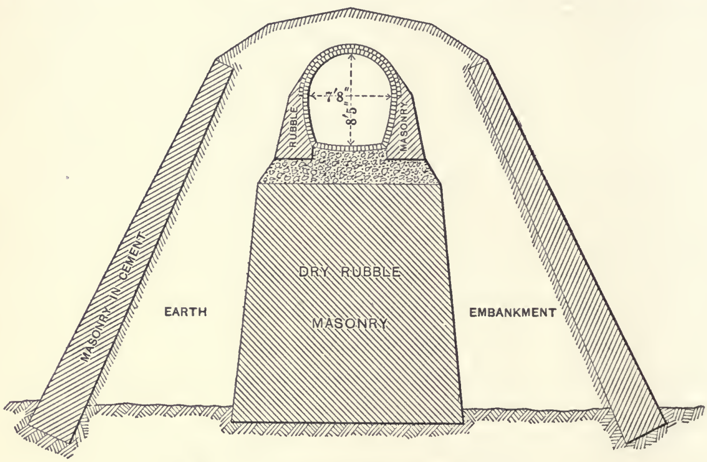

FIG. 70.—OLD CROTON AQUEDUCT.

The foundation of a masonry conduit or aqueduct must be absolutely rigid, since the least settlement and crack in the aqueduct may lead to serious results. To prevent softening of the supporting soil, as well as waste, the masonry should be water-tight. The Wachusett, Mass., Aqueduct where crossing a valley on a masonry bridge is rendered water-tight, in spite of any slight settlement of the bridge arches, by lining the channel with sheet lead weighing 5 lbs. to the square foot, and protecting this from wear by an inside layer of brick.

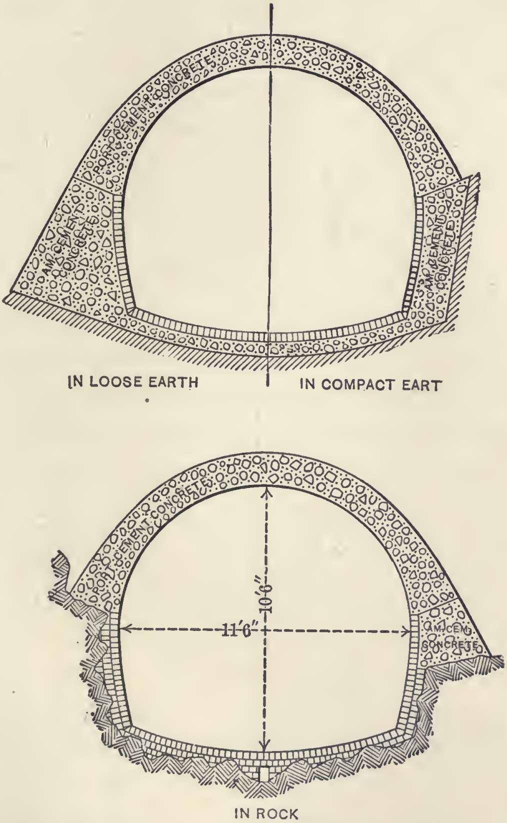

FIG. 71.—SECTIONS OF CONCRETE AQUEDUCT.
(Nashua Aqueduct, Metropolitan Water-supply.)

Conduits under pressure cannot be built of masonry; except that concrete has been so used when having imbedded in it "expanded metal," heavy screens, or iron rods both longitudinal and encircling the aqueduct, to sustain the tension. The majority of pressure conduits, however, are made of cast or wrought iron or steel, or of wood, either as bored logs or stave-pipe. In the metal pipes the tension is resisted by the cohesion of the material itself. Bored logs exert this to a certain extent; but if to sustain much pressure, they are tightly wrapped with wire or iron bands. Stave-pipes rely entirely upon iron bands or hoops for their resistance to internal pressure; except that between these the stiffness of the staves prevents springing and leaking.

The pressure to be resisted by a pipe—that is, the resultant component of all forces in any one direction in its cross-section—equals $WhdL$, in which W is the weight of a cubic foot of water, d is the diameter of the pipe in feet, h is the head upon the centre of the pipe, and L is the length of pipe considered. If we make L one inch, and express d in inches, making W the pressure upon a square inch due to one foot head, the internal pressure $P = .434hd$ lbs. per lineal inch. This is resisted equally (in a circular pipe) by both sides, and hence the tension per square inch of metal $T = \frac{.434hd}{2t}$, in which t is the thickness of the shell in inches. h must be taken as the maximum head possible, including that due to water-hammer; and a sufficient factor of safety must be used, which will vary with the substance employed. A minimum thickness must be adopted, below which the pipe would be subject to distortion or breakage by handling. If we allow 200 lbs. for water-ram (see Fig. 23), and call s the ultimate tensile strength of the substance used and F the factor of safety, the formula for thickness will be $t = \frac{(.434h + 200)dF}{2s}$. This formula does not apply to pipe where the tension is sustained by bands. In these $P = .434hd$ lbs. per lineal inch, and the tension in each rod is $\frac{.434hdb}{2}$, in which b is the distance between bands in inches. This tensile strain is of course acting in every part of the pipe or band.

The minimum limit for cast-iron pipe may be taken to be:

for 4-inch diameter...... $\frac{3}{8}$ in.	for 24-inch diameter..... $\frac{11}{16}$ in.
for 8-inch diameter...... $\frac{7}{16}$ in.	for 30-inch diameter..... $\frac{3}{4}$ in.
for 10-inch diameter..... $\frac{1}{2}$ in.	for 42-inch diameter..... $\frac{13}{16}$ in.
for 16-inch diameter..... $\frac{1}{2}$ in.	for 48-inch diameter..... $\frac{7}{8}$ in.
for 20-inch diameter..... $\frac{5}{8}$ in.	for 60-inch diameter..... $1\frac{1}{4}$ in.

The tensile strength of ordinarily good cast iron may be generally taken at 18,000 lbs. per square inch. Many formulas have been advanced for determining the thickness of pipe, allowing for minimum thickness and tensile strength. That of James B. Francis is

$$t = .000058hd + .0152d + .312.$$

$$t = .00006hd + .0155d + .296$$

is used by the Warren Foundry and Machine Company. Cast-iron water-pipes are ordinarily connected by hub-and-spigot joints, filled with melted lead which is "set up" with calking-tools. For indoor work, joints which it is desired to unmake occasionally, and some other conditions, flanged pipe is used, but is not desirable for underground work because of its inflexibility, and is more expensive than the hub-and-spigot joint. The flanges of flanged joints are faced down to a true plane at exact right angles to the axis of the pipe; and

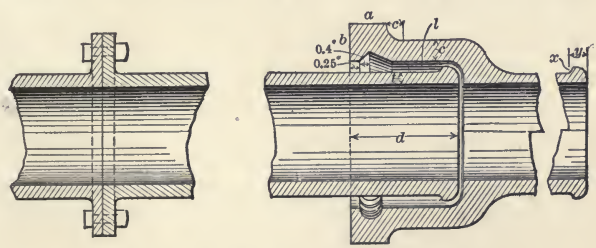

FIG. 72.—JOINTS OF CAST-IRON PIPE.

in joining them a ring of packing—sheets of rubber and cotton, corrugated copper, lead wire, and other materials are used, the first being the most common—is placed between the flanges, which are drawn tightly together by bolts.

There is no standard design of bell and spigot. The spigot is formed by casting a "bead" around the pipe at its end. The bell-opening is made about $\frac{5}{8}$ to 1 inch larger than the outside of the spigot, and is furnished with an annular recess about $\frac{1}{2}$ inch back from the face of the bell. This recess may be either circular or V-shaped, no particular advantage having been found in either form over the other. Lead is poured into all the bell-space not filled with packing and throughly calked. The dimensions of bell and thickness and weight of different sizes of pipe as used on the Boston water-works is shown in the following table; the letters referring to Fig. 72.

TABLE NO. 69.

WEIGHTS AND DIMENSIONS OF CAST-IRON WATER-PIPES.

Diameter in Inches.	Class.	Dimensions in Inches.								Weight.
		a	*b*	*c*	*d*	*t*	*l*	*x*	*y*	
3	B	1.50	1.20	0.60	4.0	0.40	0.40	0.20	0.60	180
4	B	1.50	1.30	.65	4.0	.45	.40	.20	.60	260
6	B	1.50	1.40	.70	4.0	.50	.40	.20	.60	420
8	B	1.50	1.50	.75	4.0	.55	.40	.20	.60	600
10	B	1.50	1.60	.80	4.5	.60	.40	.20	.80	815
12	A	1.50	1.60	.80	4.5	.58	.40	.20	.80	935
12	B	1.50	1.70	.85	4.5	.65	.40	.20	.80	1,050
16	A	1.75	1.70	.85	5.0	.66	.50	.25	.85	1,415
16	B	1.75	1.90	.95	5.0	.75	.50	.25	.85	1,615
20	A	1.75	1.90	.93	5.0	.73	.50	.25	.85	1,950
20	B	1.75	1.90	.95	5.0	.85	.50	.25	.85	2,250
24	A	2.00	2.10	1.05	5.0	.81	.50	.25	.85	2,590
24	B	2.00	2.10	1.05	5.0	.94	.50	.25	.85	2,985
30	A	2.00	2.30	1.15	5.0	.93	.50	.25	.85	3,690
30	B	2.00	2.30	1.15	5.0	1.10	.50	.25	.85	4,335
36	A	2.00	2.50	1.25	5.0	1.04	.50	.25	.85	4,930
36	B	2.00	2.50	1.25	5.0	1.25	.50	.25	.85	5,880
40	A	2.00	2.70	1.35	5.0	1.12	.50	.25	.85	5.900
40	B	2.00	2.70	1.35	5.0	1.35	.50	.25	.85	7,050
48	......	2.00	2.70	1.35	4.0	1.00	.50	.25	.85	6,275
48	A	2.00	3.00	1.50	5.5	1.25	.50	.25	.85	7,920
60	......	2.25	3.40	1.70	6.0	1.375	.50	.25	.85	10,960

Cast-iron water-pipes are made to lay 12 feet of conduit each. Any branch pipe or special constructions are connected with it by means of special castings, or "specials"; and valve-gates are made with bells or flanges similar to those on the pipe, and are interposed in the pipe-line where desired. The specials ordinarily used and kept as stock patterns are shown in Fig. 73, and their dimensions are given in Table No. 70. The specials here shown have a spigot at one end and hubs at the others; but they are sometimes made with hubs at all ends. The metal used in cast-iron pipes should be a superior quality of gray iron, remelted in the cupola or air-furnace, tough and of even grain, and should possess a tensile strength of not less than 18,000 lbs. per square inch. Test-bars of the metal 3 inches by $\frac{1}{2}$ inch, when placed upon supports 18 inches apart and loaded in the centre, should have a transverse breaking-load of not less than 1000 lbs., and should have a total deflection of not less than $\frac{3}{8}$ inch before breaking. These test-bars should be poured from the ladle at any time the engineer directs, before or after the castings have been or while they are being poured. The thickness of the shell of the pipe should be uniform throughout, but it seems almost impossible to obtain exact compliance with this. When one side of the pipe is somewhat thinner than the other, unequal cooling causes internal strains, and because of these and the thinner metal it is desirable to double the factor of safety which would be used were the shell of uniform thickness throughout and the weight of the entire pipe still that actually obtained. A factor of 5 to 10 is generally employed.

Cast-iron pipe will rust and, with some classes of waters passing through it, become covered with tubercles in a few years unless protected with a coating. Several substances have been tried for this, but that which has generally proved most satisfactory and is now used by many if not all foundries is Angus Smith's coating, which was first used in Manchester, England, in 1849, and in this country in the Brooklyn water-works in 1858. In applying this coating the pipes are immersed in a hot bath of boiling coal-pitch from which the naphtha compounds have been distilled off and a small

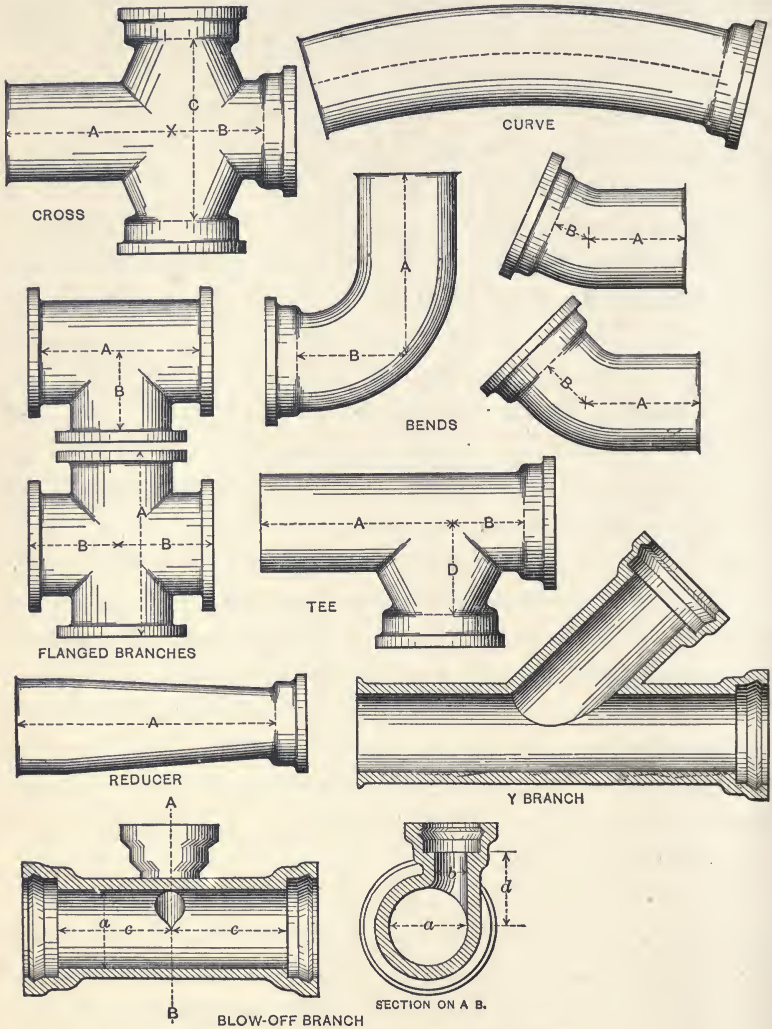

FIG. 73.—CAST-IRON PIPE SPECIALS.

amount of mineral oil added to give fluidity. The pipe should remain in this hot compound until itself heated, when it will have received a tough adhesive coat of pitch. No cast-iron pipe should ever be laid without being first coated with this or an equally good compound.

TABLE No. 70.

DIMENSIONS OF PIPE SPECIALS, IN INCHES.

Diameter.	Crosses.			Tees.			Bends.					
							90°		45°		22½°	
	A	*B*	*C*	*A*	*B*	*D*	*A*	*B*	*A*	*B*	*A*	*B*
4	18	8	15	18	8	7½	18	8	11	4	9	3
6	20	9	17	20	9	8½	20	10	13	5	11	4
8	22	10	20	22	10	10	22	12	14	6	12	4
10	24	11	23	24	11	11½	24	14	15	7	13	5
12	26	12	26	26	12	13	26	16	16	8	14	5
14	28	13	29	28	13	14½	28	18	17	9	15	6
16	30	14	32	30	14	16	30	20	18	10	16	6
18	32	15	35	32	15	17½	32	22	16	11	17	7
20	34	16	38	34	16	19	34	24	20	12	18	7
24	36	20	44	36	20	22	38	28	22	14	20	8

Diameter.	Flanged Branches. Crosses, and Tees.		Blow-off Branches.				Reducers.	
	A	*B*	*a*	*b*	*c*	*d*	Diameters.	*A*
4	15	7½	8	4	10	7	5 × 4 to 6 × 3	29
6	18	9	10	4	10	8	8 × 6 to 8 × 3	32
8	21	10½	12	4	10	10	10 × 8 to 10 × 5	35
10	24	12	16	4	10	12	12 × 10 to 12 × 6	38
12	27	13½	20	6	12	14	14 × 12 to 14 × 6	41
14	30	15	24	6	12	16	16 × 14 to 16 × 8	44
16	33	16½	30	12	13	20.5	18 × 16 to 18 × 8	47
18	36	18	36	12	13	23.5	20 × 18 to 20 × 8	50
20	39	19½	40	12	13	25.5	24 × 20 to 24 × 10	56
24	45	22½	48	16	24	29.5		

Cast-iron pipe is now used almost universally in this country for distribution systems, and for small reservoir-conduits also. House-connections are made with it at any point by tapping a hole in the pipe and inserting a corporation cock. Larger connections, such as branch lines, fire-hydrants, blow-offs, etc., are made by means of special castings.

Wrought iron or steel in thin plates is used in several ways for water-pipe. A small pipe may be made of a single plate whose edges are riveted together or are lap-welded; or by winding a long narrow plate spirally and riveting or welding the spiral lap. Larger pipe is riveted, with lap- or butt-joints, and made in sections 15 to 30 feet long, each formed of two or more plates; either every second section of lap-joint pipe being made smaller than the others and fitting tightly into their ends, or each end is slightly tapering, one end forming the inside, the other the outside, of a lap-joint. Another method of uniting the sheets recently used in

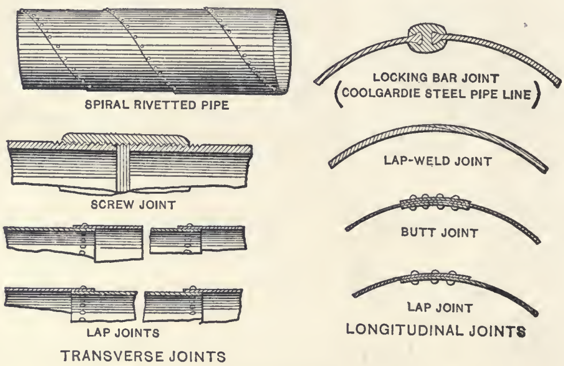

FIG. 74.—WROUGHT-IRON PIPES.

Australia is by the locking-bar; a double-grooved bar receiving the edges of the sheet and binding them tightly together under hydraulic pressure. For small pipe the lap-weld is most used; and for large, the riveted lap-joint. The pipe is generally made in 15- to 30-foot sections in the shop, and these riveted together in the field. For convenience of joining, small wrought-iron pipe is frequently furnished with a cast-iron bell at one end and a narrow strap for a bead at the other, the pipe being joined with lead as is cast iron. Most small lap-weld pipe, however, is joined by screw couplings. An expansion-joint for connecting wrought-iron pipe has been made by giving each end an outward-flaring, bell-shaped flange, the advantages claimed for it being that the expansion and contraction of the pipe does not strain the riveted joints, and that the field-riveting of joints can be done entirely from the outside with a hydraulic riveter.

Wrought-iron or steel pipe is made much thinner than cast iron, both because the tensile strength of the metal is greater and because it can be made more uniform in character and thickness of metal and can be carefully inspected in the sheet. Also much lighter pipe can be handled without danger of breaking. The thickness is determined as for cast iron, but a lower factor of safety is used. Because of the thinness of the metal this pipe rusts through much more quickly than cast iron, and hence a protective coating is even more necessary. Large wrought-iron or steel pipe is distorted by the weight of the back-filling, unless the earth against the bottom and sides be compactly rammed, and the pressure on the top be evenly distributed, as by sand or gravel. No large stones or lumps of clay should be placed immediately on top of this pipe in back-filling.

Changes of direction in riveted pipe are made either by use of short sections with converging or bevelled end-planes, or by means of iron castings. Branches, reducers, and all other special constructions are made of riveted plates in large pipes; and of riveted plates, but more often of iron castings, in small pipes; special designs being generally made for each case in the riveted work, but the castings used being standard, as for cast-iron pipe.

The metal should be tough and elastic, capable of shop- and field-riveting without any injury. The Rochester 38-inch main, laid in 1894, called for soft, open-hearth steel containing not over 0.6% of manganese and 0.06% of phosphorus, and having a tensile strength of between 55,000 and 65,000 lbs., an elastic limit of not less than 30,000 lbs., and an elongation of 22½% in 8 inches. Cold-bending, punching, and similar tests also were called for; and no plate was to be less than 95% of the required thickness at any point. The thicknesses varied between $\frac{1}{4}$ and $\frac{5}{8}$ inch. Steel from 0.1 inch to $1\frac{1}{4}$ inches, or perhaps even thicker, has been used for riveted pipe. The dimensions of some conduits of this pipe were given in Table No. 45, page 186.

For coating of riveted pipe Angus Smith's pitch was found to be unsatisfactory, and other materials are being tried, most of them having asphaltum as a principal ingredient. The Rochester, N. Y., Bundaleer, Australia, and other conduits have used a mixture of equal parts of refined Trinidad asphaltum and refined coal-tar. A japanning process of applying asphalt is used also; asphalt dissolved in bisulphide of carbon ("P. & B.") and many other compounds have been tried. The use of riveted pipe has been too recent to furnish a thorough test of these coatings. The specifications for coating the Bundaleer pipe (1898–9) were as follows:

"30. Immediately after testing, each pipe shall be thoroughly and effectually freed from rust and dirt to the satisfaction of the inspecting officer.

"31. The coating composition shall consist of equal proportions of best Trinidad natural asphaltum and refined coal-tar, or such modification of the proportions as the Engineer-in-Chief may consider advisable. The tar shall be kept for about four hours at a temperature of 210° Fahr. to permit of evaporation before being used. After these materials have been thoroughly mixed together in a proper mixing-bath, they shall be run into a horizontal dipping-bath and maintained at a temperature of not less than 350°, nor more than 400° Fahr. during the process of dipping.

"As the mixture will deteriorate after a number of pipes have been dipped, it shall be cleaned and fresh materials, in correct proportions, added when ordered by the superintending officer. The bath must be occasionally emptied and replenished with new material.

"32. No pipe shall be dipped until it has passed the hydrostatic test, nor till it has been approved by the superintending officer immediately before dipping. Every pipe shall be perfectly clean, dry, and free throughout from rust when the coating is being applied. Immediately before being placed in the bath each pipe shall be dried by a blast of hot air." It was also provided that each pipe should be dipped twice, remaining in the bath for 15 minutes the first time, and should then be sprinkled with clean sand; the entire coating of asphalt and sand to weigh "not less than 9 oz. per square foot of both sides taken together."

Wrought-iron pipe weighs about 10.1 lbs. per lineal foot for each foot of circumference and each quarter-inch of thickness (the width of laps being added to the pipe circumference and length); and steel about 10.2 lbs. for the same unit dimensions. Or, weight of steel pipe

$$W = 40.8t(3.1416d + l)(L + l),$$

in which t is the thickness of metal in inches, d is the diameter of pipe in feet, L is the length of the pipe, and l is the lap at either longitudinal or circular joints.

"Steel seems best adapted for use (as compared with cast iron). First. For the larger sizes of mains, say from 24-inch diameter upwards. At about this size the relative cost begins to be in favor of cast iron. Second. For what may be called leading mains in outlying districts—force-mains, and similar work where not likely to be disturbed or tapped. Third. As conduits in remote regions difficult of access, its lightness and ease of transportation fitting it especially for this use." (To which may be added mains subject to excessive pressure. Thick cast-iron shells are subject to unknown cooling-stresses and air-holes and other flaws.) "Under these conditions the principle advantages in the use of steel are: First. Economy in first cost. This is in many instances the controlling factor in the case. . . . Second. Less liability of breakage while in service. Steel plate is undoubtedly a much more reliable material than cast iron to resist tensile strains and the shocks incident to its use in water-mains. . . . Third. Its adaptability for special situations when the use of cast iron would be attended with risk or excessive cost." (L. M. Hastings in Journal of N. E. W. W. Ass'n, June 1899.) The flexibility of this pipe and use of riveted joints seem to adapt it especially for use in bridge-crc sings and other locations subject to vibration or slight movements which tend to open lead joints.

Wrought-iron pipe coated inside and out with cement has been used, but its use has been practically abandoned. By its use the tensile strength and uniformity of sheet metal is obtained, the cement coating protects it from rust and gives it stiffness, and does not impart a taste to the water as does the tar coating used on cast-iron pipes. But cast-iron pipe many times as strong as the wrought iron now costs 15 to 25 per cent less laid in the line, is less liable to damage from handling, is more easily laid, and service connections can be made with it more readily. In many cases the cement-lined pipe has been entirely destroyed by minute cracks in the cement, which would permit rust to form in the iron and, if on the outside, roots to enter and peel off the cement from the outside of the pipe. However, if the cement lining is put on properly and good materials used, this pipe has some advantages over cast iron, particularly where there is much trouble from tuberculation.

Wood pipe offers certain advantages over iron, particularly where the cost of the latter material or of transportation is excessive. It is also free from injury by electrolysis. If the pressure is at all high, however, the pipe must depend upon

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iron or steel bands for strength; and the pressure must be less than that which will cause the bands to crush the wood fibres. The life of the wood, if it be always saturated, should be indefinite, and that of the pipe therefore depends upon the bands.

Small water-pipe of bored logs is now but little used. It is claimed, however, that 1500 miles of Wyckoff water-pipe is in use, chiefly in the Middle and Western States. This pipe is made of wooden shells, bored and turned from solid logs and banded spirally with iron, steel, or bronze bands; the outside being thoroughly coated with pitch and rolled in sawdust. It is claimed that this pipe can be made to stand 150 to 200 lbs. pressure with safety. For branches and other connections cast-iron specials are used, the end of the pipe being driven tightly into the bell of the casting. Service connections are made with an auger and a corporation cock with lag-screw threads. The cost is said to be about 25% less than that of cast iron.

Wood-stave pipe has been used for many years for large conduits, such as flumes for water-wheels in New England and for hydraulic mining in the West. But during the past few years it has come into extensive use for water-supply conduits. It may be used for pressures up to 85 lbs. when made of redwood or Douglas fir; the limit depending upon the ability of the wood to resist crushing. The life of the wood should be considerable, although where exposed to the air in a dry climate evaporation will keep the outside nearly dry. The steel bands should be coated with protective paint, and if possible should be repainted whenever necessary. But when the pipe is buried the bands cannot be repainted. To enable them to resist rust as long as possible they should be of round rather than of flat metal; when their life should be at least fifty years in ordinary soils.

Experiments would seem to indicate that if a wood-stave pipe is exposed to a dry atmosphere a loss from percolation and evaporation of 0.05 gals. per day per square foot of surface may be expected; but if it be buried, no loss from this cause is found to occur.

This pipe is made of a number of "staves of variable length, having radial edges and concentric faces, and which are held together by metal bands usually circular in section and spread in accordance with the demands of the strains imposed.

"In the design of a wood-stave pipe, the following essential points require consideration: The staves must be thin enough to secure complete saturation and to deflect readily to the degree of curvature employed, and they must be thick enough to prevent undesirable percolation through them. The bands must be of such size that, when spaced to secure the desired factor of safety against rupture, there will at the same time be no sensible flexure in the staves and no destructive crushing of the fibre beneath the bands. While fulfilling these conditions, the proportion between the thickness of the staves and the strength and spacing of the bands must be such that the swelling of the wood will not produce injurious strains upon what might otherwise be a properly proportioned band. . . . The tensile strains resisted by the bands are from three sources: (1) The initial strain caused by crushing during construction. (2) The pressure of the water within the pipe. (3) The swelling of the staves. The strains resisted by the staves are from the following sources: (1) The compressive strain of the bands. (2) The compressive strain upon the edges of adjoining staves. (3) The pressure of the water producing flexure of the staves between adjacent bands." (A. L. Adams, Trans. Am. Soc. C. E., vol. XLI. page 27, which see for full discussion of wood-stave pipe.) Adams considers Table No. 71 page 424 to give the most economic proportions for staves and bands. The spacing of the latter of course varies with the pressure. His formula for this is as follows:

$$f = \frac{s}{(R + \frac{3}{2}t)P + E''t}, \quad \text{when} \quad P < E'' \quad \text{and} \quad s < (R + t)e$$

$$= \frac{(R + t)e}{(R + \frac{3}{2}t)P + E''t}, \quad \text{when} \quad P < E'' \quad \text{and} \quad s > (R + t)e$$

$$= \frac{s}{(R + \frac{3}{2}t)P}, \quad \text{when} \quad P > E'' \quad \text{and} \quad s < (R + t)e$$

$$= \frac{(R + t)e}{(R + \frac{3}{2}t)P}, \quad \text{when} \quad P > E'' \quad \text{and} \quad s > (R + t)e;$$

in which f is the band-spacing in inches; s is the safe tensile strain in the band in pounds; R is the internal radius of the pipe in inches; t is the thickness of the stave in inches; P is the water-pressure in pounds per square inch; e is the safe bearing-power of the wood in pounds per lineal inch of band (about 800 lbs. per square inch of contact surface of band and wood); and E'' is the permanent swelling force of the wood in pounds per square inch (assumed at 100 lbs. for all spacing).

The staves are made to break joints in the pipe. The butt-joints are made by driving into kerfs sawed in the end of each stave thin steel plates about $1\frac{1}{2}$ inches long and somewhat wider than the stave. The bands are cinched by fastening the two ends of each band in a cast-iron shoe, one end being provided with a thread and nut. The bottom of the shoe is made to fit the outside of the pipe. Curves are made in the pipe by simply bending the staves during construction, which is carried on in the field, the pipe being built in its final position. Small branches, air-escapes, etc., are attached by fastening castings on the outside of the pipe by means of bands.

Wood-stave pipe should not be coated on the inside, as this would prevent the saturation of the wood and permit decay. No coating has been found which will remain on the outside of a pipe when it is under pressure; and if buried, such a coating would probably be of no great value.

TABLE No. 71.

ECONOMIC PROPORTIONS FOR PIPE DESIGNS.

Nominal Diameter.	Stock Sizes for Staves.	Thickness of Finished Staves.	Economic Size of Bands.	S	Factor of Safety in Band.	e $\left(\text{max. value} = \frac{650}{\text{rad. of band sec.}}\right)$.
10	$1\frac{1}{2} \times 4$ in.	$1\frac{1}{16}$ in.	$\frac{5}{16} \times \frac{7}{16}$	1255	5.26	207
12	$1\frac{1}{2} \times 4$	$1\frac{1}{8}$	do.	1475	4.47	207
14	$1\frac{1}{2} \times 4$	$1\frac{3}{16}$	do.	1650	4	
16	2×6	$1\frac{7}{32}$	do.	1650	4	
18	2×6	$1\frac{3}{8}$	do.	1650	4	
20	2×6	$1\frac{3}{8}$	do.	1650	4	
22	2×6	$1\frac{3}{8}$	$\frac{3}{8}$	1508	4.4	122
24	2×6	$1\frac{3}{8}$	$\frac{3}{8}$	1650	4	
27	2×6	$1\frac{7}{16}$	$\frac{3}{8}$	1650	4	
30	2×6	$1\frac{1}{2}$	$\frac{1}{2}$	2673	4.4	162
36	2×6	$1\frac{9}{16}$	$\frac{1}{2}$	2950	4	
42	2×6	$1\frac{5}{8}$	$\frac{1}{2}$	2950	4	
48	2×6	$1\frac{11}{16}$	$\frac{1}{2}$	2950	4	
54	$2\frac{1}{2} \times 8$	$2\frac{1}{8}$	$\frac{5}{8}$	4600	4	
60	3×8	$2\frac{1}{2}$	$\frac{5}{8}$	4600	4	
66	3×8	$2\frac{9}{16}$	$\frac{3}{4}$	6600	4	
72	3×8	$2\frac{5}{8}$	$\frac{3}{4}$	6600	4	

On all pressure conduits, blow-offs should be provided at low points for use in flushing all dirt out of the pipe. In irrigation conduits or others which sand is likely to enter, sand-boxes and gates should be provided. The sand-box is essentially an enlargement and depression in the canal, in which the reduced velocity causes the heavier sediment to settle. This sediment may conveniently be removed through an opening in the bottom of the sand-box, or a sluice-gate in its side at the bottom, through which, when opened, the water will wash out the accumulated deposit.

At all summits on closed conduits there should be an air-escape. If these summits are below the hydraulic gradient this escape must be so contrived as to permit air to escape, but no water. The ordinary form is shown in Fig. 75;

b being a float which remains up and keeps the opening *d* closed while the box is full of water, but falls and opens *d* as air rises into the box and lowers the water. If a fire-hydrant or service connection be placed at a summit, the air can

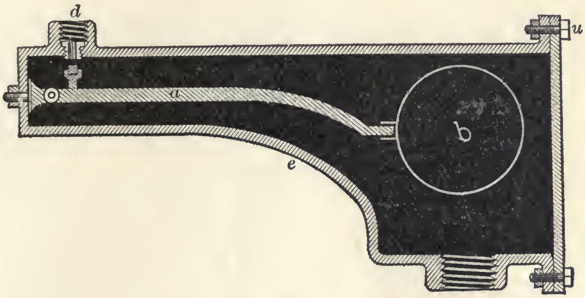

FIG. 75.—AIR-ESCAPE VALVE.

escape through this and the use of an air-escape valve is unnecessary.

The most desirable line from a reservoir or intake to the point of utilization is a straight one, if the country be level and such a location be not expensive in right of way. But detours in a hilly country may make as short a line, or one requiring less heavy pipe, and less lift if pumping be employed; and such a detour may be necessary to avoid going above the hydraulic gradient. It is largely a question of expense, and the choice between routes should be made by comparing their cost, taking account of length of conduit, thickness of pipe necessary, accessibility and cost of right of way, special river-crossings, trestles, etc., required, and increased pumping expense or size of pipe due to lengthened line and consequent flattened gradient. A gravity conduit can be carried over a summit above the hydraulic gradient by use of a siphon. But this needs special attention to remove the accumulated air frequently, and is not advisable unless within easy reach of the pumping-station or a maintenance force. The air may be removed by a small vacuum-pump; or it may be collected in an air-tight tank connected with the summit of the pipe, the air in the tank, as often as it nearly fills this, being replaced with water by closing a valve at the bottom of each leg of the siphon and pouring water into the tank.

In warm climates closed conduits may be placed upon or near the surface, although this would render the water too warm for city supplies. But in cold climates this would lead to the bursting of the pipes by freezing; and they must be protected from this, generally by burying in the ground. Open conduits also in such climates should have thick walls, or earth banked against them, to prevent contraction of the channel by ice forming on the sides. Pipes through which water is always circulating are frequently left uncovered where crossing bridges, without freezing; and as a general rule it may be said that this is permissible when the length of exposed pipe in feet is not greater than ten times the square of the diameter in inches, and the velocity of flow is at least one fourth of a foot per second. The danger that the flow will entirely stop at night-time is so great, however, that it is better to box in such pipe, and fill the box with mineral-wool, asbestos or some other non-conductor.

Water-pipes in New England, northern New York, and the entire northern tier of states should generally be placed not less than 5 feet below the surface; below these, to the southern tier, 4½ to 4 feet is the customary depth; and in the Southern States 3 or even 2 feet is permitted for small pipe, although when so near the surface this is in danger of being crushed by heavy teams. A gravity-closed conduit, if on the surface, should be covered to at least the above depth on top and sides. The above measurements are to the centre of the pipe.

A pipe conduit should rest upon a firm foundation. If in rock, the bottom of the trench should be covered with gravel, sand, or loam well rammed for the pipe to be bedded on. The pipe should slope continuously to the main depressions, where blow-offs are provided discharging into a near-by creek or sewer for removing any sediment; sags in the line favoring accumulations of sediment at the low, and of air at the high, points. The blow-off may be an ordinary T, a gate-valve, and a short piece of pipe leading to the sewer or creek; but it is better to substitute for the ordinary T the blow-off T shown in Fig. 73.

Curves of long radius may be made of chords connected by eighth-bends, or by making a slight angle at each joint. A 4-inch pipe may be swung 2 to 2½ feet out of a straight line, an 18-inch pipe 10 or 12 inches, and a good joint still be made. But more than this should not be attempted, as the lead joint cannot take a strong shape or be well calked if there be too much angle.

In crossing streams the water-pipe may be laid in the bed of the stream or carried over on a bridge. The former plan is often expensive, and the detection and repair of leaks is difficult. There is also danger in some cases that freshets will carry away the pipe. On the other hand, on an iron bridge there are vibrations tending to cause leaks, and freezing is possible; also the bridge must have additional strength to support the pipe and its contained water. Over a stone arch the pipe can be laid underground, as in a street, and the only danger is that of the freezing of the pipe where the covering both above and below it is thin, as it may be over the centre of the arch. Where there is no bridge the pipe must generally be placed in the stream-bed; but where there is one it is probably better to carry the pipe on this, either resting on the floor or suspended beneath it, and protected from freezing. If the pipe rises to the bridge-floor the joints at the elbow are apt to pull apart, and should be tied together by long bolts and clamp-rings; or steel pipe may be substituted for cast iron across the bridge and for two or three feet beyond the elbows at each rising-end.

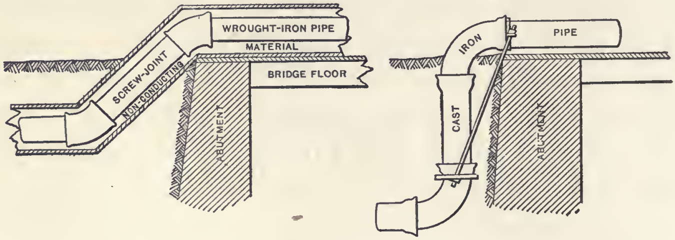

FIG. 76.—BRIDGE-CROSSINGS.

Ordinarily the pipe is simply placed in the trench, the joints made, and the trench refilled with such tamping as is required by the nature of the ground or street-surface. But where the pipe-line passes under a railroad, a building, or other structure causing considerable pressure upon the soil, it is desirable to protect the pipe from this, and to protect the structure from being undermined by a break in the pipe. In the case of a small pipe the latter may be effected by enclosing this pipe in a larger one. But a much better plan is to build a heavy arch of brick or concrete over the pipe, and fill the space around the pipe with sand or good earth.

For shutting off the flow at any point in a pressure conduit gate-valves with screw-stems are used. Sliding-stem valves, check-valves, or any kind of valve, hinged or otherwise, by which it is possible to close the opening rapidly should not be employed where the pressure is more than 10 lbs. Valves on underground pipe are reached through boxes which surround the top and stem of the valve and extend to the surface, where they are closed by a cover. Valves on 12- or 15-inch pipe or less generally stand with the stem vertical, and the box is either a standard pattern of cast iron, or is built around the valve with brick. Larger valves would ordinarily rise too near the surface if erect, and are laid upon their sides; and the largest are worked by gearing. These larger valves would require that a manhole be built around them, similar to a sewer-manhole.

All rubbing surfaces of valves should be of bronze, and all parts sufficiently strong to resist the greatest pressure (including water-hammer) which can come upon either side. Each valve should be tested in place for leaks in the stuffing-box or elsewhere.

The location of valves and fire-hydrants has already been discussed. Post fire-hydrants are those most commonly used. They are generally placed upon the same side of the street as is the pipe (this giving the shortest connecting pipe or branch) and just inside of the curb. The branch pipe may be 4 inches for 2½-inch nozzles only, but if a fire-engine nozzle is furnished on the hydrant no less than a 6-inch pipe should be used. For ordinary localities two hose-nozzles, with an engine-nozzle if desirable, are sufficient. Each nozzle may be furnished with an independent valve, but this is often omitted. The main valve is placed at the base of the standpipe or barrel of the hydrant, and may be either a sliding gate-valve, or a compression valve; and the latter may close downward against or upward with the pressure. All these have been used with satisfaction; and if the material and workmanship are good, a clear and large water-way and easy curves at the bends are more important than the style of valve. It is desirable also that it be possible to remove the valve and stem without having to dig around the hydrant.

When the main valve is closed after use, the hydrant will be full of water. If this remains there in cold weather, it is apt to freeze and burst the hydrant, and hence a drip should be provided which will always be open when the hydrant is closed, and only then. Through this the water escapes from the hydrant, and should be led away before it can freeze. This is best accomplished by connecting to the drip a 1-inch pipe leading to a sewer or drain. If there be no drain near by, the excavation around the hydrant and branch may be filled with broken stone for a height of a foot or eighteen inches, which will receive the water in its interstices and from which it will gradually soak into the ground. If the soil is any but the hardest, the hydrant should be set upon a large flat stone to prevent it from settling into the ground when

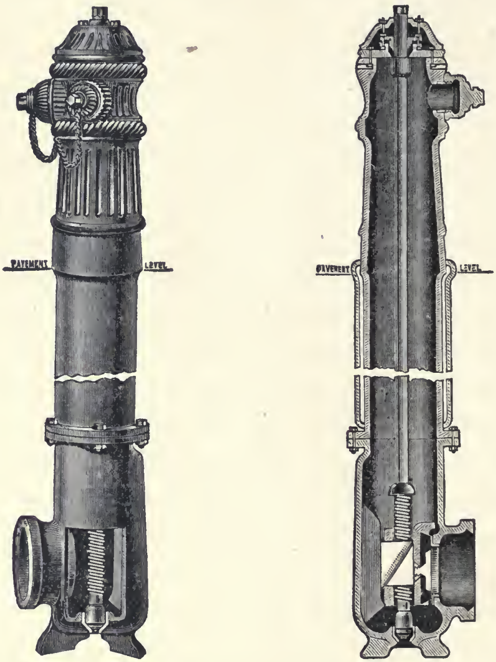

FIG. 77.—FIRE-HYDRANT (LUDLOW PATTERN).

this becomes wet from the drip. A large stone or small pier of masonry should be wedged tightly between the back of the hydrant opposite the branch and the solid earth to prevent the impulse of entering water from forcing the hydrant off of the branch.

If the branch leads from a main 10 or 12 inches or more in diameter, it should be provided with a valve-gate just outside the hydrant, that any injury to this may be repaired without closing the main pipe. A frost-case is sometimes used, i.e., a cylinder of iron fitting loosely around the hydrant-barrel at and for two feet or more below the ground-surface, to prevent the "frost" from "heaving" the hydrant, which the ground-surface in this case does not touch. The use of this, however, is being largely discontinued as being unnecessary.

The distance from the pavement to the branch should be at least as great as that to the main, since this is the more liable to freeze, there being no flow through it at most times.

When placed at a corner where a large and small main intersect, the hydrant branch should generally lead from the larger pipe. Fire-hydrants should not be placed in front of buildings needing especial protection or which would furnish a very hot fire, but should be preferably about 150 or 200 feet from such point on each side, as in the former position they would be inaccessible in time of fire on account of heat or falling walls.

Probably ten times as many fire-hydrants are injured by sprinkling-cart drivers as in any other way, where these take water directly from the hydrants. This should not be permitted, but "water-cranes"—contrivances especially designed for filling sprinkling-carts—should be placed at convenient points; and all but firemen should be prevented, under a heavy penalty, from opening a fire-hydrant. (At Newton, Mass., in 1894, 74 fire-hydrants out of a total of 760 were damaged by employés of the street and sewer departments.) A space under each crane should be paved to catch the drip and leaking of carts, and should be connected with the sewer, or the gutter if there be no other drain.

The size of the main conduits of an irrigation or city supply system are readily calculated from Art. 63 if the fall in the hydraulic gradient and the maximum quantity of flow are known. An irrigation main conduit should be capable of passing in an irrigation season of, say, 100 days the entire available supply; that is, the yield minus reservoir evaporation and seepage. The gradient is the grade of the conduit itself if this be open; if it be closed, but if there be an open conduit or reservoir at each end and the closed conduit be uniform in bore, it is the average fall from one open end to the other.

A city conduit must carry, with the available head, the maximum rate per minute and still leave a sufficient pressure-head at the point of supply. The gradient will then have one end at an elevation above the town equal to the desired pressure-head, the other at a distance below the reservoir-surface equal to the entrance and velocity-head. If the pipe be continuous and of uniform bore the gradient will be a straight line connecting these points. If a pump raise the water to the town, the further end of the gradient will be at a distance vertically above the pump equal to the pressure-head in the main at the pump. Thus, in Fig. 78, if *A* be

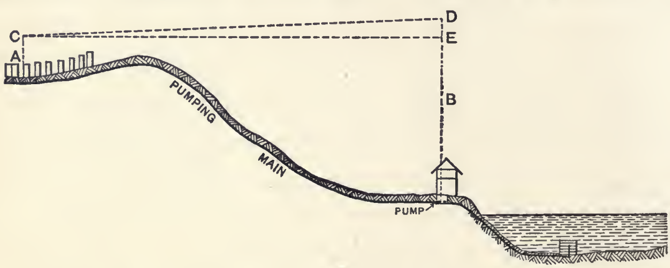

FIG. 78.—HYDRAULIC GRADIENT FOR PUMPING-MAIN.

the pressure-head at the town and *B* that at the pump, the line *CD* will be the hydraulic gradient, *DE* being the friction head. *B* can of course be made as great as desired, and thus the gradient be given any desired fall. *A* should be at least 150 feet for cities, if possible, and 100 feet for suburban districts and villages.

The maximum rate of consumption may be taken as about 200% of the average annual rate, plus the rate for fire purposes. This latter can be taken from Table No. 8, page 40. For example, for a city of 10,000 if the pressure and nozzle size be such as to discharge a fire-stream of 250 gals. per minute, 3000 gals. per minute will be the maximum amount required for this purpose; other consumption would have a maximum of, say, $\frac{10,000 \times 200}{60 \times 24} = 1400$ gals. per minute, or a total of 4400 gals. per minute as a maximum flow to be provided for.

The size of distributaries for irrigation has already been considered. That of the pipes in a city distribution system is, however, more complex. If *MA*, Fig. 79, is the main

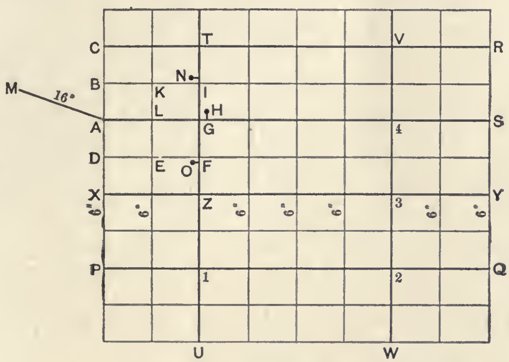

FIG. 79.—DISTRIBUTION SYSTEM.

conduit, and *H* a fire-hydrant in service, the supply for this will come partly through each of the pipes *BI*, *AG*, and *DF*, with perhaps some flow through *KE*. *BK* will supply a large part of the discharge at *N*, and *DE* of that at *O*, although a considerable amount for each will come through *AG* and *FI*. It *might* be possible to so arrange and proportion the pipe that the supply for any fire-hydrant would come equally through all connecting lines. But this would not be an economical arrangement, and the plan generally adopted is to design a skeleton system of mains and fill in intermediate streets with local distributing pipes. This gives more pressure along the main lines than on the intermediates, and hence these lines might be placed where are the highest or most important buildings, if there be any difference in this respect. In general, the mains may pass through every second or third street running in one direction, and every fourth to eighth in the other, the remaining streets being filled with 6-inch pipe, or perhaps 4-inch in short blocks (a practice not recommended by a great many engineers); it being assumed that all the supply passes through the larger mains, which is an error on the safe side. For instance, it is assumed that in Fig. 79 *AG* carries the entire supply for the line *TU* and all below, or $\frac{7}{9}$ of the entire supply; and that *VW* carries from *GS* $\frac{3}{9}$ of the supply, $\frac{1}{9}$ toward *V* and $\frac{2}{9}$ toward *W*. If the maximum supply for the city were 4500 gals. per minute, *MAG* should be of such size as to carry all this; *GT*, 1500 gals.; *GZ*, 3000 gals.; *Z1*, 2000 gals.; *1U*, 1000 gals.; *G4*, 3000 gals.; *4S*, 1000 gals.; *4V*, 500 gals.; *4–3*, 1000 gals.; *3–2*, 670 gals.; *2W*, 330 gals. Other considerations would change certain of these figures, however. For instance, under the above arrangement but two fire-streams are provided for in *WQ*, whereas five or six may be in use at once in these four blocks, and the ability to supply this number should be made the minimum limit for main lines, or, say, 1500 gals. per minute. Also allowance should be made for future extensions, generally in one pipe in each direction, as in *GS* and *TU*.

A 6-inch line should not extend for more than 800 to 1000 feet between principal mains; nor a 4-inch for more than 400 or 500 feet. If the vertical streets in the figure are to be filled with 6-inch pipes, it is hence necessary to supply more mains crossing the 6-inch lines, as at *TV*, *Z3*, and *1–2*. Each of these may be made of one half the capacity of all the branches leading from it; for instance, *TV*, *Z3*, etc., may have a diameter of $\sqrt{\frac{3 \times 6^2}{2}} = 7.4$, 8-inch being the next largest commercial size. The remaining lines may now be made 4-inch, if this size be used; otherwise these also would be 6-inch. If the pipes are subject to tuberculation, 4-inch pipe should certainly be avoided, the effect of tuberculation upon this size being so great.

In the above the term "capacity" of pipes has been used as a convenient one, but "capacity with a given uniform friction head" is implied. If 4500 gals. per minute is to pass through *MG*, the loss of head and hence the pressure head at *G* can be found. If we fix a minimum pressure head of, say, 150 feet to exist at *2* with six fire-streams playing in the section *WQ*, the head at *G* minus 150 will give that available for friction loss in carrying 1500 gals. of water per minute through *G4* and *4–2*. We have fixed the capacity of *4–3* as $1\frac{1}{2}$ times, and *G4* as $4\frac{1}{2}$ times, that of *3–2*. We may *assume* a size for *3–2*, as 8-inch, and *4–3* will then (see Art 63, (42)) be $\sqrt{\frac{8^2(1.5)}{\sqrt{(1.5)^{\frac{6}{11}}}}}$, or 9.26 inches; and *G4* will be $\sqrt{\frac{8^2(4.5)}{\sqrt{(4.5)^{\frac{6}{11}}}}}$, or 13.9 inches diameter. The head lost in passing 1500 gals. per minute through pipes of the given lengths and the above sizes is then calculated. If the total friction head thus found differs by more than 10% from that available (less than 10% need not be considered, as the commercial sizes of pipe will not permit of small changes in size), the assumed sizes—8, 9.26, and 13.9 inches—may be changed by use of the formula $\left(\frac{f}{f'}\right)^{\frac{6}{11}} = \left(\frac{d'}{d}\right)^{\frac{17}{22}}$, in which f and d are the total lost head as calculated and the assumed diameter of the pipe; f' is the available friction head, and d' the diameter sought. No considerable refinement in these calculations is called for since the commercial sizes of pipe will be used; and the assumption that all the flow to *2* passes through *3–2* is by no means true, since probably a fourth of it or more comes through *G12*; also the estimate of maximum rate of consumption can be made by no means accurately.

Where it is supposed that an extension will at some future time be made, a special should be inserted for this with a plug leaded and calked in the bell of the opening. At *Q*, for instance, a cross should be placed, and the right-hand branch plugged.

Art. 95. Standpipes and Tanks.

Standpipes are made of wrought iron or of steel; the majority in recent years being of the latter, although good wrought iron is probably more reliable than steel for this purpose. The side plates are generally of such width as each to build 5 feet vertical of standpipe, and are 6 to 9 feet long. The horizontal joints are generally single-riveted, the vertical one single-, double-, or triple-riveted. A few special designs have been employed, but generally a standpipe is circular in plan and of uniform diameter throughout, resting upon a masonry foundation. It is sometimes, but not generally, roofed over. Standpipes are much exposed to winds, and hence should generally be anchored in some way, or provided with guys. When placed upon level ground the lower 30 to 100 feet are in most cases useless for either pressure or storage, and this part of the pipe is frequently replaced by a tower, the whole thus becoming an elevated tank upon a tower.

The minimum height required for pressure is calculated by the methods already given. If the country be flat, to obtain a head which would enable fire-streams to be thrown to the desired elevation by gravity might require a standpipe

200 feet or more high. Hence the domestic supply alone is frequently considered, and steam fire-engines are relied upon for fire-streams; the head being sufficient to bring to any hydrants and connected steamers the desired quantity. There should be contained in the tank, above the elevation required for pressure, sufficient water to provide fire-service while the boiler-fires are being started up and the pumps enabled to supply the demand, even after a night's consumption without pumping. This probably should be about one fourth the daily average consumption plus the water required for fire-service during 30 minutes. This storage is supplied by a combination of height and width. If the height be great, the pressure on the lower plates of the tank is excessive, as is that in the distribution-pipes and plumbing, and the pumping must be against this great head. On the other hand, if the standpipe be made of great diameter it increases the cost both of this and of the foundation.

Whether or no a standpipe shall continue to the surface or shall rest upon a tower is determined largely by financial and æsthetic principles. The difference in cost of tower and standpipe will not often vary greatly, but for considerable heights or capacities the tower is probably the cheaper. A standpipe is at best an unsightly object, although some architectural beauty has been obtained by surrounding the pipe with a masonry tower in a few instances, as in Brooklyn, N. Y. A more graceful structure can be formed by a carefully designed steel tower, an example of which is given in Fig. 80*a*, the elevated tank of the Iowa State Agricultural College, designed by Prof. A. Marston. As he states: "The only legitimate means for enhancing the architectural appearance of an engineering structure of this kind are to select pleasing proportions and graceful outlines, and to employ only neat, strong-looking details. Any use of sham ornaments is entirely out of place." (*Engineering News*, vol. XXXIX.

page 371.) This tank is 24 feet in diameter and 40 feet high, with hemispherical bottom, the total height being 168 feet. A few tanks have been raised upon masonry towers, but this construction is not common.

The size of the tank having been decided upon, and its general design, the thickness of plates can be readily calculated. The stress in each side of the tank per vertical inch, due to the contained water, can be calculated as in the case of water-mains; the formula being $S = \frac{.434Hd}{2}$, from which $T = \frac{.434Hdf}{2se}$, in which S is the tension in each side per vertical inch, H is the height of water in feet above the point in question, d is the diameter of the pipe in inches, f is the factor of safety, T is the thickness of the plate in inches, s is the tensile strength of the material per square inch, and e is the efficiency of the riveted joints. f is generally taken as 3 to 5, probably the latter if steel be employed, or 2 to 3 if s be the elastic limit. e depends upon the size and spacing of the rivets, but is generally about .50 to .60 for single, and .65 to .75 for double, riveting. T should never be made less than $\frac{1}{4}$ inch, and seldom greater than 1 inch. The horizontal seams must withstand a stress due to the weight $\frac{W}{\pi d}$ per lineal inch, W being the total weight of the tank above the seam; and to $\frac{H^2P}{\pi d}$ per lineal inch due to wind-pressure, P being the pressure of the wind per square foot, generally taken at 40 to 60 pounds. The total maximum stress on the leeward side would therefore be $\frac{W + H^2P}{\pi d}$ per lineal inch of joint.

The bottom plates of a standpipe rest upon the foundation, and need to be of only sufficient thickness for proper calking and to allow for some corrosion on the under side without

FIG. 80*a*.—ELEVATED WATER-TANK.
(State College; Ames, Iowa.)

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leaking. One-half inch is generally sufficient. The bottom and side may be connected by flanging the edges of the bottom, but a better plan is to connect them by a ring of

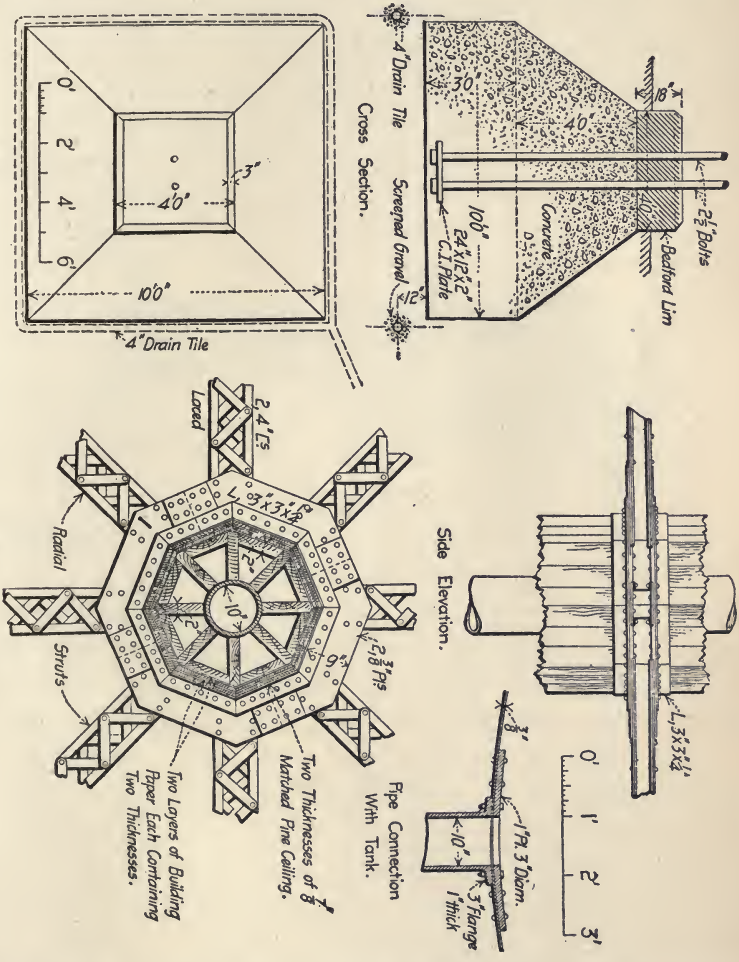

FIG. 80*b*.—DETAILS OF ELEVATED WATER-TANK, FIG. 80*a*.

angle-iron on either the outside or the inside, double-riveted to both side and bottom. The top should be furnished with an angle-iron stiffener to prevent collapse by the wind. A ladder should be fastened to the outside of the standpipe, extending from the top to within about 10 feet of the ground.

The rings may be truly cylindrical and fit, every alternate one inside the next above and below; or they may be slightly conical, the upper edge of each being outside the lower edge of the one above, to permit of effective calking while the standpipe is full. The contact surfaces should be perfectly clean when riveted. After riveting the pipe should be given three coats of paint inside and filled with water. All leaks are then calked, and when the pipe is tight the outside should

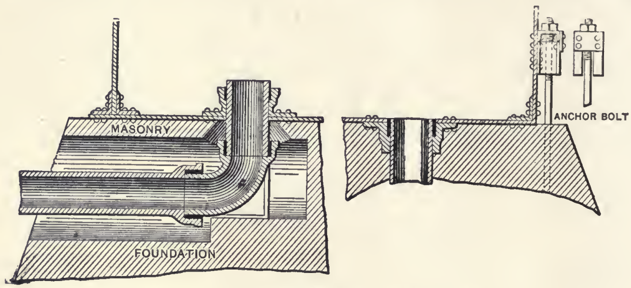

FIG. 81.—BOTTOM OF STANDPIPE.

be painted. The outside of the bottom should be painted before this is lowered onto the foundation.

An overflow-pipe is occasionally provided opening near the top of the tank, but this is often troublesome, especially in cold climates. Neither it nor anything else should be placed inside a standpipe or tank in climates where ice forms.

The height of water in the tank can be learned by a pressure-gauge at the pumping-station, unless there is no consumption; but consumption along the line causes considerable variation in the pressure recorded, and the only safe way is to provide a telltale communicating with the pump-room by wire, by which the height of water is indicated and a bell is rung when the water rises above or falls below certain elevations.

The same pipe generally serves for both inlet and outlet to the standpipe, and rises above the bottom of this a foot or two. Another outlet, flush with the bottom, may be provided for washing out collected sediment. The inlet-pipe should pass through a tunnel in the foundation sufficiently large to permit of its close inspection and repair. It should be furnished with a gate-valve just outside the standpipe, which should be under lock and key. The junction of this pipe with the standpipe is generally made by riveting to the bottom a flanged bell-casting, through which the pipe is passed, and in the bell an ordinary calked lead joint is made.

The standpipe should be bolted to the foundation, unless its weight, W, in pounds exceed PH^2. The bolts are generally 4 to 8 in number, 6 or 8 being preferable. They pass through the foundation, at the bottom of which anchor-plates are used, and are fastened to brackets on the side of the pipe near the bottom. These brackets may be made of short pieces of I beams and channels, riveted to the standpipe; one such arrangement being shown in Fig. 81. The maximum tension on each anchor-bolt will be $S = \left(\frac{M - WR\cos\theta}{NR}\right)^2$, in which S is the maximum stress in one bolt, M is the moment of the wind-pressure or $\frac{PH^2d}{2}$, R is the radius of the anchor-bolt circle, N is the number of bolts, and $\theta = 0$ if N is even and $= \frac{360°}{2N}$ if N is odd.

The foundation of the standpipe should be made absolutely solid, of first-class stone or concrete masonry. The tunnel admitting pipes to the standpipe should be substantially arched over. The top of the foundation should be perfectly level. A dry mixture of 1 part Portland cement to 1 or 2 of sand may be spread about 1½ inches thick over the foundation before the bottom is lowered onto it, a ring of the same wet into mortar being placed just under the edge of the standpipe. If there is any leakage in the standpipe bottom the cement will set up and tend to stop it. In any case the sand offers a cushion for the bottom and prevents all the weight from coming on the rivet-heads.

"Tank-steel" should never be used in standpipes. "Shell-steel," the next best grade, is not advised for use. "Flange-steel" is well adapted for standpipe use, being ductile and uniform in quality. It costs above 15 to 20 per cent more than tank steel, and 4 to 10 per cent more than shell-steel. A good standpipe-steel might be specified as "homogeneous steel made by the open-hearth process, having a tensile strength of 55,000 to 62,000 lbs. per square inch; an elastic limit of not less than 32,000 lbs. per square inch; an elongation in 8 inches of 20% in plates $\frac{3}{8}$ inch thick and under; of 22% for plates $\frac{3}{8}$ to $\frac{3}{4}$ inch, and of 25% for plates $\frac{3}{4}$ inch thick and over, with a reduction of area of 40, 44, and 50 per cent respectively. A cold-bending test to be made without signs of distress as follows: for plates up to $\frac{1}{2}$ inch thick, flat on itself; for plates thicker than $\frac{1}{2}$ inch, 180° around a mandrel having a diameter $1\frac{1}{2}$ times the thickness of the plate. The metal to contain not more than 0.04% phosphorus, nor more than 0.03% sulphur." As but a few kegs of rivets are used in a standpipe, it is not the general practice to test the material for these at the works, but to obtain the best from reputable manufacturers. A field test of value is to cut the head from a rivet which has been headed in the work. If the head snaps off, the metal is brittle and unfit; but it should gradually cut and tear off.

Elevated tanks are designed in the same way as standpipes; except that the bottom is often made hemispherical (as in Fig. 80, page 439) or conical; this construction being cheaper than steel beams and solid flooring under the tank. The trestle or tower involves the ordinary principles of structural designing. Each post should rest on and be anchored to a solid masonry foundation.

Art. 96. Estimate of Cost.

It is generally desirable, and frequently required by law, that a careful estimate be made of the cost of the work to be done. For this purpose, map, plans, and specifications should be carefully studied to obtain quantities, the character of soil or rock to be excavated should be ascertained, the accessibility of dams and other parts of the system considered, and in general as careful a study made of the conditions as a contractor would make before bidding. Also the prices of materials and supplies should be obtained, including the cost of getting them upon the ground, and from these as close an estimate made as possible of the actual cost of constructing the system. To this should be added 10 to 100 per cent for profit and contingencies, the latter amount when the work is to be done under great risks and subject to possible losses.

The cost of CLEARING AND GRUBBING reservoir-sites may often be more than met by selling the timber for lumber or fire-wood; but aside from this the cost of clearing ordinary timbered land will generally be from $15 to $50 per acre. If the land is very swampy, covered with scrub second-growth, or if large numbers of boulders must be removed, the cost may be greater.

EXCAVATION of loam by scraper, including first loosening it by plow, can be done for 15 cents per cubic yard if the average haul do not exceed 1000 feet. Add 5 cents per yard for each 1000 feet additional haul. Stiff clay or light hardpan will probably cost 30 to 35 cents per yard. If the quantity is less than 5000 cubic yards, or if it must be wasted in neat spoil-banks, the cost will probably run higher.

EMBANKMENT, i.e., spreading and compacting materials deposited from excavation, and facing the finished bank, can be done well for 2 to 4 cents per cubic yard if the necessary water can be had close at hand; otherwise add the cost of obtaining water. If the embankment is less than 5 feet high at either side, the cost will probably be greater, and small banks made in dressing up reservoir-shores may cost 20 or 30 cents per yard.

LINING banks with broken-stone and dry-stone paving 1 foot thick will cost about 5 to 10 cents per square foot besides the cost of the material. If the stones are irregular in shape and need much dressing and fitting, the cost may be increased two or three times.

PUDDLING will cost 25 to 40 cents per cubic yard in place, besides the cost of the material on the ground.

MASONRY for dam-faces will cost about $12 to $18 per cubic yard; backing in Portland cement 1 : 2, $5 to $8 per cubic yard. This will vary with the cost of the stone. One mason and helper should lay 7 to 10 yards of masonry per day; and each cubic yard of masonry will require about $\frac{1}{4}$ yard of sand and 0.85 barrel of cement. The cost of the stone and getting it to the wall, and of mixing mortar, must be added. Dimension-stone masonry for gate-houses, etc., may cost $25 to $50 per cubic yard.

TIMBER, as in foundations, will probably cost $5 to $15 per M.B.M. more than the cost of the material on the ground.

CONCRETE. Portland cement can be had for $1.80 per barrel and up; natural cement, from 65 cents up. Sand at all prices from 25 cents per load up. Broken stone costs about $1.25 to $1.75 per cubic yard. Mixing and placing, if properly done, will cost 50 to 85 cents per cubic yard. For a mixture 1 : 3 : 5 one cubic yard will require $1\frac{1}{4}$ barrels cement, 0.54 cubic yard of sand, and 0.9 cubic yard of broken stone. For thin layers of concrete on side slopes add 20 to 60 cents per cubic yard.

Flume BENCHES will probably cost 12 to 18 cents per cubic yard for excavation in loam, and 50 cents to $2 per cubic yard for rock.

TRENCHES for conduits in loam or loamy clay, sand, or gravel will cost about as follows. Sand and gravel must generally be sheathed. Hardpan will probably cost double this; quicksand, two to five times as much.

COST OF EXCAVATING AND BACK-FILLING, AND OF SHEATHING TRENCHES; DOLLARS PER LINEAL FOOT.

(Compact Loam; No Ground-water; No Machinery; No Street-paving.)

Depth of trench, feet..	4	5	6	8	10	12	14	16	20
4-to 10-inch pipe	.055	.065	.075	.10	.14	.20	.25	.33	.52
15-inch conduit................	.065	.08	.09	.125	.175	.24	.315	.39	.65
20- " "	.075	.09	.105	.15	.21	.29	.38	.465	.78
24- " "	.085	.105	.12	.175	.245	.34	.45	.545	.91
30- " "	.10	.12	.14	.20	.28	.39	.50	.62	1.04
Close sheathing...............	.20	.21	.22	.26	.32	.38	.50	.58	.75
Sheathing planks 4 feet apart..	.09	.10	.10	.12	.15	.18	.24	.27	.33

The cost of CLOSED CONDUITS is given in table on page 448 by A. L. Adams; the cast-iron pipe being assumed as costing $19 per ton (the latest prices in August 1899 are about $25 for large pipe and $27 for small; $19 is about the lowest price ever reached), and steel plates at $1.25 to $1.60 (the prices in August 1899 are 75 to 100 per cent higher).

CAST-IRON PIPE cost $24.75 to $28 per short ton in 1899; special castings, 2.7 to 3 cents per pound. The weight of pipe is given in Table No. 69, page 412.

LAYING cast-iron pipe costs about as indicated in table at top of page 449 (lead cost 4.55 cents per pound in August 1899).

The cost of VALVES, tested to 300 lbs. pressure is also given on page 449.

These prices are for iron bodies with bronze mountings, and hub or bell ends. Flanged ends will cost 5 to 10 per cent more; all iron, about 10 to 15 per cent less. Air-

COMPARATIVE COST OF PIPE AT CHICAGO.

(Including Laying, but not Hauling.)

Nominal Diameter.	Wooden Stave.				Steel Riveted.								Cast Iron.			
	25 ft. Head.	50 ft. Head.	100 ft. Head.	200 ft. Head.	No. 14 B.W.G.	No. 12 B.W.G.	No. 10 B.W.G.	No. 8 B.W.G.	No. 6 B.W.G.	$\frac{1}{4}$ in.	$\frac{5}{16}$ in.	$\frac{3}{8}$ in.	25 ft. Head.	50 ft. Head.	100 ft. Head.	200 ft. Head.
12	.42	.49	.63	.85	.32	.38	.44	......	......	......	......	......	.73	.77	.84	1.00
18	.69	.80	1.02	1.46	......	.57	.65	.78	.98	......	......	......	1.29	1.35	1.46	1.70
24	.79	.91	1.14	1.61	......	......	.85	1.04	1.28	1.55	1.99	......	1.91	2.00	2.18	2.55
30	.96	1.12	1.44	2.06	......	......	......	1.27	1.59	1.93	2.46	3.04	2.67	2.80	3.07	3.61
36	1.19	1.40	1.82	2.65	......	......	......	1.55	1.93	2.30	2.92	3.58	3.47	3.67	4.06	4.85
42	1.40	1.68	2.23	3.33	......	......	......	1.61	2.18	2.66	3.37	4.12	4.42	4.69	5.22	6.28
48	1.55	1.85	2.46	3.67	......	......	......	......	2.48	3.03	3.83	4.66	5.50	5.84	6.53	7.92
54	2.23	2.62	3.43	5.02	......	......	......	......	2.80	3.41	4.29	5.21	6.65	7.10	8.00	9.78
60	2.85	3.35	4.37	6.40	......	......	......	......	......	3.79	4.75	5.74	8.04	8.63	9.80	12.13
66	3.21	3.81	5.00	7.38	......	......	......	......	......	4.35	5.21	6.29	9.51	10.16	11.55	14.05
72	3.65	4.38	5.83	8.73	......	......	......	......	......	4.52	5.66	6.83	11.32	12.00	13.26	16.00

release valves for summits cost about $18 to $20 each. Valve-boxes cost $2.75 to $3.25 each for 4- to 10-inch pipe.

COST OF LAYING CAST-IRON PIPE; CENTS PER LINEAL FOOT.

Size of pipe.	4″	6″	8″	10″	12″
Unloading, hauling, and distributing	0.2 to 0.8	0.3 to 1.2	0.4 to 1.7	0.6 to 2.6	0.7 to 3.6
Laying (labor at $1.50)	0.4 to 0.5	0.4 to 0.5	0.5 to 0.7	0.7 to 0.9	0.8 to 1.0
Yarn (4 to 6 cts. per lb.)	0.04 to 0.06	0.056 to .084	.072 to .108	.09 to .134	.107 to .161
Lead (4 to 5 cts. per lb.)	2.06 to 2.58	2.95 to 3.70	3.80 to 4.75	4.80 to 6.00	5.55 to 7.00
Coke and furnaceman	0.15 to 0.25	0.15 to 0.25	0.17 to 0.28	0.19 to 0.31	0.22 to 0.36
Yarning and calking	0.4 to 0.6	0.5 to 0.7	0.6 to 0.8	0.7 to 0.9	0.8 to 1.0
Total	3.25 to 4.79	4.356 to 6.434	5.542 to 8.338	7.08 to 10.844	8.177 to 13.121

COST OF VALVES.

Size of valve	4″	6″	8″	10″	12″
Cost (in dollars)	6.00–7.00	9.80–11.25	14.75–17.00	21.00–24.00	22.75–30.50

Size of Valve	14″	16″	18″	20″	22″	24″	30″	36″
Cost (in dollars)	42.70–48.80	50.00–57.25	71.75–82.00	84.75–97.00	109.25–125.00	131.25–150.00	227.50–260.00	357.00–408.00

FIRE-HYDRANTS cost about as follows (2 hose-nozzles; bronze-mounted; 5 feet from ground to branch):

	6″ Connections.	4″ Connections.
With frost-cases	$22 to $27	$22 to $25
Without "	18 to 24	18 to 23

Add about $1.25 for each additional hose-nozzle, and $2.50 for a steamer-nozzle; $2.25 for each independent nozzle-gate. For each 6-inch increase or decrease in length of barrel add or deduct 35 to 50 cents. Fire-hydrants should average 7 to 14 per mile of pipe.

WATER-CRANES cost about $25 to $40 each for 3- or 4-inch branch-pipe connections.

DUG WELLS. A well at Webster, Mass., 25 feet diameter, 30 feet deep, cost about $13,000 (see Fig. 66, page 398). A well in Addison, N. Y., 12½ feet diameter and 23 feet deep, cost $2.18 per cubic yard for excavation (in clay and gravel), cement masonry $5.82 per cubic yard; the total cost being $637. The sheathing, pumping, and special details generally cost more than actual excavation and lining.

TUBE-WELLS at Savannah, Ga., 12-inch tubing, cost $4.50 per foot for the first 430 feet, and $5 per foot below that. In Pennsylvania, boring wells in the oil-regions cost about $1 per foot; in the Eastern part, 6-inch wells cost $2 up to 200 feet; from there to 500 feet, $2.75, and $3 below that. A 10-inch well on Long Island, 290 feet deep, cost $3.17 per foot, including tubing. In the Dakotas wells in 1890 cost about as follows:

Size, Inches.	Average Depth, Feet.	Cost per Foot.		
		Average.	Maximum.	Minimum.
8	912	$4.16		
6	840	5.28	$6.49	$2.15
5	1087	4.04		
4½	654	4.13	4.25	4.00
4	794	3.82	4.00	3.50
3	440	1.87	2.47	1.50
2	308	.87	2.61	0.42
1	717	2.68	3.25	0.50

In Texas the cost is from 60 cents to $2 per foot for 6-inch wells (not including tubing), averaging about $1 per foot for the first 100 feet, and 35 cents additional for each additional 100 feet.

The average for the fourteen Western States and Territories in 1890 was: depth, 210 feet; cost, $1.17 per foot.

PUMPING-ENGINES vary in price according to the duty required and, to some extent, the head against which they work. Bids for 20,000,000-gal. pumps (including foundations) in 1895 for the city of Brooklyn ranged from $62,000 to $100,235 each. A 2,000,000-gal. compound pumping-engine, duty 55,000,000, will cost about $3600; triple-expansion, duty 80,000,000, about $5400. A compound 750,000-gal. pumping-engine, duty 30,000,000, will cost about $2600; duty 45,000,000, about $3000; or duty 50,000,000, about $3200. These prices include all piping, but not the boilers. STEAM-BOILERS cost about $1000 for a 25-H.P. to $20,000 for a 200-H.P. water-tube boiler of high efficiency.

A 20-H.P. GAS-ENGINE will cost about $1200.

Two triplex power-pumps, capacity 500,000 gals. each, and two 25-H.P. gas-engines cost $3500 in 1898.

STANDPIPES cost about $2\frac{1}{2}$ cents per pound for erection and painting. Flange-steel cost 2.9 cents per pound in New York in August 1899. A trestle-tower for a standpipe in Jacksonville, Fla., 100 feet high and 65 feet diameter at the base, cost about $8000 in 1898. The tank and tower shown in Fig. 80*a*, page 439, cost $8966, the tank being 24×40 and the tower 110 feet high.

A MECHANICAL-FILTER plant will cost approximately $10 to $12 per 1000 gallons filtered per day, exclusive of building, foundations, pumps, or piping.

For an ENGLISH FILTER no general estimate can be made, the cost depending largely upon the grading necessary, ease of obtaining sand, and other local conditions. From $20,000 to $150,000 per acre would seem to be the range of cost in this country; $50,000 being perhaps a fair average.

Prices for pipe, pumps, etc., vary with the cost of metal. Before making an estimate the prices at that time should be learned from bids on recent work or from market quotations on metal.

Prices on excavation, grading, etc., will vary with the material handled and cost and efficiency of labor. The above are, however, deduced from a large amount of work in all sections of the country, and may be used for preliminary estimates when no local data are available. Labor in all the above has been taken at $1.50 and teams $3.50 per day. Additional allowance should be made for interest and depreciation of plant, as well as for contractors' profit.

PART II.

CONSTRUCTION.

CHAPTER XVI.

SUPERVISION AND MEASUREMENT OF WORK.

ART. 97. RESERVOIRS.

AFTER a reservoir-site has been cleared, but before any excavation is begun, levels of the entire ground-surface should be taken and a contour-map of the site prepared, with one-foot intervals between contours. After the completion of the reservoir, levels are again taken, referred to the same bench-marks and base-line, and the amount of excavation and embankment calculated by a comparison of these with the previous levels.

Before excavation, stakes for the direction of the contractor should be placed at frequent intervals along the lines of the inner and outer toes of the embankment, including in this the one or two feet on each side which is later to be removed in shaping and dressing the slopes. It is desirable to place monuments well beyond all danger of interference by the work, to which the centre line of the embankment or dam is referred; they being on each end of an extension of the centre line if this be straight.

During the construction of embankments the points requiring attention are: The width; each layer must extend the full required distance beyond the finished face. Depth of layers; this must at no point be greater than specified. Watering; the layers should be sprinkled uniformly in all parts, when wetting is required by the character of the material. Character of material; each load should be inspected, and no large stones, roots or other vegetable matter, or material different from that specified, should be permitted to be dumped. Form of surface; the edges should be kept higher than the centre. Rolling; the roller should be of the required weight, grooved, and passed over each spot until it becomes solid and homogeneous.

If a core-wall is used, this should be kept a little above the embankment, and the latter should be thoroughly tamped where the roller cannot reach it.

Particular attention should be paid to the puddling or concrete around any pipes which pass through the embankment; and to the total removal of all top soil under the embankment, and the compacting of the first layer with the ground thus uncovered. To insure the last, the ground should be plowed over after the removal of the top soil.

After the completion of the embankment templets are set and the slopes trimmed by these. On curves, templets should be not more than 25 to 50 feet apart, depending upon the radius. On tangents they may be 50 to 100 feet apart. Each templet is formed by driving a stout stake at the foot of the slope, and others 10 or 12 feet apart directly up the slope from this. Care should be taken that these stakes are in a plane normal to the axis of the dam or to a tangent to its curve at this point. To these posts planks are nailed in such a position that the finished slope will be a fixed distance, say 2 feet, under their lower edges. The slope is then carefully cut down to the required surface.

Masonry work requires the most careful inspection, and it is frequently necessary to discharge half the masons working before the remainder can be compelled to obey instructions. It must be remembered that the masonry must be not only strong, but also water-tight. The rock upon which it is to rest must be cleaned with brooms and then washed, and should be covered with a thick bed of rich mortar just before the masonry is started upon it. Every stone used should be perfectly clean of dirt and dust. It should be lowered into its bed and not rolled or barred into place. Each bed should be covered with an excess of mortar, and no mortar should be "slushed" in after the stone is laid. It is absolutely essential that no spall be driven under a stone after it is in place, to level it up or for any other purpose; but spalls and quarry-chips may be used in levelling a bed before the stone is lowered onto it. In using spalls and chips, however, these should never be placed in the bed and mortar spread over them, but the mortar should be spread first, and the spalls forced into it. All side- and end-joints as well as beds should be made in this way. There should be no grouting of dam masonry, for there should be no spaces for grout to penetrate. There should be no dressing of stones on the wall, but this should be done on the scaffold, or while the stone is suspended from the derrick. Courses should not be levelled up at all, but should be left as uneven as possible. The above are essential to a *tight* wall, although one on which less pains have been taken may be sufficiently *strong*.

The mortar should be rich—not more than 2 of sand to 1 of cement—and thoroughly mixed before wetting, until it is absolutely uniform in color throughout. The mixing is fully as important as the proportions of cement and sand. A slow-setting cement is preferable; and mortar should never be retempered for use, but on taking its initial set in mortar-box or -board should be at once thrown away. If the work is done by contract, it is a good plan for the party of the first part to supply the cement, and thus remove the chief incentive for parsimony in its use.

Concrete should be thoroughly mixed, and immediately put in place and rammed. The broken stone should be free of all dust or dirt, and should be wet before mixing with the mortar. Too much water will cause the concrete to be honeycombed and porous; there should be such an amount that light ramming will just bring moisture to the surface. Hard wood forms the best material for rammers; and the ramming should never be heavy, but just sufficient to compact the material. The faces of a concrete core-wall should be plastered with 1 : 1 cement-mortar as soon as the forms are removed and before the concrete is dry; and it is advisable to give the up-stream face two or three washes of cement-and-water to increase its imperviousness. All concrete should be kept damp until set, and should be shaded from the sun if possible.

For a distance of 12 to 20 feet above the ground a masonry wall can be laid to templets, carefully set and tested daily. Above this point the faces of the wall can be set by offsets from a transit-line run through its centre; or by plumbing up from known points in the face below, the transit-line being occasionally used for a check; the length of offsets depending, of course, upon the elevation of the point considered, with reference to that of the crest, which elevation must consequently be known. A convenient method for obtaining these elevations on low dams is to so place a level that its H.I. is the elevation of the crest; the rod-reading then being always the exact distance below this of the point on which it is held.

Ordinarily the test of a dam or reservoir is filling it with water. This should be done slowly, particularly in the case of earth embankments, slight porosities in which will close slowly by absorption of water unless the head above them be first made so great as to cause rapid percolation and leaks. Both masonry and earth dams may leak slightly for a few days or weeks after filling, but ultimately become perfectly tight. If, however, a leak begins to increase in volume, or the water is turbid, the reservoir should be at once emptied and the leak found and stopped.

Earthwork is generally paid for by the cubic yard, and is measured as described above. Masonry also is paid for by the cubic yard, there being generally two classifications in dams, face-ashlar and rubble backing, with a small amount of dimension-stone in gate-houses and ornamental work. The face-masonry may be paid for by the square foot of surface; or considered as extending for a given distance from the face into the wall, and paid for by the cubic yard. Gate-houses and similar structures may be contracted at a lump sum for all above the foundation; the latter being paid for by the cubic yard, since its depth is not generally known exactly beforehand.

Clearing and grubbing are generally done by the acre; the removal of top soil by the square foot or cubic yard. The handling of the water of any streams which flow across the work may be made a lump-sum item of the contract, or may be considered as included in the construction items.

Lining, whether of concrete, puddle, slope-wall, or other material, is generally paid for by the square yard.

ART. 98. CONDUITS AND DISTRIBUTION SYSTEMS.

Before the construction of a bench-flume, slope-stakes should be set as for a railroad cut (except that no part of the flume should come upon the fill), and levels of the surface be taken. After grading, centre stakes are set and the foundation-piers or sills are constructed, care being taken to have these the proper distance apart and at the exact elevations desired. In the construction of the flume the character of material, dimensions, and grade of the flume should be made to correspond to the specifications and plans.

The masonry in canal walls or lining should be built in the manner specified above for dams. Canal embankments also should be built as are earth dams.

In the case of stave-pipe the staves should be examined for wanes, shakes, splits, knots, or other defects; and the bands and shoes, for flaws of any kind. An eye should be kept upon the cinching, to see that it is not made too tight; and upon the butt-joints, to see that each is driven "home." A table should be prepared beforehand from the profile of the line, showing the proper spacing of bands at each station; and the stations should be designated upon the ground by stakes properly marked.

In constructing riveted pipe care should be taken to prevent the abrasion of the coating, and if this occur, the coating should be renewed by use of "P. & B." paint or other satisfactory compound. Rivet-holes should line up within a sixteenth of an inch, and rivets be well headed while hot.

Cast-iron pipe should be inspected for blow-holes, cold-shuts, plugs, or other defects, and tested by hammer for cracks while suspended over the trench. Each pipe should be sent "home" in the bell. Yarn should be firmly calked into the lead-space, of such thickness as to bring the bores of the two pipes into line, and leaving the required depth of space for the lead. The depth may be ascertained by use of a rule applied at various points around the inside of each bell. The lead for each joint should be run in at one pouring, and no cold-lead plugs used. No inspector can tell whether a joint is being so calked as not to leak; the test must develop this fact. It is a good plan to give all the calkers punches of different designs, and have each mark plainly with his punch each joint he calks. A deduction may then be made from the wages of each for all of his joints which leak.

The trenches for buried conduits should be inspected to see that they are of sufficient depth and width, dug to the proper line, of uniform grade on the bottom, and that no rock or large stones protrude in the bottom.

Bell-holes for cast-iron pipe should be of such size and location as to permit the barrel of the pipe to rest on the ground; and cross-trenches for riveters on wrought-iron or steel pipe should be of ample size to permit of good work. In back-filling, no large stones should be used before the filling is two feet high above the pipe; and selected material should be carefully tamped under, around, and on top of all pipe, especially riveted and wood-stave.

Before the excavation of trenches the centre lines of these should be marked with stakes spaced 100 feet apart on tangents, but closer on curves; and a stake should be driven at one side and directly opposite the point where any special device or structure is to be placed, and plainly marked to indicate what this is—as A.V. for air-valve, V. for valve, H. for hydrant, etc.

The stakes for shallow canals and distributaries are generally combined centre and grade stakes. These should be tested for grade just before the final grading of the canal bottom.

In city distribution systems the contractor may be given a list showing the distance of the pipe-line from the fence-line on each street. But if there be no fence, or the location be by offset from the street centre, centre stakes for the trench should be set. These should be a uniform distance—say 100 feet—apart, to facilitate finding them; and instead of stakes large spikes may be used, driven flush with the street-surface. A stake should be driven where each fire-hydrant is to go, and appropriately marked. Stakes should also be driven, just inside the curbing, opposite the location of each valve, special casting, or other break in the continuity of the pipe, and marked V, T, +, etc. After the pipe is strung in the trench, and before calking, the inspector should see that the instructions implied by these stakes have been followed.

The contractor should be supplied with a map of the piping, showing not only the size of pipe on each street, but also every special piece which is to be placed at each point; the latter information to assist the teamster in delivering these where they are needed. Fig. 82 shows a small section of such a map. Thus, at *A* is required a 10 × 4-inch tee, a

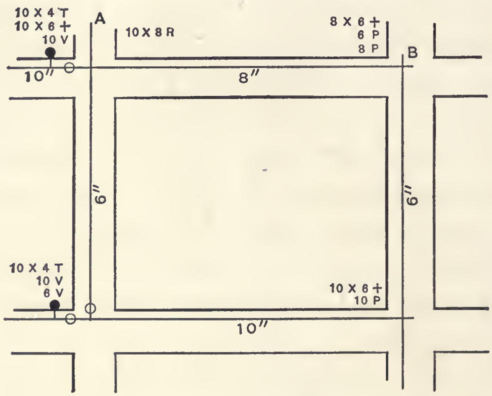

FIG. 82.—CONTRACTOR'S PIPE-AND-SPECIAL MAP.

10-inch valve, a 10 × 6-inch cross, and a 10 × 8-inch reducer; while at *B* an 8 × 6-inch cross, a 6-inch plug, and an 8-inch plug are required.

The inspector should see that the pipe is of the required weight (the weight of each pipe is marked inside of one end by the manufacturer) and without defect, and is placed at the correct location and depth; that all specials are placed where directed; that traffic and pedestrians are not interfered with more than is necessary; that blasting and other operations are so conducted as to endanger no lives; that valves and fire-

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hydrants are in good order, stuffing-boxes in shape for service, drip-connection (if any) carried to the drain or sewer, hydrants at the right depth, vertical, and well supported and braced as required. He should also see that the back-filling and paving are done as specified.

The engineer should keep a note-book in which is entered each line of pipe, its size, distance from the fence or street centre, and depth below the surface; with the location of each valve, hydrant, or special of any kind. The exact length of pipe, hydrant-connections, etc., is here recorded; the date of beginning and completing the construction, and any special work calling for extra payment.

Bench-excavation is generally paid for by the cubic yard of earth and of rock. Conduits of whatever kind are paid for by the lineal foot of each size, band-spacing, or thickness of shell; separate payments being made for material and construction, or one payment to cover both. Valves, hydrants, and other special devices are paid for at a price for each in position ready for use. It is customary to measure each line of pipe from end to end, making no deductions for any intermediate specials, these thus being actually paid for twice. Where a pipe changes size by a reducer, each size is considered as terminating in the middle of the reducer. Hydrant-branches may be measured as extending from the centre of the main pipe to that of the hydrant-barrel.

The price bid for laying pipe generally includes excavating, placing and calking the pipe and specials, and back-filling and repaving; but rock-excavation is generally paid for extra, by the cubic yard.

The price of cast-iron pipe and specials is usually stated per ton.

It is extremely desirable to test all conduits and appurtenances under water-pressure before they are covered. In sandy soil it is almost impossible to discover leaks in an underground pipe; although in clay or loam these ordinarily make themselves apparent at the surface when at all considerable. But even in such soil thousands of small leaks may escape detection if the pipe be covered, and these may and do cause a waste of 5 to 20 per cent of the consumption in many systems.

If iron pipe is exposed to the sun for several days, expanding by the heat and contracting at night, the joints are made to leak by the motion. It is hence desirable to test each section and cover it the day it is laid. If the section ends at a valve, the end is closed by this for the test; but if not, a temporary or false head may be fastened in the end of the last pipe. A convenient appliance for this is shown in Fig. 83, used by W. R. Hill on the Syracuse water-works.

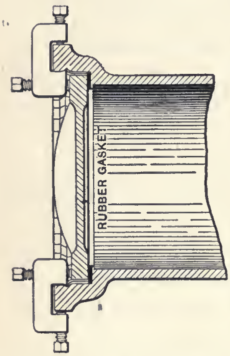

Fig. 83.—False Head for Testing Pipes.

If the pumps or reservoir be completed and ready for service before the conduit and distribution system are laid, water for testing may be introduced into them, and the pressure contributed, by these. In the majority of cases, however, the conduits and the reservoir or pump are being constructed simultaneously, and this is impossible. The test may then be made by filling the pipes by gravity or by a small pump from some stream, and applying pressure by a small steam- or hand-pump. The water can be introduced through a fire-hydrant or a special casting left unplugged.

The presence of air in a pipe-line during a test may result in unnecessary fracture of the pipe. If the advance end of the pipe is higher than the remainder, an opening must be provided here for permitting the air to escape; all hydrants and air-escapes must be opened; and all air removed from the system before the pressure is applied. If a small hand-pump is used to raise the pressure, its pipe may be connected with a hydrant-nozzle by means of a bushing, or it may be connected with the false head. While the pressure is on, all joints should be examined, and any leaks found should be calked until the pipe is tight throughout, when it may be covered. Valves and hydrants as well as pipe should be tested under pressure.

While pipe-laying is progressing the open end of the pipe should be closed with a plug each night to prevent the entrance of animals, or of stones, sticks, etc. Some refuse is almost sure to enter the pipes, however, and when they are first put into service all blow-offs should be opened and the pipes thoroughly cleaned.

Fire-hydrants should be inspected to see that their valves seat properly and are tight, and that the drip acts whenever the hydrant is closed. If the drip does not act, the barrel remains full of water, which is apt to freeze in winter; if it leaks, water is wasted and, if the drip is not connected with the sewer or drain, fills the ground around the hydrant, preventing the latter from draining when shut off.

ART. 99. OTHER FEATURES.

Pumps and boilers are generally set and connected up ready for service by the manufacturer, who is paid a lump sum. In a few instances it has been provided that this sum be diminished at a fixed rate if the plant fall short of a given duty, or increased if it exceed it. The testing of pumping-engines has been discussed in Art. 83. The quantity of water pumped is measured either by plunger displacement, with an allowance for slip; by passing the discharge over a weir or through a Venturi meter; or by discharging into a reservoir or standpipe of known capacity.

The foundations of the pumps and boilers are frequently constructed as part of the pumping-station. Plans, with dimensions, for these are furnished by the manufacturer, and must be closely adhered to. If bolts are to be built in the foundation by which to tie down the engine, a templet, with holes bored in the proper places through which the upper ends of the bolts pass, should be used for setting them.

In building the chimney, reliance should not be placed upon a batter-board, but a bolt should be fastened in the foundation at the centre of the shaft and plumbed down to after every few feet of added height. The foundation should be perfectly solid, carried down to rock or hardpan if possible, and of such size, unless on rock, as to give a pressure of but $1\frac{1}{2}$ to 3 tons per square foot. It should be carried as deep as the pump-pit, or as any other excavation within 50 feet of it, unless it first reach bed-rock.

The above requirements hold for standpipe foundations also. The standpipe is always erected by the manufacturer, and the tests are those conducted at the factory on the material, and that of filling the standpipe with water to determine leakage. A standpipe usually leaks when first filled, but small pin-holes generally fill up in a few days. After the pipe is erected, but before painting, it should be carefully examined for riveting cracks or other flaws. Defective plates can be removed from any part of a completed standpipe and replaced with but little trouble.

Wells are driven sometimes by the city or water company, more often by contract at a fixed price per foot. In some instances the contractor guarantees a certain minimum yield; in which case the guarantee should extend through six months or more of actual service, and should designate the depth to which the ground-water must be lowered at the well during pumping. In making a test-measurement the water discharged should not be permitted to flow upon the surface near the wells, as it may then enter the ground and be repumped. Samples should be taken every few feet of the material through which the well passes. Care should be taken to have every joint of the casing made air-tight.

CHAPTER XVII.

PRACTICAL CONSTRUCTION.

ART. 100. RESERVOIRS.

PROBABLY the easiest way to clear wooded land is to dig down to and cut off all tree-roots a foot or more beneath the surface, and then pull down the trees by a rope previously attached to them some distance above the ground. There is then no stump to remove. If there are stumps, these may be burned out, pulled out by a stump-puller, or blown up by dynamite. The trees being down, the brush is next cut with sharp brush-hooks (the Spanish-American machete is excellent for this purpose). This and the tree-tops may be burned in piles. A contractors' plow may then be used to loosen the soil for a foot in depth; a heavy-framed, long-toothed harrow will drag out most of the larger roots; and the smaller roots and loam can then be removed by scraper. This top loam should not be used for embankments, but may be used for grading up low spots above water.

For dressing up and banking the shores of the reservoir, casting by shovel and short wheelbarrow runs will generally be found the most effective methods. Where the soil is of loam, sand, or gravel, or any material easily loosened by plow, the scraper is probably the most economical method of excavating and embanking; unless the haul exceed about 1000 feet, when a cart filled by hand is more economical.

In either case the ground should be well loosened by means of a rooter-plow and a 2- to 6-horse team. A wheel-scraper is preferable to a drag for most work. In making embankment several scrapers should be used, and should travel a regular route, coming on the embankment generally at one or both ends, and leaving it at the other end or the middle. One scraper to each 300 or 400 feet of distance travelled by them will generally keep all hands busy; there being one man in the hole loading scrapers, one on the bank to dump them, and a driver accompanying each team.

If carts be used, two are generally provided for each team, one being loaded while the other is being hauled to the embankment and dumped; the number of men loading each cart being just sufficient to fill it while the team is absent with the other. This number will of course depend upon the length of haul and ease of shovelling. If the haul is of considerable length, it will be better to use three carts to two teams, or even four to three teams. Under some conditions, where time is a very important consideration, teams may be distributed at a uniform distance of 75 to 150 feet apart over the whole course and pass continuously around the circuit, being loaded by receiving a shovelful from each of 50 to 75 men as they pass them, and stopping only to be dumped. But this is not generally an economical method.

It pays to keep in good shape the road by which carts or scrapers travel from the pit to the dump. On a clay or sandy soil, which is heavy in wet or dry weather respectively, a good road can be made of plank spiked to two 12-foot stringers and made up in sections 8 feet long, with about 2 feet of each stringer protruding beyond each end of the section; alternate sections having their stringers at the ends of the road-plank and about one foot from the ends. This forms a portable road which will give a good surface if laid on levelled-off ground. The width may be 6 or 8 feet; and

18 or 22 feet B.M. per running foot is required for these widths (see Fig. 84).

Wetting of the embankment should be done by a spraying-nozzle on a hose or watering-cart. Puddles are formed if a bucket be used. In case a stream is flowing near by above the elevation of the embankment this can be brought for use by constructing a temporary earth, timber, or brush dam across it and from this carrying a flume to a point above the

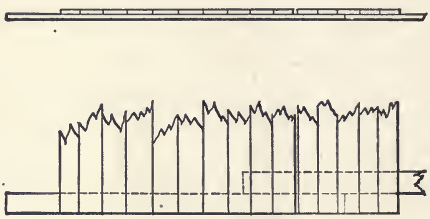

FIG. 84.—PORTABLE PLANK ROAD.

embankment. Here a gate can permit of filling a sprinkling-cart, or a hose can be attached; the surplus water discharging from the flume below the embankment. This method can often be employed on storage-reservoirs. If the flume would be too long, however, or the temporary dam too expensive, it may be cheaper to fill the sprinkling-cart directly from the stream by means of a boat-pump, diaphragm- or other hand-pump, and cart it to the embankment. In constructing a distributing-reservoir, carting water is generally necessary.

For dressing the face of an embankment a broad grub-axe is the best tool. A foothold is furnished by a plank having cleats nailed to it about a foot apart, which is kept from silding down the bank by a stake driven at its lower end. The bank is first graded under the templet, beginning at the top, and is surfaced by eye between templets. Selection should be made by trial of men who can obtain the most uniform surface in this manner. The surplus dirt can be used for filling in low places left for this purpose.

Puddle material, when made by mixing different materials, should be measured and turned over carefully by hand, with as much thoroughness as is concrete, before being put in place. It may then be thoroughly rolled; but a better puddle is made if it be rammed by barefooted or rubber-booted men with small-headed rammers, or if trodden by cattle. In India, goats are much used for this purpose, and they were used on the Santa Fé Dam in this country. If there be lumps of clay in the material, these should be fine-cut by spades before mixing; but if ready-mixed hardpan be used, this may first be rolled to break up the lumps, and then wetted and puddled. Puddle should never be wet, but only damp, and the materials should be thoroughly mixed.

Broken stone for lining must generally be carried to the top of the slope and from there lowered to where wanted, wooden chutes being convenient for this purpose. Slope-wall stone is delivered to the workmen in the same way. Concrete for slopes is frequently delivered through chutes; but when it is, it should be remixed before using, as the stone is sure to separate out. It is better to construct a wheelbarrow run along the slope, maintaining it at such a position, by frequently moving it up the slope, that the material can be dumped directly upon the top of concrete already in place. It should never be dumped upon the ground, to slide or roll down the slope. If a plank be held as shown in Fig. 85 to catch the concrete when dumped, none need be wasted. The run can rest upon supports driven in the bank as shown, and spaced about 6 feet apart.

Concrete on slopes should be laid in horizontal and not vertical strips; as by the latter method that at the bottom, not being set before the top is placed, will be pushed out of position by the weight of that above, and the whole strip of concrete tend to slide down the bank. Concrete lining should be lightly rammed, and carefully brought to surface.

Concrete for the bottom should be dumped where needed, and never shovelled up off the ground. Both here and on the slopes the proper thickness and surface elevation can be obtained by the use of horizontal strips on edge, the top edge being at the elevation of the concrete surface, to serve as guides (see Fig. 85, *a*).

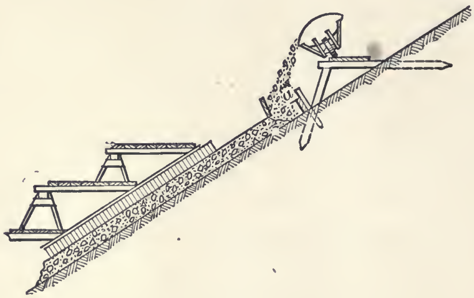

FIG. 85.—CONCRETING SLOPES.

Where the slopes are to be paved, masons' scaffolds can be constructed as in Fig. 85.

If a masonry dam be less than 20 feet high, probably the best way of handling the stone is by derricks on the ground. If it be higher than this, derricks can be placed upon the dam, and provided with main-falls sufficiently long to reach the ground and raise the materials from there. But the derricks must be raised continually as the dam rises; and this raising is a source of expense, and of delay unless done out of working-hours. To permit of raising, the guys should be adjustable in length. A gin-pole can be used for lifting the derrick-masts.

In some instances a travelling derrick is used, the track being supported on a scaffold which rests against one face of the dam. The cableway has been much used of late for bringing material to the dam, and is generally the most economical contrivance when the dam is of considerable size. A cableway 667 feet span between towers, used for constructing the Sodom Dam, cost $3750 erected ready for use.

Spans of 1500 feet and more have been used. The cable is generally used for bringing the material from the cars or carts at one end of the dam to where it is needed, but not for setting the stones, both because the cable extends only over the centre line of the dam, and because its swaying prevents a nice adjustment of the stone. Hand-derricks are generally used for setting the stone.

Water for mixing mortar and washing the stones can be brought by flume, as for embankments, or in casks by derrick or cableway.

In the construction of storage-reservoirs, the handling of the stream is frequently troublesome and difficult. It is generally necessary to carry it over the heart-wall trench in a flume. The outlet-pipes and base of the gate-house may be constructed first, and the stream turned through these, it being meantime carried around the structure in its natural channel, or in an artificial canal or flume, into which it is turned by a temporary diverting dam. The great danger is from sudden floods; and until such time as the dam forms a basin capable of holding so much of the entire maximum yield of a storm—say 3 to 5 inches over the entire drainage area—as the outlet-pipes will not discharge, this dam and flume should be able to carry such flood around the works. Should water overflow an embankment it may destroy it. If a masonry dam under construction is overflowed, it may afterward be necessary to remove the top stones because of cement washed out of and dirt deposited in the joints. Derricks and all movable plant are likely to be washed away, also. It is of course desirable to build the lower part of a dam during a season free from rain-storms; and this is particularly true of a cut-off wall, or such part of the dam as is below the natural surface.

ART. 101. DISTRIBUTION SYSTEM.

Iron pipe is generally delivered to a contractor in gondola cars on a siding, and must be unloaded and distributed by him. Ordinary 4-, 6-, and even 8-inch pipe can be loaded onto a wagon by rolling it down two skids about 9 feet apart placed from the wagon to the top of the car-side; it being controlled by a rope passing around each end, one end of which is fastened to the car and the other held by hand and payed out. Two men in the car lift the pipe onto the skids and hold the ropes, and one in the wagon receives it. For heavier pipe three or four men in the car and two in the wagon are required; or a yard-derrick or portable stiff-leg may be used for unloading.

The pipes are distributed along one side of the street, and care should be taken that they are the right size and that just sufficient are strung along each block to lay the line without any rehandling. In unloading, one end of the pipe is placed on the ground while the other rests in the wagon, a bag of hay is placed under the tail of the wagon, upon which the pipe falls when the wagon is started forward. Pipes are generally delivered on the side of the street opposite that on which they are to be laid.

In excavating trenches the paving is generally placed on the side toward the centre of the street, the dirt toward the sidewalk. By this arrangement the pipes do not need to be lifted over the dirt-pile when placed in the trench. Fig. 86 shows a method of preventing the excavated dirt from blocking the sidewalk and from stopping the gutter-flow. The first earth cast out should be thrown to what will be the outside edge of the bank, since it cannot be thrown there when the trench is deeper without double handling.

Excavating in winter is expensive, but must sometimes be done. It is facilitated by thawing the surface along the line of the trench. This has been done by the application of manure for a day or two before excavation, or of salt. But a cheaper and better method is that used in Providence, R. I. Two boxes, 72 feet long, 24 inches wide, and 12 inches high, are placed close together in parallel lines on the line of the proposed trench, all joints and the ends being covered with bagging and earth. These are then filled with steam from a

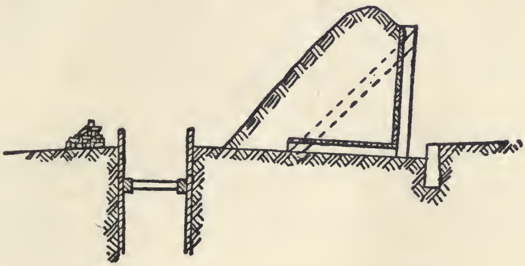

FIG. 86.—EXCAVATION-PLATFORM.

portable boiler through a $\frac{3}{4}$-inch steam-hose. The cost of thawing by this method was 1.7 cents per square foot.

In blasting rock, the drill-holes should be 3 or 4 inches deeper than the required trench-bottom; with certain kinds of rock or pitch of strata still deeper holes may be necessary. Each blast should be covered by bundles of branches or small trees tied into fascines, or by planks or logs chained together; the former better preventing small stones from flying. If the earth banks near by are not quite solid, they should be braced before the blasting. In case of boulders it is often cheaper to remove these bodily than to blast them in the trench at risk of caving the banks.

It is desirable to place the pipe in the trench as soon as possible. If caving of the banks then occurs, only the bells need to be uncovered for making the joints.

Four-, six-, and eight-inch pipe can be lowered into the trench by two men, by means of ropes passed around either end of the pipe. Each man slips under one end of the pipe, as it rests on the bank of the trench, a knotted end of a rope. Standing on this he holds the other end in his hand, and, by paying it out, lowers the pipe into the trench. Then, straddling the bank, these men lift the pipe clear of the trench-bottom, while a third pushes it home into the bell of the pipe last laid, a fourth meantime passing a strand of packing around the spigot of the pipe just back of the bead, and guiding it into the bell. The third man is provided with a wooden bar which he thrusts into the bell-end a foot or so, and by which he pushes and lifts the pipe. With this, under the foreman's direction, he lifts the pipe into the correct position. A straight line of pipe takes less piping to cover a given distance than does a crooked, the joints are more easily made and stronger, and it has a more workmanlike appearance.

For heavy pipe some other method of lowering is necessary, and some form of derrick is generally used; one light and easily portable being necessary. Fig. 87 (from *Eng. News*, vol. XXXV) shows a good appliance for this work, for heavy pipe. The derrick is carried along the trench by three men, and is rolled from street to street on its own wheels. For light pipe the drum and wheels may be omitted, the fall being payed out by hand. While being lowered, the pipe is guided by the two men in the trench. Before lowering, the pipe is rolled onto skids which span the trench, and which are removed when the pipe is suspended from the derrick.

The laying of pipe should be begun at a special whose position is fixed—as a valve, or an intersecting pipe-line. The pipes are laid in succession until near where another special comes, when this special is put in place and the closing length of pipe—which will generally be less than a full length—is temporarily omitted, the laying beginning again with this special. Behind this gang come two men who fit and cut the closure. The length required is accurately measured, and a piece about $\frac{1}{4}$ inch shorter is cut from a pipe, and is sprung into place by raising one or two pipes on either side of it, entering spigots into bells, and dropping all into position again. In cutting cast-iron pipe a ring is marked around the pipe for a guide, a 4 × 4 to 6 × 6 pillow-block is placed under the pipe at this ring, and by successive applica-

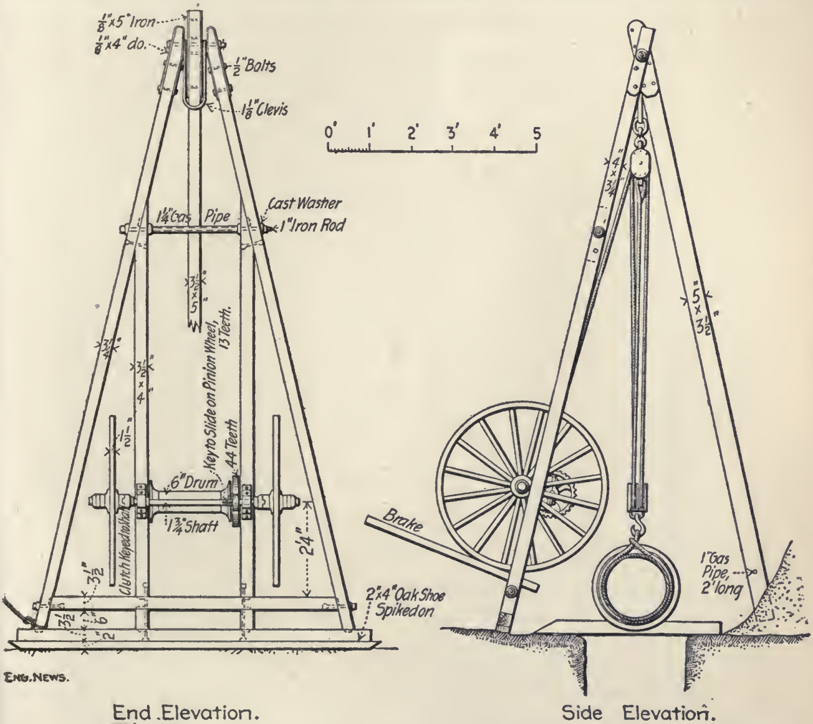

FIG. 87.—PIPE-DERRICK.

tions of a dog-chisel (Fig. 88, 1) and 8- or 10-pound sledge a slight cut is made around the pipe. This cut is made deeper and deeper, the pipe being meantime rolled back and forth along the pillow-block, and the blows of the sledge being made heavier and heavier, until at length the pipe breaks off at this ring. If the iron be good and the sledge-blows not too heavy at the beginning, there is little danger of breaking the pipe except where desired. Iron chips are apt to fly into the eyes of the workmen during this operation, and eye-shields of transparent celluloid are recommended for use.

Fire-hydrants are set and their branches laid, generally by this second gang, and the blocking placed behind the hydrant. All valves and specials and other appurtenances requiring lead joints are set in position, in order that the lead-gang may not need to revisit this line.

After the pipe is laid a "yarner" places the packing. A tightly twisted length of packing, long enough to go around the pipe, is inserted between the bottom of a bell and its spigot, and pushed back to the shoulder of the bell; the spigot being lifted, if necessary, by driving a steel wedge between it and the bell, until the annular space at the bottom is slightly greater than that at the top. The packing is then calked tightly back against the shoulder of the bell all around, more being added where necessary to leave just the required depth of lead space. Tarred packing can be calked harder and is easier to handle than untarred. To hold the molten lead in the joint when it is poured, either a "roll" or a jointer are required. The former is made of well-tempered fire-clay and a strand of rope a little longer than the circumference of the pipe. The rope is embedded in a roll of the clay, and this is rolled back and forth on a smooth, wet board until a roll of about an inch in diameter is formed with the rope for its core. This is placed around the pipe against the face of the bell, the rope-ends on top, and pressed firmly against both spigot and bell-face, the two ends being brought together on top a little distance from the bell, so that a pouring-hole is formed between the bell and the roll about 1½ inches across each way. Into this hole the lead is poured slowly but steadily until it is about to overflow the pouring-hole. In 10 or 15 seconds the lead will be hard, and the roll may be removed and smoothed up for the next joint.

The weight of the lead frequently forces off the roll, particularly on large pipes, and the lead runs out. For this reason and because of their general convenience patent jointers, made of canvas, asbestos, and other substances, are to be preferred.

The lead is melted in a kettle supported in a small portable furnace, in which charcoal is ordinarily used for fuel. The furnace is generally simply a sheet-iron cylinder with grate and coal-door at the bottom, and two dogs at the top for supporting the lead-kettle; but furnaces can be purchased mounted on wheels and with short stove-pipes attached to the closed top. The molten lead is carried from the kettle to the pipe in a ladle, from which it is poured into the joint. If the pipe be larger than 10 or 12 inches, a two-handled ladle is necessary, handled by two men both on the surface and in the trench; and for very large pipe the lead is poured from a kettle suspended over the joint by a light derrick.

When the roll or jointer is removed, the lip of lead at the pouring-hole is cut off, a chisel is driven lightly between lead and pipe all around, and the calking tools are then used (see Fig. 88). The lead at the bottom is compacted first, and calking proceeds up both sides until the top is reached. In calking pipe up to 16 or 18 inches in diameter, the calker passes one arm around each side of the pipe, holding the calking-tool in one hand and the hammer in the other. For calking very large pipe the men work in pairs; each hooking a leg and arm in that of his fellow, one holds a calking-tool and the other the hammer in his free hand, and they are thus able to reach the bottom of the pipe. The lead should be upset and thoroughly driven until there is no more "give" to it. The lips and other pieces of lead are saved and remelted.

The line is now ready to be tested; and after all leaks are stopped by recalking, the trench may be refilled. The author's remarks in "Sewerage" (pages 212 and 268) on back-filling and repaving are applicable to water-pipe trenches also and will not be repeated.

In placing the ordinary extension valve-boxes, their tops should be set a little below the top of the filling, and two or

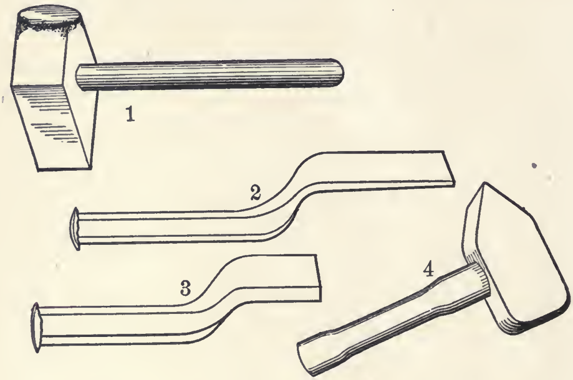

FIG. 88.—YARNING AND CALKING TOOLS AND HAMMER.
1. Pipe-cutting Chisel. 2. Yarning-iron. 3. Calking-tool. 4. Calking-hammer.

three times as much above the original street-surface; when the trench has settled they will then be flush with the surface, as they should be.

Every end supposed to be plugged should be inspected just before it is buried, to make sure that the plug is there and leaded in and calked.

In crossing streams under water the methods described in "Sewerage," Art. 79, may be employed. Many other plans have been followed however, the details varying in almost every case. At Delray, Mich., a 72-inch pipe was put together in two sections, each about 95 feet long, bulkheads were built in each end of these, they were floated over a trench dug in the bottom to receive them, and sunk to place by admitting water into them, the abutting ends being connected by a diver. In another case a main 119 feet long, weighing 63,300 lbs., 48 inches diameter, was lowered as a whole by derricks. In other cases pipes have been lowered singly and jointed by divers. But in the majority of cases flexible joints are used, on every pipe or at intervals of several pipes. The pipe is then laid from scows, through the ice, or connected on shore and dragged across the river-bottom. Several methods are described in the Transactions of the American Society of Civil Engineers, vol. XXXIII. pages 264, 273, 281, 284, and vol. XXXIV. page 31. A 12-inch flexible-joint pipe laid across Hell Gate, East River, N. Y., cost $7 per lineal foot in place.

ART. 102. WELLS.

Dug wells for water-works are generally not less than 12 feet in diameter, and should always be sheathed and braced. They are generally circular in plan, since a given amount of material will thus make a stronger wall and give greater capacity. The sheathing, however, is seldom made circular, owing to the difficulty of forming the wales or ribs. (Sheathing is not required when the walls of the well are built above the surface and sunk bodily into the ground.) It is generally made square or octagonal in plan, each course of wales being composed of four or eight lengths of 4×4 to 12×12 timber, strongly braced together at the ends; as at *a* and *b*, Fig. 89. It is not advisable to make any one wale-piece more than 8 or 10 feet long, and 6 feet is a safer limit unless braces are used across the well. This limits an unbraced well to 12 to 20 feet outside, or say 9 to 16 feet inside, diameter. Braces across a well which is larger than this require to be stiffened against buckling, and the danger of collapse increases rapidly with the diameter. For large wells the method *d* may be adopted, the excavation being practically an annular trench around the earth core *d*, which can be well braced, and will not collapse because of the failure of one brace or wale. This trench is carried to the required depth and the wall is built in it, earth being packed between the wall and the outer sheathing as the former is built, and all braces being left in. After the wall is completed the centre core is excavated and inside sheathing removed. The

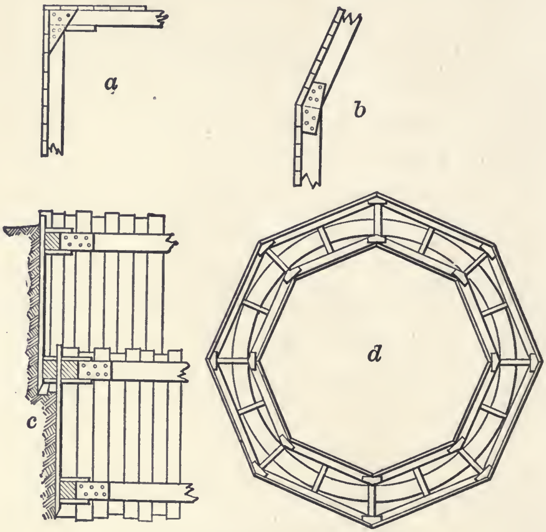

FIG. 89.—SHEATHING DUG WELLS.

braces may be sawed off at the inside face of the wall; or may be pulled out and the openings in the brickwork filled up, or these may be left to serve as weep-holes. This construction can be used for any size, shape, and depth of well.

With either of these methods the earth is best removed by "staging" it out, down to about 15 feet, below which a derrick and large dirt-buckets are more economical. The derrick may be placed on *d* in the annular-trench plan, but is preferably placed a few feet back from the well on firm soil. If the well be large, two or three derricks will be necessary. The booms should be long enough to remove the dirt some distance from the well; unless it be immediately carted away, which is the better plan, since the weight of extra dirt on the surface around the well adds to the strain on the wales and braces, and none should be placed within 20 or 30 feet of the excavation. At 30 feet depth a horse-power derrick can handle three half-yard buckets filled by six men; below this not so many diggers per derrick can be used. With a steam-power derrick four one-yard buckets and 15 to 18 men can be kept busy. These quantities are approximate averages.

Another method of sinking wells, which does away with bracing, consists of sinking the masonry lining into the ground by its own weight. A stiff timber or iron footing is placed in the bottom of an ordinary hand-dug well 8 or 10 feet deep, and on this the masonry wall is built. When the wall has been carried to the ground-level, the soil is excavated from under the footing equally all around, and the wall sinks of its own weight. This operation is continued, the wall being kept built up to the surface-level, until the required depth is reached. The footing is generally a foot or two larger than the wall, and is frequently provided with a sharp cutting-shoe around its outer edge. The outside of the wall should be plastered and made smooth, or board sheathing placed around it nailed to furring built in the wall, to reduce the friction between the wall and the soil if the latter caves, as it generally does. The greatest danger is that the well will get out of plumb by the more rapid settling of one side than of the other. If the wall "hangs" at any time and refuses to settle vertically, the excavation under the footing should be stopped at a uniform level all around and not more than 6 to 12 inches under the footing, and not continued until the wall has settled down to a bearing. Care must be taken that the excavation under the footing is at all times level. A "hung" wall is lowered by building it higher or loading it on top with pig iron, helped in extreme cases by the careful application of water-jets around the outside of the well to loosen the soil.

In all well-work the water must of course be kept out. For this purpose a pulsometer is excellent for handling small amounts of water, a centrifugal pump for larger amounts. Reciprocating pumps are frequently used, but are subject to great wear from muddy water; they have the advantage of compactness, however. A centrifugal pump is generally run by belt from the surface. If electricity is available, an electric motor geared to the centrifugal pump is advisable, since the belts are apt to slip in a damp well, and are greatly in the way.

Pump-pits are built in the same manner as are wells, except that their walls are made heavier and water-tight and a water-tight foundation is provided.

Small 1½- or 2-inch wells not more than 30 or 40 feet deep are sunk through the softer soils by simply driving them down with a hammer; but this is generally impracticable through hardpan and large gravel, and of course through rock. Such small, shallow wells are not advised for water-works, but 2½ to 10-inch wells through clay or rock should generally be employed. The best method of sinking these in most cases is by the "jetting down" process. A small pipe, whose diameter is about one third to one half that of the well-casing, is placed inside the casing and water is forced through it by a pump, passing up between the two pipes and out at the top of the larger. The jet from the bottom of the water-pipe loosens the soil, and it is carried up by the ascending water. As the soil is thus removed the casing is driven down by a hammer. The water-pipe is generally given a churning up-and-down motion in firm soils. In sand or loam it will in most cases drop more rapidly than the casing can be made to follow. An arrangement for this work is shown in Fig. 90. As both water- and casing-pipe are lowered they are extended by screwing additional pieces onto their top ends. The casing is held upright, and the water-pipe is supported and churned, by a derrick. Ropes from the water-pipe and hammer pass through sheaves at the top of

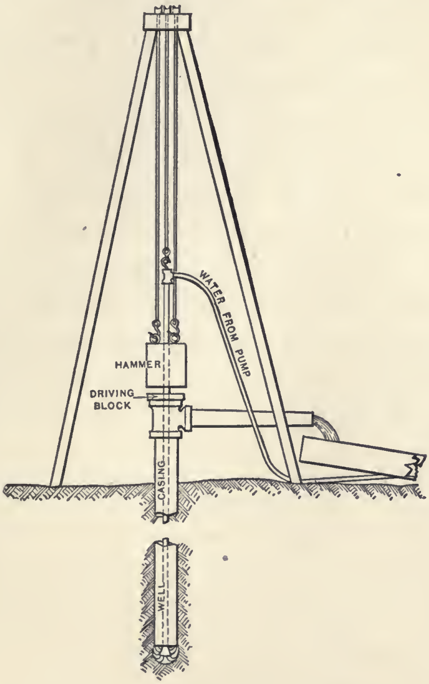

FIG. 90.—APPARATUS FOR JETTING DOWN WELLS.

the derrick to a two-drum hoisting-engine, or a cam-and-lever well-boring engine. In sand and clay 10 to 20 feet per hour can be sunk by this method. The most difficult material is gravel, the jet-water passing out through the gravel and not up the casing.

Rock of course cannot be bored by this method, but when this is encountered either a diamond-drill is used, or, as is more common, a two-, three-, or four-sided cutting-drill, to which are fastened drill-rods and "jars" to increase the force of the blow, the drill being raised, revolved and let fall with each blow. The drill is lowered through the casing. If the casing is to follow through the rock, an expansion-drill is used. The chips are washed out of the hole by water-jet.

A casing can generally be driven 400 to 800 feet, after which the friction becomes so great as to prevent further motion. A smaller casing is then lowered inside this and carried down another 400 to 800 feet.

The casing is generally lap-welded wrought-iron pipe, with screw-joints; the coupling being made by a sleeve; by a "flush-joint," i.e., half the thickness of the metal is removed from the outside of one pipe and the inside of the other for a distance of 3 to 6 inches from the end, and threads are cut on these, the pipes when screwed together being thus flush both inside and out; or by an "inserted" joint—one end of each pipe being expanded and threaded inside, thus practically forming a sleeve.

PART III.

MAINTENANCE.

CHAPTER XVIII.

RESERVOIRS, HEAD-WORKS, AND INTAKES.

ART. 103. MAINTAINING QUALITY OF WATER.

THE necessity for maintaining a watershed free from all sewage or excreta has been explained. The most desirable means of effecting this is to purchase such watershed entire; but this is seldom feasible. As much of a margin around the reservoir as possible should be obtained, however; and such laws as exist or can be got through the legislature for preventing the pollution of reservoir-sites and watersheds should be strictly enforced.

Vegetation should be cleared from the borders of a reservoir at least twice a year, and nothing but lawn and evergreen trees and shrubs should be allowed within 50 feet of the shores. No loafing, picknicking, or bathing around or in the reservoir should be permitted; but driving or walking around it on roads or paths it is well to encourage. It is desirable to construct a gutter a little distance back from the shore which will catch all surface-flow and lead it into the reservoir without washing the roads or paths.

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No cattle should be allowed to enter the streams feeding the reservoir. If the land through which these pass cannot be purchased, arrangements can sometimes be made with the farmers—such as constructing new cow-stables for them for pasturing their cattle elsewhere, and cultivating the banks of the creeks.

Swamps are not desirable on a watershed, and should be drained or filled in; and no stagnant pools should be permitted anywhere. It is seldom that a farmer will object to replacing a surface-privy with a water-tight cesspool, if it be at the expense of the city or water company; and this should be done, and the cesspool examined at least once a year to insure its being properly cleaned. The contents of cesspools should be removed from the watershed. In every way the good will of the farmer should be kept; and the consumers should also be encouraged to make their influence felt with such of them as they may come in contact with in a business way, to the end that the watershed be kept free from pollution.

If vegetable matter left on the reservoir-site or carried in by the run-off causes algæ or other pollution, every effort should be made to prevent the water distributed from suffering thereby. Only the purest water in the reservoir should be taken into the conduit; before the fall turn-over the foul bottom-water should be drawn off; and at least once a year advantage should be taken of low water to clean the upper slopes of the reservoir-bottom, care being taken not to pollute the water. Grass and weeds should not simply be mowed, but the cuttings should be removed and not permitted to fall or blow into the reservoir.

In short, all organic matter should be removed and kept out of the reservoir at all times. This does not apply to fish, which are beneficial to a reservoir, and a number of clean lake-fish should be supplied if not already present. The fish-screens should be kept clean at all times. An excellent scraper for cleaning screens of sheet brass is the rubber "squegee" used for cleaning large windows.

If organic matter is found to have collected in a reservoir, it should be removed and the bottom and sides of the reservoir thoroughly cleaned. The beginning of the rainy season is the best time for cleaning a reservoir. A storage-reservoir should not, of course, be emptied except when the distributing-reservoir contains sufficient water to last for several days, including fire-service. If there be no distributing-reservoir, small pools of water may be made at the mouths of the feeding-creeks by temporary earth dams, and the reservoir then emptied and cleaned; the water from these pools being led to the conduit by wooden flumes laid on the reservoir-bottom, and the supply thus maintained.

When distributing-reservoirs are to be cleaned, the water from the storage-reservoirs or pumps is generally carried direct to the city through a by-pass around the reservoir. The cheapest method of cleaning a small reservoir with paved bottom and sides is, generally, to flush out the sediment with water. In the case of a distributing-reservoir a hose can often be used, the water being obtained under pressure from the pumps or the storage-reservoir. The waste- or mud-pipe being open, cleaning is started at the end furthest from the outlet and at the highest point needing it; and is continued around the upper edge of the sediment, which is washed and scraped toward the outlet by hose and scrapers similar to snow-shovels, the whole of the material being worked from all sides toward the waste-pipe; the surplus water meantime flowing out through this and carrying away the lighter matters. The algæ and some other deposits are gelatinous until dry, when they form a leathery layer difficult to remove in this way. For this reason the sediment should be kept wet; generally by closing the waste-pipe at night, thus flooding the reservoir-bottom until work is resumed in the morning. It is often desirable to empty a reservoir slowly, cleaning the bottom as the water lowers, and thus giving the sediment no opportunity to dry.

In large reservoirs or those with unprotected earth bottoms or having no mud-pipe this method is inapplicable, and the deposit must generally be removed by either wheelbarrows or carts. It is desirable to complete the cleaning without loss of time, but the material should be as dry as possible, and to effect this the reservoir should be emptied quickly. For this reason, also, dry, hot weather is to be preferred; but it is generally inadvisable to empty the reservoir at a time when it cannot be quickly refilled.

Whether wheelbarrows or teams are used for removing the deposit will depend upon the same conditions as in the case of reservoir-construction.

Water for irrigation should be kept free from sand and gravel, which is also necessary to prevent rapid destruction of the conduits; but few, if any, other matters which are ordinarily received by stored waters are objectionable. Sand is excluded by properly arranged head-gates, but must be kept cleared away from these by use of the flushing-out sluices (Fig. 64, page 388). Experience will soon show how often these must be used in any given plant; in some cases after every storm, in others only once or twice a year, while in a few cases it may be desirable to leave the sluices open during every flood to prevent an accumulation of sand which might, during one storm, exceed the intercepting capacity of the gates.

ART. 104. MAINTAINING QUANTITY OF WATER.

The reservoir should be kept as tight as possible. Any leaks which develop should be repaired; and if the dam be of earth, it should be emptied immediately if the leak appears to be increasing or if the water from it is at all muddy, as a rupture is then imminent. If the water run perfectly clear, however, and do not increase in quantity, such desperate measures may not be necessary, but the repair may be postponed to a more favorable time.

The upper end of a leak in a masonry dam is exceedingly difficult to find. If chopped hay or sawdust be thrown into the water over where the leak is supposed to be, it will often be drawn into the opening, sometimes stopping the leak, but at least indicating its location when the water is drawn down. The same plan may be adopted in the case of an earth embankment; but a leak in this is generally apparent, having a funnel-shaped opening in the reservoir. It may be possible to locate the horizontal zone in which the leak exists by emptying the reservoir and refilling it slowly, noting at what elevation of water the leakage again begins.

A leak in the masonry is generally repaired by thoroughly cleaning the opening, forcing rich Portland-cement mortar into it until it is completely filled, and plastering the face of the wall with the same. If there is a general leakage, it will be well to clean the wall thoroughly and face it with 4 to 6 inches of concrete, plastered with rich cement-mortar. A leak can never be repaired from the down-stream side, but only from the up-stream or water side.

A leak in an embankment may be very troublesome to stop. If it be small, forcing sand and clay into the hole, together with a little water, may be effective; or the upper end of the opening may be enlarged and compactly filled with dry puddling material well mixed and then moistened slowly, sheet-piling being in some cases driven across the opening, extending several feet on each side of, below, and above it, the puddle being compacted behind this and good embankment material in front of it. The permanent stoppage of a leak will sometimes require that a V-shaped cut in the embankment be made down to the leak, and refilled with the best of material thoroughly puddled as dry as possible.

Loss of water by evaporation can generally be decreased only by reducing the area of the water-surface; although it is possible that a high board fence around a small reservoir would have this effect by shutting off the wind, and it at least would serve to keep leaves out of the reservoir to a large extent. The exposed water-surface can be reduced in many cases by draining all ponds on the watershed, and storing no water except in the reservoir; and all watercourses should be kept clear and open for the same reason.

The slopes of the watershed are in their most desirable condition when covered with sufficient vegetation to prevent rapid evaporation and erosion of the soil, but of such character and amount as will extract the least amount of moisture in their growth. Slowly growing trees seem to meet this requirement, while weeds and other plants of rapid growth deplete the ground-storage unnecessarily.

It is not sufficient to get the maximum amount of water into the reservoir as quickly as possible, and to prevent loss of this by leakage; but that lost over the spillway should be reduced to a minimum, and that stored should be used to the best advantage. The first requirement is met if all distributing-reservoirs be full when heavy downpours occur. The use of flash-boards in the spillway is recommended only in extreme cases, and even then they should be so made as to break or in some way automatically and without fail be removed before the water rises to a distance below the top of the embankment equal to the maximum wave-height.

Economy in use can be partially controlled from the distributing-reservoir if there be one, or otherwise from the storage-reservoir, by regulating the amount of water admitted to the distribution system. If this be less than the normal consumption, the upper part of the conduit will be only partly filled, the head will consequently be reduced and the flow from each fixture become less. In such a case arrangements should be made for instantly opening the gates at the reservoir and obtaining full head in case of fire. Other methods of regulating consumption will be considered in Art. 110. If the reservoir-bottom is not already so graded as to completely drain from all parts to the outlet and thus utilize all the water stored, the first opportunity should be taken for effecting this. Probably the greatest opportunity for economy occurs in selecting the times for necessary wasting of water, as in cleaning the reservoir, removing water from the stagnant layer, etc. These should be selected at the beginning of a season during which the yield exceeds the consumption; for if before this, the ordinary supply cannot be fully provided until this season begins; and if after, all surplus yield between the beginning of this season and the emptying of the reservoir is lost.

The storage-reservoir is designed to supply the deficiency due to a certain dry-weather rate of consumption. When the consumption is found to be exceeding this rate, additional storage should be provided, unless the rate can be reduced by some method. As a check upon the original calculations, and an aid in preparing plans providing for increased consumption, one or more rain-gauges should be maintained upon the watershed, and the daily evaporation at the reservoir measured. It is desirable also to obtain the temperature and humidity of the air, and velocity and direction of the wind, to assist in studying the run-off phenomena. A daily record of the height of water in the reservoir should be kept, and the increase of height due to each storm should be noted. An accurate topographic map of the reservoir should be had, and from this the cubic feet of water corresponding to each foot of elevation ascertained; a table showing these being prepared for convenient use. During rainless weather the drop in the reservoir will indicate the consumption plus evaporation and seepage, and the latter should be known that they may be deducted to give the consumption.

There are many reasons why such calculations of consumption can seldom be accurate, and it is desirable to provide a meter on the supply-main near the reservoir. On any but the smallest systems the Venturi meter is probably the one best adapted for this purpose. The reading of this should be recorded at least once a day, and twice a day is preferable, particularly if there be no distributing-reservoir.

Art. 105. Prevention of Deterioration or Destruction.

Watersheds should be preserved from erosion by the run-off, reliance being placed mainly upon vegetation. Streams and rivulets should not be permitted to erode their banks and beds; the use of loose-stone or brush dams to decrease velocity, and of facines or of slope-walls of field stone to protect the banks, being preventives which are generally applicable.

Slope-walls around the reservoir should be maintained intact, and no vegetation should be permitted to grow in their interstices. Roads and walks should be kept weeded and neat in appearance, nothing so inviting vandalism and petty damages as apparent neglect of appearances on the part of the authorities. All gates, valves, and other machinery should be tested frequently to insure their serviceability if suddenly called upon; and advantage should be taken of every opportunity to examine, lubricate, and otherwise maintain in the best condition all valves, screens, sluice-gates, etc.

The tops of no embankments should be permitted to settle below their designed elevation, but should be immediately raised to this with good material if any settlement occur. Low spots in the banks have caused the destruction of more than one reservoir by floods. The outer slopes of the banks should be kept sodded or paved to prevent rain from washing gullies in them. Gutters and drains on the berme should be kept open. Sodded slopes should be mowed occasionally to encourage the growth of a thick protective sod.

The available area of the spillway should never for a day be in any way diminished, unless temporarily when water in the reservoir is low. The channel below the spillway should be so confined and kept clear that no waste-water can wash the foot of an embankment or in any way endanger its safety.

ART. 106. INTAKES, WELLS, ETC.

Intakes generally require little attention beyond excluding or removing stoppages at their outer ends. Fish, sticks, and floating matters are occasionally caught in the screen, and require to be removed. But the greatest trouble is that experienced with anchor-, needle-, or slush-ice. This collects around some intakes to such an extent as to completely close them and stop all pumping, and is caused by needles of ice which are so tossed about by rough water as to prevent their cohering into a sheet. The motion of water towards the intake draws these needles with it, where they collect and freeze into a more or less solid mass. They never occur in still water, and seldom on a windward shore. The most feasible method of preventing their entering the intake is to make this opening so large that the velocity of flow towards it is not sufficient to draw the ice-needles from the surface. They have been prevented from entering the intake by use of warm air or steam, as described in Article 91. At a Chicago intake, during the winter of 1898–9, the interior of the intake-tower was heated by steam-radiators to prevent the formation of ice therein. If the intake be in a river-wall, or intake-tower rising above the water, needle-ice may often be prevented from entering the opening by fastening over it a raft floating upon the surface; the quiet water existing under this permitting the needles to freeze together and to the raft. It is thought this method was first used in Jersey City, where the raft was made of 12 × 12 timbers and was four times the length of the screen.

Wells seldom freeze, whether deep or shallow, the temperature of ground-water seldom falling below 40° to 45°. An exception occasionally occurs in the case of deep non-artesian wells, some of which "breathe" or take in air when the barometer is high and give it out when it is low (owing to the existence of large air-reservoirs in the soil); the inbreathing in cold weather resulting in freezing the pumps if these be above water, together with so much of the suction and delivery as are not under water.

The main difficulties experienced with wells are: leaking of air into the well or collecting-pipe; choking of the well with sand; and decreased yield, which is frequently due to one or both of the defects just mentioned, but in most cases to other wells tapping the same water-stratum, in which case the only remedy is greater suction or more wells, or both. If the remedy of new wells is adopted it is desirable to drive them to a new water-stratum if such can be found.

Air in wells or collecting-pipe and the remedy have been referred to in Art. 92. If the defect be in a well, it may be located by closing, one at a time, the valves between the wells and the collecting-pipe, and noting when air ceases to enter. If air still enters when all wells are shut off, the leak is apparently in the collecting-pipe, and this must be uncovered (if buried) and the leak stopped by repacking and tightening up joints if these be flanged, by recalking if of lead, or by calking between pipe and sleeve at each joint if these be threaded; the joints being afterward thoroughly coated with hot asphaltum.

If the leak be in a well it cannot generally be stopped, but a smaller pipe may be inserted reaching to the bottom of the well and connected at the top with the collecting-pipe, this connection replacing that with the defective well.

If the bottom of a tube-well become filled with sand, this may generally be removed by a "sand-bucket," a "sand-pump" (working somewhat on the principle of the steam-siphon), or by washing the sand up through the well to the surface by the use of the water-pipe of the jetting-down method of well-sinking. The latter plan also tends to clear the screen-openings of any material which may be choking them; but if the soil is very porous, it may fail to be effective because of the loss of water into the soil.

CHAPTER XIX.

PUMPING-PLANTS AND FILTERS.

ART. 107. PUMPING-PLANTS.

THE pumping-plant is the heart of a system, and this vital part should be put in charge of none but an experienced and faithful man. The pumps should be given the best of care, both for reasons of economy and because of the seriousness of a breakdown. There should be a reserve of pump, engine, and boiler power, which should be used occasionally to test its readiness for immediate service in case of an emergency. The lack of such reserve has often resulted in direct money loss, owing to the fact that the pump in daily use could not be put out of service for a few days to permit of a needed overhauling, and hence was wrecked when its usefulness might have been preserved for many years.

The "slip" in a pump increases with age, and, since it reduces the amount of water which the pump can lift, may become a serious matter. It is one, however, which it is not difficult to remedy, and the slip should not be permitted to become excessive.

Air from leaks in the suction-pipe causes a pump to "pound" and thus to gradually rack itself to pieces. This should be prevented by closing such leaks; or, if this is found to be impossible, by placing an air-drum on the suction-line and removing the air from this by means of a small vacuum-pump.

Sand entering a pump will cause it to cut its plunger and plunger-ring, and to increase its slip rapidly. If sand cannot be kept out of the suction, it should be intercepted before reaching the pump.

Maintaining the efficiency of a well-designed plant generally requires a careful, intelligent, skilful engineer, provided with the proper tools, good fuel, and water which deposits neither scale nor mud in the boilers. Opportunity should be given for overhauling each engine and boiler twice a year, either by excess of storage, or duplicate pumping-plant, or both. The overhauling can generally best be done in the spring and fall, when the consumption is lowest.

The main problem in connection with the pumping-station is the one of economy; to do the desired pumping at the least expense. In Table No. 72 it is seen that in the cities cited the salaries constituted from one to three fifths of the total cost of pumping, assuming the cost of coal at $2; and that the coal used amounted to from two to three fifths of the total cost, except in the cases of the high-duty pumps at Boston and Milwaukee, where it constituted 31 and 39 per cent respectively. At Newton, Mass., in 1896, the coal-bill was but 35 per cent of the total expense of pumping, although coal cost $4.18 per ton, the engines developing a duty of 111 million foot-pounds; while at Woonsocket, R. I., during the same year the cost of coal was 48 per cent of the total cost, the duty of the engine being 54 million gallons; the remaining expenses in both cities being almost exactly the same per million gallons raised one foot. The cost of oil and stores is very small; that of repairs also is small unless the engineer be careless or the pumps old and well worn; the number of men employed is practically fixed by the size of the plant; and coal-consumption offers the chief field for economy. This of course varies with the efficiency of the plant and of the men who run it, the quality

TABLE

COST OF PUMPING

Name of Station.	Total Coal used. 1 = 1000 lbs.	Total Pumpage. 1 = 1,000,000 Gallons.	Head Pumped against, Feet.
Chicago: North Side	43,707	22,544	108.6
West Side	32,489	19,271	104.2
Central	15,005	11,359	105.1
Lake View	9,297	4,600	108.0
14th Street	17,829	15,501	120.4
68th Street	22,595*	12,881	133.5
Whole city	140,920	86,156	113.0
Pittsburg: Brilliant Station	114,118	14,184	400.0
Philadelphia: Fairmount Station	....	9,912	100.0
Spring Garden	98,388	39,096	150.6
Belmont	27,584	6.361	220.4
Roxborough, Main	38,704	5,222	349.0
" , Auxiliary		16	83.2
Frankfort	16,278	4,107	209.9
Boston: Chestnut Hill †	4,210	3,511	126.7
Mystic	9,188	4,074	149.4
Louisville: Station No. 1, E engine		1,082	174.7
" " 1, W "		1,124	173.9
" " 1, W "		311	175.8
" " 1, N "		224	175.8
New Station, No. 3 engine		2,144	179.1
River-bank pumps		7	181.1
Combined operations	13,315	4,891	176.6
Cleveland: Division Street	62,316	15,390	192.6
Fairmount	2,223	639	162.4
Milwaukee: North Point	12,211	9,116	161.7
High-service engines	1,648	1,934	80.8
Combined operations	13,859	11,051	....

FROM REPORTS

Attleboro, Mass	456.9	115.6	175 and 188
Bay City, Mich	3,535.8	1,909.5	113
Newton, Mass	1,592.8	666.3	254
Oberlin, Ohio	290.0	24.6	80
Yonkers, N. Y	4,309.5	1,243.2	200 ±

A, Anthracite; B, Bituminous. * Coal equivalent to oil burned; 1 gallon of oil assumed as equal to 10.435 lbs. of coal, corresponding to 14 lbs. evaporation for oil, and 8.75 lbs for coal;

No. 72.

IN TWELVE CITIES IN 1893.

Duty. 1 = 1,000,000 Foot-pounds.	Coal used per Million Gallons per Foot Head, Pounds.	Average Price of Coal used, per Ton.	Comparison of Cost, in Cents, of Pumpage per Million Gallons per Foot Head, on Uniform Basis: Head = 200 feet; Coal at $2 per ton.				
			Salaries.	Oil and Stores.	Maintenance and Repairs.	Coal.	Total Cost.
46.7	17.840	$5.69A, 3.07B	1.106	0.058	0.150	1.784	3.098
51.5	16.180	2.82	1.208	0.048	0.192	1.618	3.066
64.4	12.560	3.10	1.464	0.058	0.092	1.256	2.870
44.6	18.720	3.15	2.500	0.170	....	1.872	4.542
87.3	9.560	5.40A, 3.12B	1.712	0.102	0.154	0.956	2.924
63.5	13.140	....	1.708	0.086	0.066	1.314	3.174
57.6	14.480	....	1.467	0.075	0.132	1.448	3.122
41.5	20.114	1.12	1.048	0.134	0.340	2.011	3.533
....	Water Power	....	0.411	0.025	0.286	....	....
49.9	16.710	1.73	0.655	0.051	0.577	1.671	2.954
42.4	19.676	1.73	1.227	0.080	0.577	1.968	3.852
39.3	21.222	2.04	1.300	0.203	0.853	2.122	4.478
44.2	18.880	1.74	1.226	0.154	1.088	1.888	4.356
88.1	9.464	4.75 to 5.30 B	1.673	0.097	0.283	0.946	3.000
55.2	15.097	4.12 to 4.45	1.346	0.106	0.442	1.510	3.404
48.4	....	....	....	....	....	....	....
48.0	....	....	....	....	....	....	....
24.3	....	....	....	....	....	....	....
25.2	....	....	....	....	....	....	....
92.6	....	....	....	....	....	....	....
26.8	....	....	....	....	....	....	....
54.1	15.418	2.42	1.626‡	0.147	0.636‡	1.542	3.951‡
39.7	21.018	1.42	1.329	0.104	0.281	2.102	3.816
38.9	21.422	1.19±	3.217	0.269	0.378	2.142	6.006
100.6	8.220	5.45A	1.121	0.098	0.082	0.822	2.123
79.1	10.544	5.45A	2.246	0.393	0.065	1.054	3.758
98.1	8.50	5.45	1.318	0.150	0.079	0.850	2.397

FOR 1896.

			Cost per Million Gallons Raised 1 Foot High.	
			Station Expenses.	Total Maintenance.
39.75		4.60	$0.0996	$0.765
....		4.77	.064	0.393
111.00		4.18	.05	0.63
5.66		2.25	.61	2.38
....		4.13	.055±	....

oil at 6¼ cts. per gallon. † Evaporation equals 11.63. ‡ Above the usual cost. The total should be about 3.23, according to Mr. Charles Hermany, Supt.

of the coal, the amount pumped, and its relation to the capacity of the pumps and boilers.

The cost of pumping in seven large cities in 1893 is given in Table No. 72 (by S. G. Artingstall, before the Am. W. W. Ass'n, May 1895).

For smaller plants the cost is generally greater. At Newton, Mass., it was 4.9 cents per million gallons raised one foot, in 1894; 3.9 cents in 1895, 5 cents in 1896, and 5 cents in 1897. At Taunton, Mass., it was 10.44 cents in 1893, 15.5 cents in 1895, and 15.67 cents in 1896. At Montreal it has varied between 37.6 and 7.8 cents in fourteen years, averaging 19.5 cents. The following shows the cost of raising 1,000,000 gallons one foot high in several New England systems.

	Attleboro.	Brockton.	Lynn.	Middleboro.	New Bedford.	Newton.	Plymouth.	Reading.	Taunton.	Waltham.	Woonsocket.
1895	9.27	17.2	3.66	17.4	5.30	3.9	22.3	14.7	15.5	10.5	6.3
1896	9.96	11.8	3.99	16.0	4.29	5.0	23.2	18.1	15.7	6.3	6.2

The plan of offering a bonus to the fireman and engineer for every million foot-pounds by which the duty exceeds a certain amount has much to recommend it, since their carefulness and efficiency form a large factor of the duty obtained from a given plant.

ART. 108. OPERATING FILTERS.

The maintenance of mechanical filters is a very simple matter when the water is of uniform quality—as ground-water generally is; requiring only that for a given quantity of water, as registered by a meter or by the engine-counter, a fixed amount of chemical be added, which is generally performed automatically; and that after a certain amount has passed the filter this should be cleaned by opening and closing a few valves.

If the water be from a river, however, the condition is different, and care must be taken to so proportion the chemicals to the amount treated that the desired purification is obtained, but without waste of chemicals or permitting these to enter the effluent in injurious quantities. For this purpose it is desirable to make a chemical analysis of the water at least once a day, and more frequently if changes in it be rapid. If the water carry any considerable amount of matter in suspension, the test for this (by platinum wire or otherwise) should be made at least twice a day, and the coagulant regulated accordingly. The daily chemical test of the crude water should be accompanied by one of the effluent. Once or twice a month tests should be made of samples taken hourly throughout the day, as the river-water may be affected by manufacturing wastes discharged at certain hours of the day, or by other periodic pollution.

At the beginning of the operation of a filter-plant it is of course necessary to make continuous tests of both crude and filtered water, and meantime to vary the amount of coagulant until it has been learned what amount gives the best result with each condition of water. From these experiments a table can be prepared showing at a glance what amount of coagulant is desirable if the chemical analysis be known. If at any time the daily or hourly tests indicate that the desired purification is not being attained, this study of relations between condition and treatment should be repeated.

Many filtration-plants are run without such analyses being made, the only test being the appearance and taste of the water. But such a lack of method is almost sure to result in low efficiency, with probably as great or even greater expense for coagulant than is necessary for good results; and justice to the consumers demands better water than they receive from plants so run.

The cost of running a mechanical-filter plant will be that of chemicals and of labor. Alum suitable for use as a coagulant costs about $27 per ton. The labor for a small plant—500,000 to 750,000 gals. per day—can generally be performed by the pumping engineer. For larger plants probably one man for every three to five million gallons will be necessary. The life of a plant cannot be stated, as the modern mechanical filter is but a very few years old.

The maintenance of an English filter includes regulation of the rate of filtration, cleaning of the beds, washing and replacing the sand; and removing ice if the filter is uncovered. The basin of the filter should need no repairs or attention; but the sand will need to be renewed from time to time. Analyses should be made of the crude and filtered water, as in the case of mechanical filters, and these should be taken daily for each bed. If the efficiency from any bed becomes low, the rate of filtration should be reduced, or the bed examined for "blow-outs," or holes in the sand permitting the free passage of water. Water sometimes follows the walls down, and reaches the drains without having been filtered. This should be prevented by making the walls as rough as possible, by building horizontal ledges in them a short distance above the bottom, or by banking the sand against them all around.

The beds are generally cleaned with wide, square-pointed shovels, and but one-quarter inch to two inches of sand removed at a time. The men should not stand behind the shovels, but at one side, and should not walk upon any part of the bed which has been cleaned. If, however, the sand has become considerably clogged by long use, this precaution is not necessary, as it should be worked over with a garden-fork after cleaning, and levelled off with a rake. This is made necessary by the fact that in time a gelatinous film and fine silt work their way through the entire bed, and bind the sand particles together. When this clogging reaches within 6 or 8 inches of the bottom of the sand, the whole of this should be removed and washed.

If there is any ice upon the water in the filter before cleaning, this should be removed. During use, if water be kept continually a foot or more deep above the sand, the ice is not particularly objectionable if it do not injure the walls or banks of the filter-basin or other structures. After the ice, if any, is removed the water is drawn off of the surface of the filter (means should be provided for this), and that in the filter is drained out. Scraping is then completed as rapidly as possible and the filter refilled and put into service again. The refilling is generally done from the bottom, but in some cases from the top; the objection to the latter method being the washing of the surface caused by water flowing over the uncovered sand. When filled from the bottom, filtered water should be used and the sand covered with this 6 inches or more deep.

Cleaning should be done in fair weather, neither freezing nor very hot; and the sand should be moist, but not wet. If the sand freeze, it is of course practically impossible to remove the desired thin layer. If the sun be too hot, the gelatinous matter and silt bake into a cake, nearly as difficult as ice of treatment. Rain falling upon a cleaned surface will so compact and wash it as to make it necessary to fork and rake it before a desirable rate of flow is obtained. The disadvantages of ice and rain are avoided by covering the filter.

The sand removed is washed or thrown away, whichever is cheaper. After being washed, it is generally stored until the thickness of the sand-bed has been reduced to its minimum limit, when all the sand is replaced upon the bed. After some years of use the sand grains seem to become rounded and the rate of filtration possible decreases, until it becomes necessary to obtain new sand throughout.

Washing is done in a number of ways: by piling up the sand and simply turning a hose upon it; by washing it through a sluice having strips nailed across the bottom, the sand collecting behind the strips while the water carries the silt past them; by revolving cylinders of different kinds in which the sand is placed and through which water is passed; by hoppers, either single or in series, the sand falling into the hopper from above, while a stream or jet of water enters it from the smaller bottom opening, the water overflowing the hopper and carrying the silt with it, while the sand falls into another hopper or is piled up for future use.

A bed is cleaned whenever the maximum head possible or desirable fails to force through it the desired amount of water. This may vary from three or four days to as many months in the same plant; depending upon the amount of clay or loam carried by the river and the length of time the sand has been in use. If cleaning is required oftener than once in three or four weeks with clear water, the efficiency of the plant as well as economy would require an entire renewal or cleaning of the sand. If the muddiness of the river compels cleaning oftener than this, sedimentation-basins should be provided; unless such a condition lasts for but two or three weeks in the year, and even then they are desirable.

At Poughkeepsie one man cleans about 150 square feet of filter per hour, including removing the sand from the bed, and forks and rakes 500 square feet. Washing the sand by a double hopper costs about 24 cents per cubic yard, the water used being about eighteen times the amount of sand washed. Hazen gives the average cost of washing sand by machinery as 30 to 40 cents per cubic yard; and 140 to 170 square feet per hour as the amount of filter one man can scrape. At Lawrence, Mass., 400 cubic feet of water are used for washing a cubic yard of sand, or about 15 to 1. The cost of removing ice from this filter during 1897 was $2323, or $930 per acre. The entire cost of maintenance at Lawrence was $10 per million gallons filtered; at Poughkeepsie it was a little less than $3. At Ilion, N. Y., the cost of cleaning is about $0.82, and at Hudson $0.88, per million gallons. These figures cannot be compared, however, since some include all running expenses and probably interest on the cost of the plant also. It is thought that $10 per million gallons filtered should be an ample allowance for one million gallons or more per day; this sum including interest on the cost of the filter-beds and sedimentation-basins, as well as of daily chemical analyses and proper and intelligent supervision.

If no sedimentation-basins or filters are provided for a river-supply, the manager of the water-works should use foresight and ingenuity to exclude muddy water from the distribution system. The reservoirs should be kept filled while the water is clear, and pumping discontinued while the river carries much silt. Two reservoirs should always be provided; or one made into two by a dividing wall. While the river-water is roily the supply should be drawn from one basin only, and when this is empty it may be filled with roily water, which is allowed to clarify itself by sedimentation while the supply is drawn from the other basin. The pumps should be at least capable of supplying the entire maximum daily consumption in twelve hours; then by working 24 hours per day in filling a reservoir with muddy water, one half or more of the time occupied in emptying the other reservoir can be given to sedimentation. Water can be greatly improved by sedimentation in this way, but filtration is much to be preferred, and should be obtained if possible.

CHAPTER XX.

PIPES AND CONDUITS.

ART. 109. MAINTAINING QUALITY OF WATER.

AFTER entering the conduits water often undergoes changes in chemical composition, and generally in temperature. The latter is more uniform at the taps than in the reservoirs. Fig. 91 shows the mean temperature for four

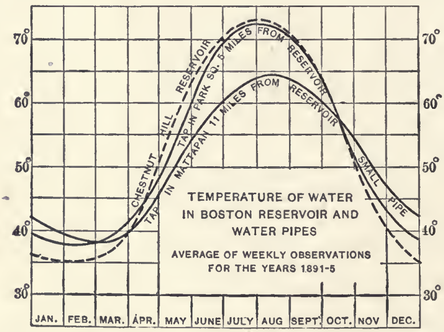

FIG. 91.—TEMPERATURE OF WATER IN PIPES.
(From paper by Geo. C. Whipple, before N. E. W. W. Ass'n.)

years in a Boston reservoir and the mains fed by it, Park Square being five miles from the reservoir, and Mattapan eleven miles.

If the water in the mains contain organic matter, the decomposition of this probably will continue to a certain extent; but the chief chemical changes will be those due to the taking into solution of zinc and lead from service-pipes, as already referred to. Before a given supply is used tests should be made to ascertain whether these metals are dissolved by it, and if they are, the use of pipes containing them should not be permitted in connection with the system.

The chemical changes in organic matter tend to purify the water, and for this reason a purer water is frequently found at a distance from than in the reservoirs. It is generally found also that water is purer in the high than in the lower parts of the city. In Boston, for five years from 1891–95, the following number of standard units of organic and amorphous matter per cubic centimeter was found at the points stated:

	Number of Standard Units per cc.	
	Organisms.	Amorphous Matter.
Chestnut Hill Reservoir, gate-house	248	209
Brookline " "	215	212
Tap in Park Square	194	190
" ' Mattapan	84	105

The reduction seems to occur in the smaller pipes. From November to April the reduction in the pipes was 44 and 24 per cent respectively; and from May to October it was 62 and 53 per cent. Bacteria decreased from 0 to 60 per cent between August and April, and increased 0 to 60 per cent between April and August, when the most organisms are dying in the mains. Some organisms settle out, some break up, die in the dark, and are devoured by sponges. A 16-inch main in Boston was found to contain a lining of sponges and polyzoa $\frac{1}{4}$ inch thick, as well as snails, mussels, and countless infusoria. The animal organisms depend upon algæ and infusoria for food. Chicago was last year troubled with countless numbers of snails in the intake and mains. Probably all surface-water systems contain such organisms in their pipes, but so long as the animal organisms are sufficient in number to destroy all vegetable and dead animal matter no harm need result. It is thought that these organisms are not found in ground-water systems.

The life of the animal scavengers in a system seems to require a circulation of water past them, and in dead-ends amorphous matter generally accumulates and contaminates the water. To prevent this, water should be frequently drawn out of all dead-ends, thus providing circulation and also removing putrefying matter. Deposits of mineral sediment also collect in dead-ends and should be flushed out. It should be the duty of some one employé to flush out each dead-end at stated intervals, and keep a record of the date each one is flushed.

But little sand or gravel should find entrance to an irrigation-conduit, the head-gates being utilized to exclude it (see Art. 103). Some, however, may enter during floods, and open conduits often receive considerable sand blown in by the wind during the sand-storms of our Western plains. This sand is generally intercepted by a sand-box or sand-trap, which should be inspected daily, and flushed out whenever the sand rises within a foot of the conduit.

ART. 110. MAINTAINING QUANTITY OF WATER.

It is the function of a conduit and distribution system to distribute to consumers through regular appurtenances all the water which enters their furthest end, and to deliver it at the desired rate. This is interfered with by any leaks, breaks, or other losses; or by contraction of the channel due to deposits or other accidental stoppages, by the closing of gates, or by reduction of pressure in the pipes. Under this head are included, therefore, the detecting and repairing of leaks and breaks; the removal of deposits and accretions; thawing of frozen pipes; the regulation of gates, pressure-regulators, service-cocks, and fire-hydrants, etc.; and incidentally regulation of the consumption.

In clay or compact loam, leaks in water-pipes will generally make themselves evident at the surface immediately above them, unless this be paved with water-tight material, such as asphalt or any pavement on a concrete foundation. In sand or gravel, however, the existence of a leak may be unsuspected for years, and its exact location be almost impossible of determination when its existence is known. In paved streets, also, a leak makes itself known in adjacent cellars or basements, if at all; but the building in which it appears may be at a considerable distance from the leak. When the ground-surface is frozen for some depth the effect may be similar to that of a pavement. On steep hills, water from a leak frequently follows the pipe for some distance, appearing at the surface where a flatter grade begins.

Where the appearance of water at the surface indicates a leak which is found or thought to be at some distance from the point of emergence, its location can be more exactly determined by so closing valves as to shut off the pressure from different sections successively, each section being made as short as the location of the valves will permit. The evidence of leakage will generally decrease soon after the right section is cut out. An examination of the valve-boxes will often be sufficient to determine the location, water standing in these being evidence of its existing in the ground at that point, and of the leak being up-grade from there. An examination of fire-hydrants also should precede more extensive investigation, the top of each being removed to discover whether water is not entering the barrel from a partly open valve and escaping through the drip into the ground. Where the ground is frozen or sealed with paving, openings cut through this at intervals directly above the pipe-line will often permit the nearness of a leak to become

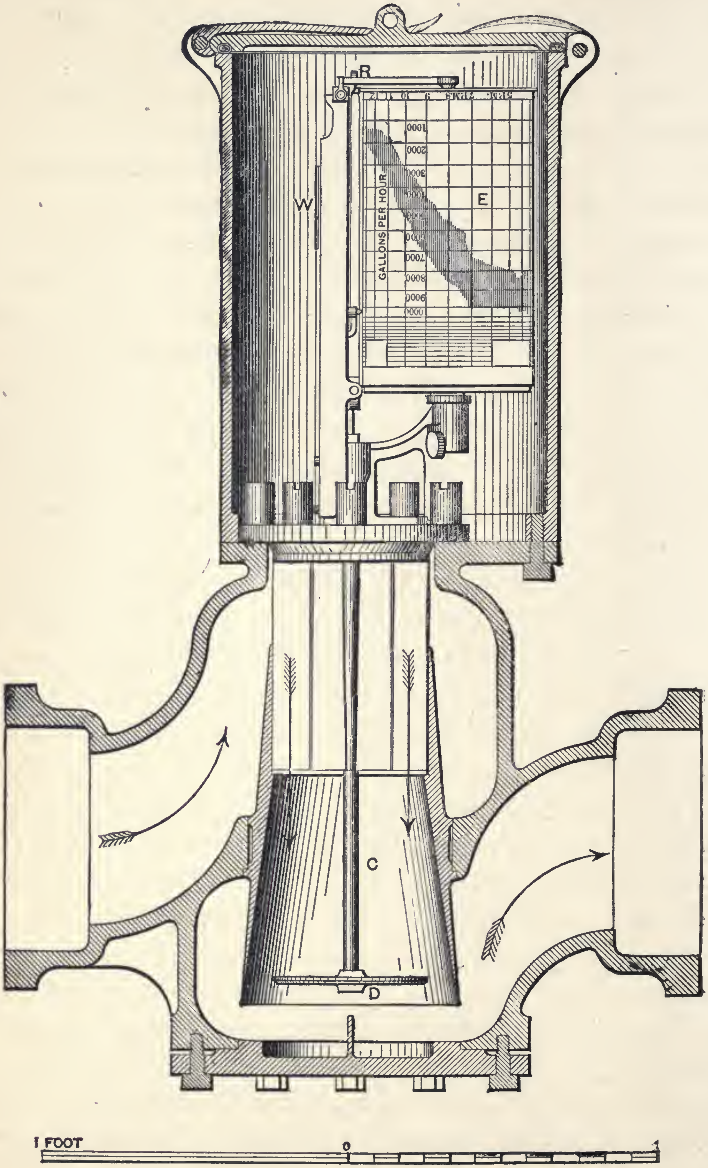

FIG. 92.—DEACON WASTE-WATER METER.
(From Trans. Am. Soc. C. E., Vol. XXXIV).

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evident. If valves are closed in making the investigation, it is of course best to make this at night, when the consumers will be least inconvenienced. If the leakage is considerable, it can often be detected by the "singing" of water passing a valve which is almost closed, all other valves and service-cocks connected with the section being closed. Or, in place of this method, a Deacon Waste-water Meter (Fig. 92) may be connected to the pipe at both sides of a closed valve, all valves and service-cocks being closed, and any leakage will be registered by the meter. This meter has been used to a considerable extent in England. and in Boston, Philadelphia, and probably some other cities in this country, for discovering waste of water by consumers; for this purpose all service-cocks being open. But it is desirable to first use the meter with these all closed, as otherwise waste might be looked for when leakage should be the object of the search. When leakage is in sand or gravel about the only method of locating it is to test the entire piping system section by section, either by the use of the Deacon meter or by noting the result as each section is cut out.

Leakage having been approximately located, the pipe must be uncovered and the leak stopped. If it is found to be at a lead joint, calking is the general remedy. If at the joint between a corporation cock and the main, it may sometimes be stopped by calking the threads of the cock against the tap-hole; but it is better to disconnect the service-pipe, the corporation and service cocks being first closed, and screw up the corporation to a tighter joint. If the leakage be at a valve, the packing probably needs renewing or the bolts tightening up. If the leak be caused by a crack in the pipe, the damaged pipe should be broken out and a new one put in its place. If the crack be near one end, it may be well to cut the pipe in two well beyond the end of the crack and remove only the broken section. For cutting pipes in the trench special hand-machines are obtainable (see Fig. 93), the use of chisel and hammer for this purpose being an exceedingly slow method, likely to result in failure in many cases. If the crack is very short, or the leakage be through a blow-hole in the iron, it may be repaired by fastening over it a sleeve, made in two halves bolted together and surrounding the pipe, lead being calked between the pipe and each end of the

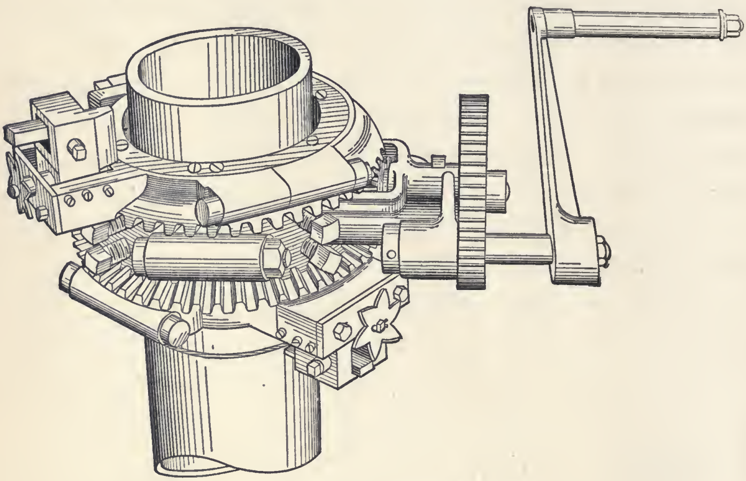

FIG. 93.—PIPE-CUTTING MACHINE.

sleeve. The use of a sleeve is necessary also when inserting a new section of large pipe to replace one removed; an alternative method when a whole length of small pipe is replaced being to melt out the next joint each way from the break, raise the free ends of the two end pipes, enter the joints at each end of the new pipe while all are suspended in the air, and let all three drop to place in the bottom of the trench.

A joint in which there is a bead on the spigot-pipe cannot be pulled apart, and it is very difficult to cut the lead out. In melting it out a hole is generally dug under the joint and a wood-fire started there. In some cases, especially where there is water in the trench, this is impossible; but if the top half is melted out, the joint can generally be loosened and pulled apart. A better plan has been used by S. W. Frescoln and is described by him in *Engineering News* of Sept. 9, 1897. He used a Wells light rigged for the purpose. "The vertical standpipe was taken off, and by means of a sufficient length of flexible rubber hose and fittings two burners were attached to the cylinder containing the oil, so that the cylinder could be placed upon the pipe and a burner be operated upon two joints at once. The constant services of one man were required to operate the burners and compress the air in the oil-cylinder. By means of suitable tongs and rigging made from ordinary stiff wire, the flame from each burner, about two feet long ordinarily, under a blast of 30 lbs. was directed at short range exactly against the lead joint at the top of the bell. . . . The average speed made was from 8 to 10 joints per day per man, and 1 bbl. of oil sufficed for about 12 joints."

If the leak is found to be at the valve of a fire-hydrant, it will most frequently be due to a stick, stone, or other matter on the seat which prevents the valve from closing entirely. This matter may often be washed out by simply opening the hydrant. If it is too large to pass the nozzle, remove the cap of the hydrant and let the water come directly out of the top of the barrel. Stones weighing several pounds can be thrown out in this way. If this will not remove the obstruction, a boat-hook, a long pair of tongs, or other contrivance may serve the purpose; or as a last resort it may be necessary to dig around the hydrant, remove the barrel, and thus get at the obstruction.

If the valve of the fire-hydrant is cut or worn, the stem sprung, thread cut, or other damage done, the hydrant or damaged part should be removed for repairs and immediately replaced with another.

Breaks of considerable magnitude call for prompt action to avert great damage to adjacent property. The section in which the break occurs should be at once cut off from the systems by closing the necessary valves, and the ability to immediately locate these in such an emergency is the chief advantage of the methodical placing of valves as described in Art. 94.

Contraction of the bore of a pipe generally results either from deposits of inorganic matter in the pipe, from the growth of organic matter upon its inner surface, or from tubercles. The first should be prevented to a great extent by purifying the water before admitting it to the pipes; but a deposit, when once in them, is generally removed by discharging water in as great volumes as possible from fire-hydrants and from blow-offs at depressions in the line; the increased velocity which results stirring up the sediment and removing it. In some cases the deposit is so compact as to require other methods for its removal. Scrapers of different designs have been used for this purpose. Some are dragged through by a rope, which has first been passed through a section of pipe by making an opening at each end of this and floating a string through it. The scraper generally consists of a number of stiff steel arms fastened to a stout centre, the arms being designed to scrape against the surface of the pipe with some force. While dragging the scrapers, water must be forced through the pipe to wash out the material loosened up and prevent its choking the pipe in front of the scraper. In using these scrapers it has not been found practicable to clean more than 1000 feet of pipe at a time, and consequently breaks in the pipe-line must be made at intervals of not more than this distance. It also is not found possible to drag the scraper around a bend of more than 20° or 30°, and a break must be made wherever a greater angle exists. These breaks must of course all be repaired by new pipe. In making the breaks the cutting-machine already referred to will be found

Fig. 94.—Pipe-scraper.

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very useful. In St. John, N. B., special hatch-boxes are provided to permit of introducing the scraper without breaking the pipe.

In place of dragging a scraper through the pipe certain contrivances have been employed which are forced through by pressure of the water behind them. One such is shown in Fig. 94, being one used in 24-inch pipe in St. John, N. B., in 1898 by Mr. William Murdoch, and described by him in the Journal of the N. E. W. W. Ass'n for June 1899. The force used to operate this was one foreman, one mechanic, two watchers, six assistants, and two express teams and drivers. "The watchers walked the line on the cleaner being started, and had no difficulty in following the sound outside the city, but on coming inside city limits the noise of the traffic drowned the sound of the cleaner, and all that remained was to watch and wait for its arrival at the terminal hatch-box. . . . The cost of the work was as follows:

Furnishing and placing in position nine 24-inch and four 12-inch hatch boxes................	$3,468.95
Cost of one 24-inch cleaner....................	40.00
Total cost of operating cleaner.................	274.42

"The operating expenses of cleaning the main, which measured 4.3 miles, was $63.84 per mile."

The method of cleaning distribution-pipes in Boston is described by Mr. Dexter Brackett in the same number of the Journal of the N. E. W. W. Ass'n, as follows: "As many of the service stop-cocks project into the pipes from $\frac{1}{2}$ to $\frac{3}{4}$ inch, it was necessary to have the scraper arranged so as to pass by such obstructions, and at the same time remove the coating of tubercles. The machine, which was very successfully used, consisted of a flexible central shaft about three and one half feet in length, composed of coiled steel springs connecting small castings, to which were hinged two sets of steel scrapers, arranged radially around the shaft about twelve inches apart. The scrapers were kept against the sides of the pipe by coiled springs, which permitted them to turn back so as to pass taps or other obstacles. Back of the scrapers were two rubber pistons placed about two feet apart so as to insure a pressure on the machine when passing the branches. . . . A section was cut out of the pipe long enough to receive the scraper, which was then inserted, and the joints made with lead in the ordinary manner, except that clamp-sleeves were used so that the section could be again easily removed and the scraper inserted if desired. A similar piece was cut from the pipe at the other end of the main to be cleaned, and the scraper was forced through the pipe by the ordinary water-pressure, which varied from 30 to 45 lbs.

"As occupants of buildings on the lines of the pipes were without water while the work of cleaning was in progress, and as it was not thought advisable to pass the scraper through the valves, the pipes were cleaned in lengths averaging 1000 feet. The scraper generally passed through this distance in from three to four minutes, or about as fast as a man could walk. In a few instances the scraper was stopped by obstructions in the pipe, the one causing the most trouble being lead which had run into the pipe at a joint. . . . Some difficulty was experienced from the stopping of service-pipes and house-plumbing by rust forced into the pipes by the pressure of the water following the scraper, but this difficulty could be generally overcome by applying a force-pump to the house-plumbing and forcing the obstructions back into the main.

"By this method the tubercles were removed from 58,000 feet of 6-inch pipe at a cost of 14 cents per foot, and from 20,300 feet of 12-inch pipe at a cost of 20.6 cents per foot. These prices include 5 cents per foot royalty paid for the right to use the scraper. . . . The discharging capacity of the pipe was more than doubled by the removal of the tubercles."

Tubercles are the obstructions most commonly found and removed by scrapers. These are cone-shaped projections from $\frac{1}{8}$ to $1\frac{1}{4}$ inches in height, generally hollow, and easily broken by the finger-nail. They may be scattered at intervals of several inches over the interior of the pipe, or may be so numerous as to form a continuous rough surface. They are not formed by some waters, and only on pipes which are not properly coated; although in pipes which are apparently well coated they may form where minute surfaces of iron are in contact with the water. In cleaning the Sudbury Aqueduct (Boston) after it had been sixteen years in use about two cubic yards of tubercles were removed from 22,619 square feet of surface; and from some pipes a cubic yard of tubercles has been removed for each 1000 square feet of surface.

Mains which are too near the surface sometimes freeze, and this prevents the flow, if it does not burst the pipe. Probably only the smallest mains—4- and 6-inch—ever freeze shut without bursting. Most trouble of this kind is in connection with the service-pipes. Several methods have been employed for thawing pipes. Steam is forced from a portable boiler through a steam-hose and nozzle, and by directing the steam against the ground a hole is bored as by a water-jet, the nozzle following down, until the pipe is reached and thawed by the steam. Or a fire is built on the surface over the pipe. Or the pipe is uncovered and thawed out with hot water. Probably the best method yet contrived was first used, so far as is known, in the early part of 1899, originating with members of the faculty of the University of Wisconsin. It consists of heating the pipes by passing electricity through them, the current being conveniently taken from the electric-light wires. In Ithaca, N. Y., a 4-inch cast-iron main over

100 feet long was thawed by use of a current of 160 amperes in 5 hours and 40 minutes, the pressure in a 2-inch copper wire being 9 volts. At Watertown, Wis., 320 feet of 6-inch pipe was thawed by a current of 350 amperes at 100 volts in about two hours. To answer numerous inquiries the University of Wisconsin issued the following directions:

"The current which is required for satisfactorily thawing service-pipes up to 1½ inches in diameter is from 200 to 300 amperes. The source of current should have a pressure of not less than 50 volts. Where electric-light lines carrying alternating currents are available a transformer or transformers in parallel may be used as a source of current. It is very important that direct connection of pipes to house-lines be avoided on account of danger of fire, in which the house is placed by such connection. Where alternating currents are not available continuous-current feeder-lines may be used, but these should be entirely separated from the distributing network of conductors.

"The accompanying sketch will show the way in which the appliances should be connected when an alternating

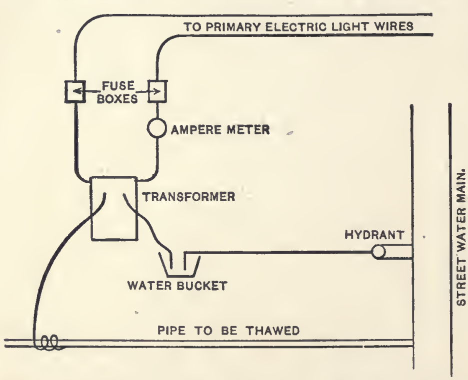

FIG. 95.—THAWING PIPES BY ELECTRICITY.

current is used with transformer. The secondary leads from the transformer should be quite large, such as No. 3 B. & S. gauge, or larger. In making connection to the pipes, one of the secondary leads should be taken into the house to which the frozen service-pipe leads, and contact made at that point by some form of metallic clamp or by simply giving the conductor two or three tight twists about the pipe at any point where the pipe is exposed or at a faucet in the house. The other secondary lead should be put in contact with the water system outside of the house, and in a similar manner. This contact may be made at a hydrant or at an adjoining service-box, or pipes in a neighboring house. When there are two houses near together, each with frozen service-pipes, the two secondary leads may be connected to the pipes within these houses and both frozen service-pipes thawed out at once.

"While the thawing process is going on, the faucet should be open in the house to which the service-pipe leads. In one of the secondary leads should be inserted a water-resistance which consists, for convenience, of a bucket of water containing a bowlful of salt, and two sheet-iron or copper plates, to which the ends of the severed lead are attached. This serves to control the current. In the primary leads from the electric-light line to the transformer it is highly desirable to have a fuse in each lead, and an ampere-meter. When all connections are made, the plates are placed in the bucket and are then moved towards each other until the ampere-meter records a proper current. If the primary pressure is 1000 volts and the secondary pressure 50 volts, the current should ordinarily approach 15 amperes. If the primary pressure is 2000 volts and the secondary pressure 50 volts, the ampere-meter reading should ordinarily approach 7.5 amperes.

"Water ordinarily begins to flow in a time not much less than 10 minutes or not greater than one hour. If the secondary current is quite close to 300 amperes, the period seldom exceeds one-half hour. The pipes are often split by the action of the frozen water, and these at once begin to leak when the ice is thawed away. For this reason it is desirable to have a plumber where he may be readily called to care for the leaky pipe.

"The electric current when properly used will not damage the pipes. It is desirable to watch brass and iron connections to lead or iron service-pipes, as they sometimes heat on account of poor contact. If such heating appears to be excessive, the current may be reduced with a resulting increase in the duration of time for thawing.

"After the pipe has been thawed it is desirable to let the water run continuously for a considerable time, inasmuch as the ground all around the pipe is frozen and the pipe is liable to freeze again unless water circulates."

Fire-hydrants sometimes freeze, generally because of a leak in the valve. They can be thawed by the use of steam from a portable boiler; but a more convenient plan is by the use of hot air from an oil-stove, which is led to the inside of the hydrant by a hose, through which motion of the air is caused by use of a small hand-blower. Fire around the hydrant, burning waste inside of it, and similar methods are not recommended, as their use is liable to crack the cast iron, cause the valve-stem to bend out of line, and produce other injury to the hydrant.

An inspection of all valves and curb-cocks should be made occasionally to ascertain whether they are in working order, and are closed or open, whichever is desired. Circulation through the distribution system is sometimes interfered with by a closed valve, which thus creates two dead-ends. Detroit, Mich., has a special gang to look after the 6000 valves in the system; by which, in 1895, 671 valves were found to be out of order, and 94 were found shut; 237 valve-boxes or manholes were repaired, and 147 cleaned.

Waste of water can be prevented only by the greatest vigilance if meters be not used. A house-to-house inspection should be made at least twice a year, every appliance for using water be examined, and the service-pipes be inspected so far as they are exposed to detect the existence of any unauthorized fixtures. The most common locations of leaks and waste are named in Art. 14. Also a block-to-block test of the mains should be made between 1 and 5 A.M., when it is assumed that no water is being used, and if any flow be found (by methods described in the first part of this article), its outlet and cause should be investigated. The use of meters is preferable to inspection, however, being more certain in its results. Each meter should be tested for accuracy before being set, and in some cities each meter is tested once in one or two years, and whenever its accuracy is impugned by a complaining citizen. Sensitiveness also is desirable in a meter, that it may register small wastes as well as large. If a large number of meters are in use, a mechanic should be employed who is able to make necessary repairs upon these; and he will also find additional occupation in connection with fire-hydrants, valves, curb-cocks, etc. Repairs to meters are required generally because of freezing, of stoppage caused by a stick or other foreign substance in the water, of breaking of the diaphragm or piston (unusual in a good meter), or of wear in the recording mechanism. One or two designs of meters are now provided with frost-bottoms, which are claimed to prevent damage to the meter if it freezes. In most meters the recording mechanism can be removed and replaced with a new one without disturbing the meter or its connections. The maintenance of meters, including reading the dials quarterly, should not cost more than \$.40 to \$1.50 each per annum. In Fitchburg, Mass., in 1898 maintenance of 2177 meters cost an average of 65 cents each. The cost of keeping in repair 582 meters in Reading, Pa., during the year of 1898–9 was \$62, or $10\frac{2}{3}$

cents each. In 1892 repairs in Waltham, Mass., cost 13 cents per meter; in Springfield, Mass., 13 cents per meter; in Detroit, Mich., 12 cents; and in Binghamton, N. Y., 36 cents.

Art. 111. Service Connections and Extensions.

The connection between the main and a house is generally called the "service connection." The service-pipe is ordinarily from $\frac{1}{2}$ to $1\frac{1}{2}$ inches diameter, $\frac{5}{8}$ or $\frac{3}{4}$ being a common size for dwellings. Galvanized iron, lead, cement-lined, tin-lined, lead-lined, and black iron pipe are all in common use. The use of black iron should be prohibited in the majority of systems, if not in all, since it is subject to tuberculation and rust, which decrease the capacity of the pipe and often close it altogether. Tin-lined pipe is an excellent pipe if well made, since few if any potable waters have any effect upon tin. Cement-lined pipe gives good service under the same conditions. But if the cement become cracked by rough handling or freezing, or is of poor quality, it flakes off and not only leaves the pipe unprotected, but frequently clogs it or the meter or plumbing fixtures. Galvanizing on iron seems to be less subject to flaking and peeling in actual service than do cement and tin as generally applied, is cheaper, and is in much more common use. A disadvantage of all linings is that the iron is exposed at joints or wherever the pipe has been cut. Lead-lining has been used which can be hammered and pressed around exposed edges to protect them, but this pipe is rather expensive. The chief objection to lead pipe is its low tensile strength, its cost, and the danger of the metal being taken up by the water and poisoning consumers (see Art. 6). All of these objections are real ones; the last, however, only when waters with certain characteristics are used.

The service connection consists of a "corporation cock," which is a stop-cock attached to the main and generally made to open and close with a socket wrench; a "goose-neck" formed of about three feet of lead pipe bent to the shape of an inverted U, forming a flexible connection between the service-pipe and the cock in order that settlement of the former may not break the latter (this is sometimes omitted—unwisely, the author thinks); a length of pipe leading to the curb, where a "curb-cock" or valve is placed and provided with a valve-box, permitting the water to be turned on or off by use of a suitable key; and from this a pipe leading to the house-plumbing.

The corporation cock is made of brass throughout. Those used some years ago were tapering and were driven into a tapering hole in the main, where they were held by friction. In a few places these are still used; but those now in common use are screwed into the main, a hole having been drilled in this and threaded. When the main is empty the operation is very simple; but if the system is in service the necessity for cutting off a section from use by closing all valves and emptying the pipe of water is a serious objection. To avoid this, several machines have been designed which will drill and thread the hole in the main and insert the cock without the loss of more than a quart of water, and without shutting off the pressure. Among these are the Mueller, Lennox, Smith, Hall, and other tapping machines. The cock is closed when attached to the pipe, and is not opened (except for an instant to test it) until the service connection is completed.

The goose-neck may be either soldered to the corporation cock, a "wiped joint" being made, or expanded into a union designed for this purpose and screwed to the corporation cock. If no goose-neck is used, the pipe is screwed directly onto the end of the corporation cock.

The curb-cock is somewhat similar to the corporation. It is turned by a key, and is provided with a "curb-box" extending above it to the surface, by which it is made accessible. The curb-box should be adjustable in length when in place and without digging around it, unless to be placed in a concrete or other permanent sidewalk.

It is desirable to make the whole service connection in a practically straight line, as it can then be cleared of stoppage in many cases by running a stiff wire through it from the house-end (the corporation cock being first closed, and afterward opened and closed suddenly and for an instant only).

The service connection as far as and including the curb-cock is generally constructed by and retained as the property of the company or department, in order that the consumer may have no right to interfere with the turning off or on of the supply at the curb-cock, and to prevent the tapping of the mains by any but experienced employés of the company or department.

Extensions from the end of a main or from a branch-special previously inserted for this purpose are made in a manner similar to the original construction. If a plug has been inserted in the end of the pipe or branch, the pressure must be cut off from this by closing the necessary gates, the plug be broken out, and the water which flows from the mains drained or pumped away, before the extension can be laid. In some instances where an extension in the near future was probable it has been thought desirable to build this at once as far as the nearest gate and leave this closed. The extension can then be laid at any time without interfering with the service, and the gate opened when it is completed.

In case it is desired to lay a branch-line from a point where no branch-special was inserted, a length of the pipe may be cut out here and a suitable special inserted. Machines have been designed, however, for connecting with any size of main branches of 2 to 42 inches diameter under pressure up to 1000 lbs.; working similarly to the corporation-cock tapping-machines. These machines are somewhat expensive, but may be either bought or rented.

Art. 112. Prevention of Deterioration.

Mains when once laid are subject to few destructive influences not provided against by a proper designing and coating. The internal pressure is resisted by the tensile strength and thickness of the iron shell; pressure from without is seldom if ever sufficient to injure the pipe if this be given the depth of covering specified in Art. 98, except that a settlement of ground under the pipe or away from one side of it may cause a break in the line (although a lead-jointed pipe-line can generally be distorted for several inches before breaking); friction of the flowing water has little if any effect upon mains or conduits, unless the velocity be greater than is ever obtained in a distribution pipe-system, and if it be greater in any conduit than the maximum given in Art. 53 the error in design should be corrected. This can be accomplished in the case of open conduits by building in them at intervals low weirs with either overfalls or sluices at their lower sides; the reduction in velocity necessitating an enlargement of the conduit, unless the diminished flow will still be ample.

Chemical action frequently takes place in metal conduits, as previously stated, unless these be properly coated. A good coating seems to be permanent and require no renewal, no instance having come to the author's notice where this has been attempted. Moist soil outside the conduit, as well as the water within, causes an injurious chemical action, which not only corrodes and pits the surface of the metal, but frequently changes its entire character; iron pipe having been found which could be cut with a knife. This can be prevented only during construction, and the maintenance department can but substitute for weakened pipe other which has been properly protected.

During the last few years electrical action upon iron pipe has become a matter of greater concern than either mechanical or chemical; not so much because its effect is more serious as because its prevention is more difficult. There is probably some electrolytic action taking place in many water-mains, particularly at their junction with branches of brass or other metal, even when there are no large artificially created currents in the ground; but the greatest danger seems to exist where there are wandering currents from trolley-roads. So great has the danger under these conditions seemed to be that several cities have employed experts to examine and report upon the electrolysis taking place in their distribution systems; among these being Dayton, Ohio, Jersey City, N. J., Brooklyn, N. Y., and others. The experts employed in 1898 by Dayton—Messrs. H. P. Brown, E. E. Brownell, and J. H. Shaffer—state that: "The current to operate the trolley-cars leaves the dynamo and passes through the trolley-wires in various parts of the city, then through the motor of each car and the wheels to the rails. To complete the circuit it must return to the negative pole of the dynamos. To reach this pole, two paths are open to it: first, by the rails and feeder wires leading from the rails to the dynamo; and second, through the moist earth to the water- or gas-pipes below, along which it passes until within 1000 to 3000 feet of the power-house, when it leaves the pipes, again passing through the moist earth to the rails. A circuit of electricity passing through a conducting fluid like water decomposes it, and the oxygen and acids, if any, in the compound are delivered at the positive or outgoing pole, while the hydrogen and alkalis are delivered at the negative or receiving pole. The oxygen corrodes the metal of the positive plate, while the hydrogen produces no chemical effect on the negative plate." "The rate of electrolysis depends directly upon the electrical condition of the electrical pressure between the positive and negative plates (the pipes and rails). This rate is increased by the presence of any acid or alkali in the fluid (or soil in contact with pipes and rails). A slight leakage from the gas-pipes, or a small amount of acid from the surface, or the presence of an alkali in the soil will increase the action, and the current itself, by carrying the metal oxide into the ground-water, reduces the resistance of the solution, and tends constantly to cause a further increase of action." The effect of electrolytic action in wrought iron is three times, and in lead seven times, as great as in cast iron; hence service-pipes are especially liable to injury by electrolysis.

The general method of ascertaining whether electrolytic action is taking place in a certain line of pipe is to take voltmeter-readings between the pipe and the trolley-rails above it. (Electric connection with the pipe may be made at a gate or a fire-hydrant.) Where the potential of the pipe with reference to the rail is positive there is a probability of such action existing; $1\frac{1}{2}$ to 2 volts being sufficient to cause alarm and call for further investigation. The remedy lies to a large extent not with the water department or company, but with the trolley-road, and this is generally the most troublesome feature of the difficulty. Insulation of existing water-mains, or even of those laid for the first time, seems to be impracticable, and to prevent the passage of return currents through them it is necessary to provide other return circuits offering less resistance to the current, or to increase the resistance offered by the pipe. Since most if not all of the injury to the pipe is confined to the point or points at which the current leaves it, it is generally possible to so connect the pipe and rails at this point as to permit the return of the current without injury to the former. To increase the resistance to the current, wooden pipe has been substituted for iron in some plants (see Art. 94); and it has been suggested that an occasional pipe of wood or other poor conductor interspersed along each line would be effective. Mr. H. P. Brown, in a paper read before the American Society of Municipal Improvements during the summer of 1899, says: "I again urge the importance of thorough and frequent electrical inspections of the water-pipes and friendly consultation with the railway managers when dangerous conditions are found to exist. And I again advise the insertion of lengths of iron-banded wooden pipes into the water-mains at intervals, so as to make the mains of less conductivity than the rails. This, with good bonding [of the rails], suitable and properly balanced return wires from rails and water-mains, and continued electrical tests, will prevent any serious trouble from electrolysis."

The maintenance of irrigation conduits and distributaries, which are generally upon the surface even when closed, requires constant inspection to prevent or quickly repair any break or other injury due to floods, land-slides, or boulders, or maliciously inflicted. For this purpose the system is ordinarily divided into sections, each with a tool-house and a small gang of men to patrol the canal and make repairs, after the general plan adopted on railroads. It is also desirable to have a telephone system extending along every canal, enabling any section-station or the main office to be reached from any point, each inspector being provided with the necessary instruments for tapping the line. In each section-house should be kept the tools adapted to repairing whatever kind of conduit exists on its section, and the bulky material necessary for this—as lumber, clay and gravel, wooden staves, etc.—should be stored at intervals along the line.

Many of the suggestions made in Arts. 104 and 105 relative to masonry dams and earthen embankments are equally applicable to canals and masonry conduits.

CHAPTER XXI.

CLERICAL AND COMMERCIAL.

ART. 113. KEEPING RECORDS.

A WATER-WORKS department should so record and file all data relative to every part of its construction and maintenance that they may be easily and quickly referred to and be perfectly intelligible; and the information so recorded should embody a complete description of the plant, and the nature and date of all changes and repairs. These data will generally be recorded in the form of maps, written descriptions, and financial statements. It will be convenient to index each of these, and to use a common system of indexing for all. Thus D9 may refer to a certain street intersection, and under this heading be filed all maps of piping and appurtenances, all descriptions of repairs and maintenance operations conducted at this point, and all expenses and receipts connected therewith.

An excellent plan is to have one map of the entire city to a small scale, showing all pipe-lines but no details or appurtenances. This map may be divided into squares by lines drawn approximately at right angles and through the middle of each third, fourth, or fifth block, each line being continuous across the entire map. Successive strips running East and West may then be given the letters of the alphabet, and those running North and South be numbered, a sufficient number of the first letters and smallest numbers being omitted to allow for future extensions. A combination of letter and number would then designate a particular square; D9, for instance, indicating that square common to the East and West strip called D, and to the North and South one called 9. D9–10 would indicate a point on the line separating strips 9 and 10. Reference to this general map would then show at once under what heading to look for data concerning a given location.

This method applies chiefly to the distribution system. Data concerning each pumping-station, reservoir, filter, etc., should be filed separately, but these will not generally be so numerous as to require a system of indexing.

One drawer or compartment may then be given to all maps referring to strip A, another to B, etc.; one being reserved for pumping-station and engine details, another for reservoirs, filters, etc. In each drawer or compartment the maps are arranged consecutively according to the number of the strip. An index of streets may be prepared, giving the blocks crossed by each street and the part of the street contained by each block. If this arrangement is carefully carried out, the finding on the map of any part of the city should be the work of but a minute or two. An excellent method of storing the maps is that in use at Detroit, described in *Engineering Record* for February 12, 1898; the maps being uniform in size and bound in sets of about 30, each set being suspended from a rod in a cabinet 6 feet high, 12 feet wide, and 2½ feet deep. Each bound set may contain the maps for one strip A, B, etc.

The records should give the size, location, depth, and exact arrangement of all pipes, specials, valves, hydrants, and all other appurtenances of the system; as well as details concerning any gas or other pipes, sewers, electric conduits, or other underground structures of which data are obtained from other departments or during excavations in connection with the water-works system.

All work done should be described both in writing and by drawings, with accurate dimensions, recorded on the spot in the engineer's note-book, and the information on each page of this should be entered on the detail maps and check-marked when so entered, and in the index of streets the number and page of note-book should be entered under the proper street.

If meters are used, a record of these should be kept in a separate book, the record of each meter being kept separately (it being convenient to place upon each page the record of the meter having a like number): the date of its purchase; result and date of any tests made upon it; date of placing it in service; that of any repairs, with character and cost of these. The monthly or quarterly readings may be entered in this book, or in a separate book devoted to accounts with the consumers.

An account should be kept of each pumping-station, and of each pump and boiler in the same; showing the amount, kind, and cost of coal used, as well as of all other supplies; the daily pumpage of each engine, with the average head pumped against and the steam-pressure; the days of service of each boiler; describing also any repairs made, with their cost, and giving the names of engineers and firemen on duty each day.

Records should also be kept of reservoirs, showing the height of water each day, giving the dates and circumstances of any pollution or unpleasant taste; together with the daily evaporation, temperature, and atmospheric moisture, and both the daily rainfall and maximum rate. This will require an evaporation-pan, a hygrometer, a maximum and minimum thermometer, and a recording rain-gauge.

It is generally good policy to publish a summary of these data annually and forward it to water departments in other

WATER-WORKS STATISTICS.

I. General and Pumping.

Date of Construction.
By whom Owned.
Source of Supply.
Mode of Supply.
1. Builders of Pumping Machinery.
2. { Coal Used. / Per Cent Ashes. / Price per Ton of 2240 lbs.
3. Coal Consumed for the Year, lbs.
4. $\frac{\text{Pounds of Wood Consumed}}{3}$.
5. Total Fuel Consumed for the Year, lbs.
6. Total Pumpage for the Year, gallons.
7. Average Static Head against Pumps.
8. Av. Dynamic Head against Pumps, feet.
9. No. of Gallons Pumped per Lb. of Coal.
10. Duty in Ft. Lbs. per 100 lbs. of Coal, no Deductions.
11. Cost per Million Gallons Pumped into Reservoir, Figured on Pumping-station Expenses.
12. Cost per Million Gallons Raised One Foot High, Figured on Pumping-station Expenses.
13. Cost per Million Gallons Pumped into Reservoir, Figured on Total Maintenance.
14. Cost per Million Gallons Raised One Foot High, Figured on Total Maintenance.

II. Financial.

Receipts from Consumers:
A. Rates, Domestic.
B. " Manufacturing.
C. Net Receipts for Water.
D. Miscellaneous Receipts.
E. Total Receipts.
Receipts from Public Funds:
F. Hydrants.
G. Fountains.
H. Street-watering.
I. Public Buildings.
J. General Appropriation or Miscellaneous.
K. Gross Receipts.
AA. Management and Repairs.
BB. Interest on Bonds.
Miscellaneous Expenses.
Total Maintenance.

III. Consumption.

Estimated Population:
1. Total at Date.
2. On Line of Pipe.
3. Supplied at Date.
4. Total Gallons Consumed during Year.
5. Quantity Used through Domestic Meters, gallons.
6. Quantity Used through Manufacturing Meters, gallons.
7. Average Daily Consumption, gallons.
Gallons per Day:
8. Each Inhabitant.
9. Each Consumer.
10. Each Tap.

IV. Distribution.—Main Pipes.

1. Kind of Pipe.
2. Sizes of Distribution Pipe, inches.
3. Length Extended during Year, feet.
4. " Discontinued " " "
5. Total Length in Use, miles.
6. Cost of Repairs per mile.
7. Number of Leaks per mile.
8. Length of Pipe less than 4 in. diam., miles.
Hydrants:
9. Number Added.
10. Total in Use.
Stop-cocks:
11. Number Added.
12. Total in Use.
13. Small Stop-gates less than 4 in., total.
14. Number of Blow-off Gates.
15. Range of Pressure, pounds.

V. Distribution.—Service-pipes.

Service-pipe:
16. Kind of Pipe.
17. Sizes of Pipe, inches.
18. Length Extended during Year, feet.
19. " Discontinued " " "
20. Total Length in Use, miles.
Service-taps:
21. Number Added.
22. Total in Use.
23. Average Length of Service.
24. Average Cost of Service.
Meters:
25. Number Added.
26*a*. Now in Use, Domestic.
26*b*. " " " Manufacturing.
Motors and Elevators:
27. Number Added.
28. Total in Use.

cities which will do the same; since much of value can be learned by this interchange, and, the various departments being in no way competitors, there can be no objection to the publicity. The value of these data is greatly increased if those of all cities be analyzed and arranged uniformly.

Two or three general systems have been suggested, but probably more water departments follow that adopted by the N. E. W. W. Ass'n than any other. This arrangement of statistics is given on page 535.

Art. 114. Meters and Rates.

The desirability of preventing waste was stated in Arts. 14 and 16; and the use of meters for this purpose has been referred to. These devices have been in general use for a few years only, but there are few if any cities in which they have been introduced whose water-works officials and citizens are not in favor of their use, although much opposition to their original introduction is generally encountered. It is better to introduce meters at the very first, both because considerable senseless opposition is generally raised to their introduction later, and to prevent the consumers from acquiring the habit of wasting water, which, like other habits, it is difficult to overcome. One of the best arguments for meeting public opposition is the proof—generally obtainable—that in the majority of cases where meters are not used ten to twenty per cent of the consumers do most of the wasting, which the other eighty or ninety per cent must pay for (see page 43); and also that if the water be metered a less sum than the average rates which are being or would otherwise be charged need be paid by any but recklessly wasteful consumers. Where there are no legal obstructions, it is generally desirable to set as a minimum amount which will be charged for, one sufficient to fully meet all requirements of health and cleanliness.

There are a number of excellent meters on the market,—and several worthless ones. No cheap meter has yet been designed which is also good; and the hard service combined with accuracy required would seem to make this impossible. Delicacy—the registering of small quantities—is, in the author's opinion, more important than extreme accuracy; since a leak or other constant flow at a low rate is most often the cause of waste; while the actual charge for water does not warrant incurring extra expense to insure the accuracy of the bill within only a few gallons. It is generally considered advisable that a meter under-register rather than over-register, since one consumer's complaint found justifiable will more than outbalance, in the popular mind, a dozen false claims of overcharging.

For ordinary domestic service a $\frac{5}{8}$-inch meter is amply large. For large hotels, factories, railroads, and other large consumers, 4- or even 8-inch meters may be necessary. The price increases rapidly with the size. For service-pipes of 8-inch diameter or above, a Venturi meter can be used to advantage, but for smaller services its cost is excessive, and it is not so sensitive as is a good rotary or disk meter.

There is of necessity a certain amount of head lost in each meter, required to work the measuring and registering mechanism. Meters are obtainable which consume no more than 1 to 5 lbs. in ordinary service. Four- to eight-inch factory meters have been found to cause a loss of 1.1 to 9.3 lbs. for a discharge of 250 gals. per minute, and up to 40 lbs. for 500 gals. per minute (see Journal N. E. W. W. Ass'n, December 1897), although the loss during a flow of 500 gals. has, with certain makes of meters, been found to be as low as 4 lbs. in a 4-inch and 0.7 lb. in an 8-inch meter. Of fourteen $\frac{5}{8}$- and $\frac{3}{4}$-inch meters of different patterns tested by Mr.

J. W. Hill in 1898, the head lost varied from 0.50 lb. with a discharge of about 170 gals. per hour to 26.68 lbs. with a discharge of 865 gals. per hour (see Trans. Am. Soc. C. E., vol. XLI. page 326).

Under any but exceptional circumstances a well-made meter should give good service for many years. The consumption of a family will average about 10,000 cubic feet per year. Six meters tested by Mr. J. Waldo Smith in 1894–5 registered from 711,000 to 1,644,000 cubic feet each, and the four of these which had passed the most water were still in running order; giving a life of 71 to 164 years of average use. The meter may of course be damaged by a stick or stone or may be stopped by tubercles or other matter collecting in it; but this is due to a faulty condition of the pipe or water which should be remedied; and large matters may be kept out of the meter by placing a screen at its inlet. While the accuracy, sensitiveness, loss of head, and accessibility for repair of any meter can readily be ascertained in a few minutes, the durability cannot readily be tested, but can be judged from the mechanical perfection of the various parts and the hardness, toughness, and non-corrosiveness of the materials employed. Meters are in some cases owned by the company or department, in others by the consumer; the latter is occasionally required to deposit a sum sufficient to cover any damage to the meter caused by him, or is charged a rental for its use. Their use is compulsory in some cities, optional in others. Probably the majority of cities begin by metering the largest service-pipes, although some have found the greatest waste to occur from plumbing in cheap houses.

The services rendered by a water-supply system may be considered under the two general heads of public and private, and returns from each of these may be justly demanded. The public services include fire-protection, street-sprinkling, sewer-flushing, public fountains, and other similar purposes for which the municipality as such makes use of water. The private services include all those for which water is taken from house-service connections, to be used upon private premises, whether in the house or the barn, in sprinkling lawns, running elevators, etc. A determination of the rates will generally be based upon the value of the services rendered to the consumers, and the cost to the company or department; the latter of which is much the more readily estimated and forms the principal basis of charges in most cases.

Fire-protection requires the furnishing of little additional water, but proper provision for it increases by at least one third the original cost of most plants. The supply for sprinkling streets, flushing sewers, and other public purposes probably amounts to between 5 and 25 per cent of the total consumption in most cities. It would, upon this basis, seem just to obtain from the public treasury (*a*) 33% of the interest upon the cost of construction and of the annual contribution to the sinking fund, and (*b*) 10 or 15 per cent of the running expenses; the remainder (*c*) to be paid by private consumers. The payment of (*b*) and (*c*) is frequently based upon actual measurement, where this is possible.

Where the water-works are owned by a private company the revenue from fire-protection generally takes the form of hydrant rental; that from other public services is in some cases determined by meter, in others a lump sum is paid, but in most this is included in the hydrant rental, or is given as a return for the franchise. When owned by the city the water-works are usually under the control of a separate department, and all revenue is received and disbursements made by this. In many cities the department receives no revenue for public services, but instead asks for and sometimes receives annual appropriations from the public treasury. The praiseworthy practice is becoming more general, however, of making the department self-supporting, and of paying into its treasury revenues (*a*) and (*b*), as a private company would be paid. The advantages of this plan are appreciated by every water-works superintendent who has annually struggled with successive councils for his appropriation, and has conducted his department with embarrassing uncertainty as to the amount he will receive.

The hydrant "rentals" received by different water companies vary between very wide limits, as is shown by the following table. This variation is partly due to differences in the cost of the plants, but more to the shrewdness of the company or carelessness of the city officers in preparing the franchise. The majority of rates given in the table are seen to lie between $15 and $50.

HYDRANT RENTALS IN THE UNITED STATES IN 1888.

(From the "Manual of American Water-works.")

Hydrant Rate.	North-western.	South-western.	Pacific.	Total for Western.	Total Eastern.	Total United States
Free	1	1	15	17	26	43
$ 5	....	....	2	2	9	11
10	....	....	2	2	16	18
15	....	....	....	....	29	29
20	....	....	4	4	22	26
25	....	1	6	6	24	30
30	1	2	2	5	33	38
35	1	1	....	2	26	28
40	3	2	4	9	44	53
45	1	2	1	5	27	32
50	13	12	4	29	60	89
55	3	....	....	3	8	11
60	9	8	1	18	24	42
65	2	4	1	7	....	7
70	4	4	....	8	....	8
75	12	4	....	16	24	40
80	5	4	11	20	....	20
85	3	6	1	10	....	10
90	....	3	....	3	4	7
100	18	15	3	36	5	41
120	....	....	1	1	....	1
125	....	1	....	1	....	1
150	....	1	....	1	....	1
160	1	....	....	1	....	1

The geatest differences and difficulties are found in fixing rates for private consumption. In probably a majority of cases a rate is fixed for each kitchen faucet, water-closet, and other appliances for using water. There seems to be no uniformity in the rates for each of the various fixtures, either actual or relative. Of thirty cities having a population of 80,000 or over whose rates were studied by Mr. August Hermann in 1899, the cost for a six-room house occupied by one family on an 18- or 20-foot lot, having a yard hydrant, kitchen sink, stationary wash-stand, bath with hot and cold water, water-closet, and two-tray laundry, varied from \$3.70 per annum in Detroit to \$31.75 in New Orleans, the average being \$15.42. For a twelve-room house having, besides the above, two stationary wash-stands, one bath with hot and cold water, and two water-closets, the rates varied from \$13 in Baltimore to \$52.25 in New Orleans, averaging \$28.84. In some cities the rates are given for seventy-five to one hundred items, in others for not more than twelve or fifteen. These items include fixtures as such, purposes for which the water is used, character of building, size of building or lot, and nature of business conducted therein. Brooklyn and Albany, N. Y., charge for vacant lots, the latter 10 cents per annum per front foot, the former from 10 cents per running foot for lots assessed at \$100 or under, up to 20 cents for those assessed at \$2000 or over.

Meter rates are in some cases graduated, in others "straight" or "flat." Straight rates range from 5 cents to \$1 or more per 1000 gals., probably averaging about 20 or 25 cents. Graduated rates are based upon the amount of consumption, large consumers obtaining low rates. There are two general methods: basing the rate for each month, quarter, half-year, or year (the period for which payments are made) upon the total consumption for that time; or charging a certain amount for the first unit quantity of con-

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sumption during that time, and a decreasing amount for each additional unit quantity. By the first method the charge might be 50 cents a thousand gallons for 20,000 gals. or less, 30 cents for more than 20,000 and less than 40,000 gals., etc. In this case, if a consumer found that his meter registered between 19,000 and 20,000 gals. at the end of the quarter, it would be to his advantage to waste sufficient water to bring this above 20,000 and obtain the lower rate. To prevent this several cities have adopted the second plan. At Madison, Wis., the rates are as follows:

For the first 5,000 cu. ft. in six months,	20 cts. per 100 cu. ft.
" " next 15,000 " " " " "	10 " " " " "
" " " 10,000 " " " " "	5 " " " " "
" " " 30,000 " " " " "	3 " " " " "
" " " 30,000 " " " " "	2 " " " " "
Over 90,000 " " " " "	3 " " " " "

The rates at Reading, Penn., are:

First	1,000 gallons or less per month,	50 cts. per 1000 gallons
Next	2,000 " " " " "	22 " " " "
"	2,000 " " " " "	18 " " " "
"	5,000 " " " " "	14 " " " "
"	10,000 " " " " "	8 " " " "
"	10,000 " " " " "	6 " " " "
"	20,000 " " " " "	5 " " " "
"	50,000 " " " " "	4½ " " " "
"	100,000 " " " " "	4 " " " "
"	300,000 " " " " "	3½ " " " "
"	500,000 " " " " "	3¼ " " " "
"	2,000,000 " " " " "	3 " " " "
Over	3,000,000 " " " " "	2¾ " " " "

In many if not a majority of cases a minimum charge is made of from $1 to $5 to cover reading the meter and clerical work, with interest on the cost of the service connection.

ART. 115. FINANCIAL.

A city or irrigation water-supply system which is owned and operated by a private company will be conducted upon ordinary business principles; but to apply these it is necessary to know beforehand what will be the cost of construction and of maintenance, and what the life of the plant. In the case of municipal ownership there is generally the additional problem of paying for the construction. Few cities could pay cash for this from their treasury, nor is it considered desirable to compel the present inhabitants to furnish a water system free of debt to their descendants or successors; but these should pay as much per annum for the service rendered as does the present generation.

An ideal plan would be to issue such bonds and make such annual payments toward a sinking fund that when the piping system, for instance, shall need renewing the bonds by which it was built will just mature and be exactly met by the sinking fund; and that the same be true of the reservoirs, pumping-plant, and other parts of the system. Extensions would be paid for in the same way; and repairs on a given section would be paid for out of the sinking fund connected with that section. This plan, if carried out in all its details, would be unnecessarily complicated, but it can be followed in a general way; the plant, for instance, being divided into the reservoirs and permanent head-works, the piping system, the pumping-plant, and the filtration-plant; any smaller items being combined with that class which is thought to have a similar length of life. Extensions could be paid for by the proceeds from a bond issue made once every five years or so. Service connections and maintenance would be paid out of the annual receipts; the former generally being charged against each property when it is connected with the main. The rates must be so fixed that the receipts will be sufficient to meet the maintenance expenses and interest on the bonds, and make the annual payments into the sinking fund.

The cost of constructing the various parts of the plant has already been considered. The life of a plant can seldom be estimated with any exactness. A well-built reservoir should have a life terminated only by the insufficiency of its capacity. A pipe-line of cast iron also should, if well coated, be serviceable until its capacity is exceeded by the consumption. The experts appointed to appraise the value of the property of the Los Angeles City Water Company considered the life of the cast-iron pipe to be from 50 to 80 years; of the riveted-iron and steel pipe to be 15 to 25 years; of wrought-iron standard-screw pipe to be from 17 to 30 years. An internal depreciation also, due to tuberculation, was allowed for. The life of pumps depends upon their original character and the care taken to maintain them. A well-designed and constructed pump should give good service and efficiency for at least forty years. The boilers will not ordinarily last so long. Probably thirty years would be a fair estimate of the average length of life of a pumping-plant. The life of filter-plants has already been considered in Arts. 77, 78 and 108.

The cost of maintenance of filter-plants has been treated of in Art. 108; that of conducting a pumping-station in Art. 107. The cost of repairs in the distribution system in Fitchburg, Mass., in 1896 was $5.32 per mile of pipe, and in Taunton the cost of repairs and maintenance in 1893 was $20.55, in 1895, $25.33; and in 1896, $29.85. The total maintenance in Taunton during the same years cost $39.37, $49.04, and $49.82; and the total expenses of the plant were $119.57, $163.55, and $166.55. At Newton, Mass., the maintenance in 1897 cost $27.55, and interest on bonds $140.67, per million gallons. Maintenance in the two latter cases was the total maintenance of the plant. Maintenance of the distribution system will ordinarily cost $15 to $35 per mile per annum, including pipes, valves, fire-hydrants, etc., but not service connections. If there be an unusual number of leaks and breaks in the line, these may increase the cost to $100 or more. Service connections will generally cost about $5 to $7 each up to and including the curb-cock, and 20 to 35 cents per foot from here to the house; but the cost of these is generally paid by the consumer at the time the connection is made, although the company or department retains all rights of ownership of that part up to and including the curb-cock. Under the head of maintenance must be included all costs of administration, office force, collectors, etc.

Water-rents are collected monthly in some cities, quarterly in others, and in still others half-yearly. Probably the plan of quarterly collections is the one most generally adopted. In most cities the failure of any consumer to make payment within a given time—say two weeks—of presentation of the bill results in the water being shut off from his premises until all indebtedness is settled, including a small additional sum for the trouble of closing and opening the curb-cock. In some instances a discount of 5 to 20 per cent is allowed for immediate payments; in others interest is charged after about ten days from the time when payment is due.

A few water companies and departments have made it a rule that excessive waste, if continued after a warning, be stopped by cutting off the supply from the premises of the offender. Probably a better plan, seeming less arbitrary and being more remunerative, is to introduce meters on such services (if they are not already in general use), and follow the customary rule of shutting off the supply until payment has been made for all the water passing the meter.

Since the receipts during the first year or two after the construction of the plant are likely to be small, while the expenses of maintenance will then be as great as, if not greater than, after all weak points have been discovered and repaired, and experience has pointed out many methods of economizing, it may be desirable to begin the regular payments into the sinking fund only after the first or second year, as otherwise the high rates necessarily charged would discourage rather than encourage the general use of the system.

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