PUBLIC WATER-SUPPLIES

REQUIREMENTS, RESOURCES, AND THE CONSTRUCTION OF WORKS

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PREFACE TO THE SECOND EDITION.

SINCE the publication of the first edition of this work, in 1901, there has been a noteworthy development in the design and construction of works for public water-supplies. While this development has been greatest in the methods of water purification and in the construction of purification works for the large cities of the country, yet it may be said that the engineering of water-works has in general been brought to a more scientific as well as economical basis. In the revision of this work the authors have endeavored to bring it into accordance with the best modern practice.

The chapters relating to the purification of water have been thoroughly revised, that on mechanical or rapid filtration being rewritten and greatly enlarged. In view of the essential differences between the two systems of filtration and the direction along which their development is taking place the authors decided to change the term "mechanical filtration," formerly used, to "rapid sand filtration," and to employ the term "slow sand filtration" for the other system. The subject of coagulation is now made an important part of the chapter on Sedimentation and Coagulation. Besides the matter relating to purification many other changes and additions have been made in nearly every chapter. The most important of these relate to methods of bacterial examination of water, the investigation of ground-water and the construction of collecting works, data on the use of water, data on rainfall and flow of streams, the construction of dams, and the application of reinforced concrete to conduits, dams, filters, reservoirs, and tanks. The literature of each chapter has also been extended and brought up to date.

F. E. T.
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MADISON, WIS., July, 1908.

PREFACE TO THE FIRST EDITION.

THE present volume has been prepared with particular reference to the needs of teachers and students in technical schools in which the subject of Water-supply receives a considerable amount of attention. The work is based chiefly upon the experience of the first-named author in teaching the subject for a number of years in the institution with which he is connected, and has been written with special reference to use in his own class-room.

In the discussion of the various subjects treated, the endeavor has been to lay stress upon fundamental principles rather than upon details of practice, although methods of construction have been freely given where they might serve to illustrate the principles involved or bring out the effects of differences in conditions. With the same idea in mind many problems, usually treated empirically, have been subjected to analysis, more or less crude, but useful for calling attention to certain general laws and limitations. It is believed also that such analyses may often be of much assistance in utilizing the results of observation, and that, if properly applied, they will aid much in the cultivation of the judgment. The necessity for the designer to keep constantly before him the question of true economy has been frequently emphasized, and to aid the beginner a brief general discussion of this subject has been given in Chapter XI. No apology is necessary at this time for the comparatively full treatment given to the subject of the Quality of Water-supplies in Chapters VIII, IX, and X. The authors have felt that the great importance of questions relating to the purification of water requires a more thorough presentation of the sanitary phase of the subject than has heretofore been customary in works designed for engineers. The subject of Ground-water has also received considerably more attention than is usual, but, it is thought, not more than the importance of the subject will justify.

References to authorities are numerous, and the plan has been adopted of giving, at the end of each chapter, a brief list of the best literature of the subject treated. It is believed that this feature will prove of value not only to the student, but especially to the young practitioner who finds it necessary to make a special study of a particular branch of the subject. According to the authors' view, there is no branch of the profession in which a good working library, consisting largely of periodicals, is more necessary than in that of municipal or sanitary engineering.

To the water-works specialist there is doubtless little that is new to be found in this work, but it is hoped that the form in which a large amount of widely scattered information has here been presented will prove of convenience to this class of readers.

With regard to the authorship it is proper to say that Chapters VIII, IX, and X are by Prof. Russell; also several of the articles of Chapters XIX to XXIII, which relate more specifically to bacteriological and chemical features. The remainder of the work, with the exception of the chapter on Pumping-machinery, has been written by Prof. Turneaure.

The authors desire to acknowledge their indebtedness to the various engineers and water-works officials who have kindly responded to requests for information. They are also under special obligations to Mr. C. B. Stewart, Assoc. M. Am. Soc. C. E., for a very thorough investigation of the literature of the flow of water in pipes, the results of which appear on pages 227–234, including the diagram of Fig. 34. Of the large number of original articles and papers which have been consulted, a great many have appeared in the *Engineering News*, the *Engineering Record*, or the Transactions of the American Society of Civil Engineers; and to the publishers of these journals special thanks are due for many of the illustrations which appear in this work.

F. E. T.
H. L. R.

MADISON, WIS., March, 1901

CONTENTS.

CHAPTER I.

INTRODUCTION.

PART I.

REQUIREMENTS AND RESOURCES.

A. QUANTITY OF WATER REQUIRED: SOURCES OF SUPPLY.

CHAPTER II.

QUANTITY OF WATER REQUIRED.

CHAPTER III.

SOURCES OF SUPPLY.

CHAPTER IV.

THE RAINFALL.

CHAPTER V.

EVAPORATION AND PERCOLATION.

CHAPTER VI.

FLOW OF STREAMS.

CHAPTER VII.

GROUND-WATER.

B. QUALITY OF WATER-SUPPLIES.

CHAPTER VIII.

EXAMINATION OF WATER-SUPPLIES.

CHAPTER IX.

QUALITY OF WATER.

CHAPTER X.

COMMUNICABLE DISEASES AND WATER-SUPPLIES.

PART II.

THE CONSTRUCTION OF WATER-WORKS.

CHAPTER XI.

GENERALITIES PERTAINING TO WATER-WORKS CONSTRUCTION.

CHAPTER XII.

HYDRAULICS.

A. WORKS FOR THE COLLECTION OF WATER.

CHAPTER XIII.

RIVER AND LAKE INTAKES.

CHAPTER XIV.

WORKS FOR THE COLLECTION OF GROUND-WATER.

CHAPTER XV.

IMPOUNDING-RESERVOIRS.

CHAPTER XVI.

EARTHEN DAMS.

CHAPTER XVII.

MASONRY DAMS.

CHAPTER XVIII.

TIMBER DAMS; STEEL DAMS; LOOSE-ROCK DAMS.

B. WORKS FOR THE PURIFICATION OF WATER.

CHAPTER XIX.

OBJECTS AND METHODS OF PURIFICATION.

CHAPTER XX.

SEDIMENTATION.

CHAPTER XXI.

SLOW SAND FILTRATION.

CHAPTER XXVI.

PUMPING-MACHINERY.

CHAPTER XXVII.

DISTRIBUTING AND EQUALIZING RESERVOIRS.

PUBLIC WATER-SUPPLIES.

CHAPTER I.

INTRODUCTION.

HISTORICAL SKETCH.

1. **Water-supplies in Ancient Times.**—The earliest method of artificially obtaining a water-supply was doubtless by the digging of wells. These were naturally at first mere shallow cavities scooped out of the ground in moist places, such as are used at the present time by savage tribes; but as necessity arose and the use of implements developed, these wells were gradually deepened.

The digging of wells dates from a very early period. In the vicinity of the pyramids there still exist wells which were in use when those great works were constructed. Joseph's well at Cairo is perhaps the most famous of all ancient wells. It is a remarkable work and exhibits in a high degree the skill of the people of ancient Egypt in matters pertaining to construction. It is excavated in solid rock to a depth of 297 feet and consists of two stories or lifts. The upper shaft is 18 by 24 feet, and 165 feet deep; the lower is 9 by 15 feet and reaches to a further depth of 130 feet. Water is raised in two lifts by means of buckets on endless chains, those for the lower level being operated by mules in a chamber at the bottom of the upper shaft, to which access is had by means of a spiral pathway winding about the well.*

Frequent mention is made by the old historians of important wells in ancient Greece, and remains of such works are numerous in Assyria, Persia, and India. Probably the deepest wells were dug by the Chinese, depths of 1500 feet or more being reached by methods almost identical with those now in common use.

* Ewbank's Hydraulics, p. 45.

Besides the digging of wells, the ancients executed many works for the storage and conveyance of water. In Jerusalem underground cisterns were built for the storage of rain-water; and other reservoirs were constructed near the city to store the water which was brought thither in masonry conduits. Aqueducts were also built in ancient Greece, one mentioned by Herodotus as built to supply the city of Samos being still in good preservation. Some of these ancient aqueducts included inverted siphons of cut-stone blocks. Ruins of extensive underground reservoirs are to be found on the site of ancient Carthage, which it is believed were constructed prior to the capture of the city by the Romans. Works for irrigation in Egypt, Assyria, and India were established on an immense scale, one reservoir in Egypt, Lake Maeris, having had, it is said, an area of 30,000 acres. In the Presidency of Madras, India, the English found at the time of their occupation about 50,000 reservoirs for irrigation purposes, the construction of which had involved the building of 30,000 miles of earth embankment. Many of these reservoirs were doubtless of ancient construction.

2. **Water-works of the Romans.**—Among ancient systems of water-supply the works of no other nation equaled those of the Romans, either in point of size or number; and no city in the Roman Empire was more abundantly supplied than the city of Rome itself. Previous to about 312 B.C. Rome obtained its water from the Tiber and from springs and wells in the immediate vicinity, but this water finally became so badly polluted that a purer supply was sought from distant sources.

3. *Aqueducts.*—The conveyance of water from these new sources necessitated the construction of long conduits or aqueducts. These were often led through hills in tunnels, or carried over valleys on long lines of arches that are to this day the object of our wonder and admiration. The Romans, and indeed the Greeks, well understood the principle of the inverted siphon, and used it on occasion; as, for example, in the works of Lyons, France, where they constructed a siphon consisting of nine miles of lead pipe from 12 to 18 inches in diameter, working under a 200-foot head. The only materials, however, which could be used for this purpose were stone, lead, and pottery, iron pipes being unknown; and the engineers of that time adopted what was doubtless the most economical method of crossing depressions, that is, by carrying the conduit on arches.

The first aqueduct built to supply Rome was called the Aqua Appia, after its builder, Appius Claudius. It was constructed about

FIG. 1.—ROMAN AQUEDUCTS CLAUDIA AND ANIO NOVUS; 38–52 A.D. (HERSCHEL.)
(Anio Novus is built on the top of Claudia.)

FIG. 2.—AQUEDUCT OF SEGOVIA IN SPAIN. (HERSCHEL.)
Built about 109 A.D and still in use.

312 B.C. and had a length of about 11 miles. A second was built about 270 B.C. with a length of 39.5 miles, 1080 feet of which was supported on arches. Others were constructed from time to time until, with the completion of the Anio Novus about 52 A.D., there were nine aqueducts furnishing water to the city of Rome. These are described in detail by Frontinus, a Roman surveyor and water commissioner, in a work written A.D. 97,* in which he also gives much interesting information concerning the various matters coming within his official duties. Five more aqueducts were constructed after the time of Frontinus, the last dating about 305 A.D. The aggregate length of the fourteen was 359 miles; and aggregate length of arches, 50 miles. In cross-section the aqueducts of Rome varied from 3 to 8 feet in height by 2½ to 5 feet in width, and were built with vertical sides and flat or arched roofs. The interior was finished with great care to secure imperviousness, but in spite of this they were constantly getting out of repair.

The Romans not only built works for supplying their chief city, but also executed many works of great importance in all parts of the Empire, as at Paris and Lyons in France, Metz in Germany, and Segovia and Seville in Spain. One-half of the aqueduct at Metz is still in use, although built in the year 130 A.D. That at Nimes, France, is famous for its great aqueduct bridge, the Pont du Gard, where three tiers of arches rise to a maximum height of 158 feet.

4. *Distribution System.*—The distribution of water in this age was by no means general. In Rome the water from the aqueducts first passed into large cisterns, and from these was distributed through lead pipes to other cisterns, and to the fountains, baths, and various public buildings, and to private consumers. The last class was very limited in number, most of the people being obliged to get their supply from the public fountains. Each service required a separate pipe leading from the distributing cistern, and the amount of water to which the consumer was entitled was measured by means of a short tube of specified diameter. At the time of Constantine there were in Rome 11 great thermæ, 926 public baths, 1212 public fountains, and 247 reservoirs.†

5. *Quantity of Water Supplied.*—The amount of water supplied to ancient Rome was very liberal. It has been estimated as high as 400 million gallons per day at the time of Frontinus, but after a careful study of the evidence, and allowing for the fact that usually some of

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* See reference (13), p. 14.

† Lanciani. The Ruins and Excavations of Ancient Rome (1897), p. 56.

the aqueducts were out of repair, Mr. Herschel estimates the probable quantity delivered within the city at about 50 million gallons daily, or about 50 gallons per capita. Even at the latter figure the supply must be considered as very liberal.

6. *Quality of Water.*—The ancients had some clear notions concerning the quality of water-supplies. In his time, Hippocrates knew something of the danger of drinking water which had passed through lead pipes, and even recommended the boiling and filtering of polluted water. At Rome the different aqueducts brought waters of quite different qualities. The best was used for domestic purposes and the other for baths and various public purposes, the water from one aqueduct being of such poor quality that as a rule it was used only for irrigation and for supplying the basin of a marine circus. In some cases water was passed through artificial reservoirs to purify it by sedimentation.

7. **The Middle Ages.**—The fall of Rome brought with it the destruction of the aqueducts and the general neglect of the entire subject of water-supply. The Popes maintained with various interruptions a supply to the city of Rome, and a few other important cities were scantily provided with water. In other places, however, the supplies entirely ceased; and it is said that in some cases the inhabitants even forgot the use to which the old works had been put.

The terrible ravages of pestilence during the Middle Ages were doubtless due in large measure to the use of grossly polluted water, and it was not until about the end of the sixteenth century that general improvement began to be made in sanitary matters. However, as exceptions to this there should be mentioned the construction of a few important works in Spain by the Moors, such as those at Cordova in the ninth century, and the repair of the Roman aqueduct at Seville in 1172.

Paris depended entirely on the river Seine for its water-supply until a small aqueduct was constructed in 1183, but as late as 1550 the supply amounted to only one quart per head per day. In London small quantities of spring-water were brought to the city as early as 1235 by means of lead pipes and masonry conduits. The first pump was erected on the old London bridge in 1582 for the purpose of supplying the city through lead pipes. In Germany water-works were constructed as early as 1412, and pumps were introduced in Hanover in 1527. Mention should here be made also of the aqueduct of Zempola in Mexico, constructed by a Franciscan monk between 1553 and 1570, which for two centuries served to convey water from Zempola to

Otumba. It had a length of 27.8 miles and included three arch bridges of a maximum height of 124 feet.*

8. Development of Modern Water-works in Europe.—During the seventeenth and eighteenth centuries progress was slow, and confined mainly to the cities of Paris and London. Pumps operated by water-power were erected in Paris in 1608. The aqueduct of Arcueil was completed in 1624 and delivered about 200,000 gallons per day, but at the end of the seventeenth century the supply to Paris was as yet only 2½ quarts per head. In London various pumps were erected on the bridge from time to time which drew their supply from the river and were operated by the current. In 1619 the New River Company was incorporated and laid its pipes throughout the city. It received its supply from the New River, and for the first time the general principle was adopted of supplying each house with water. This company still supplies a part of London.

The application of steam to water-pumping in the eighteenth century gave a great impetus to the development of water-works. Probably the first use of steam for this purpose was in London in 1761. A steam-pump was also erected in Paris in 1781 and another in 1783, and a second in London in 1787. In all these instances the supplies were taken directly from the adjacent rivers.

Since 1800 the supplies of both London and Paris have been greatly augmented from various sources. Some of the works are very noteworthy, as, for example, the two aqueducts, of respectively 81.5 and 108 miles in length, constructed to bring spring-water to the city of Paris.

In 1890 the supply of Paris was about 65 gallons per capita, of which about three-fourths was drawn from rivers and used for street-washing and other public purposes, while only one-fourth, or about 16 gallons per capita, was drawn from springs and used for domestic purposes. The latter quantity having been found inadequate, an additional supply of about 30 million gallons was brought to the city in 1892 by means of another aqueduct 63 miles long, thus giving an additional supply of about 12 gallons per head. A still further addition of some 15 million gallons has recently been provided for.

The water-supply of London was brought under municipal management in 1904, previous to which time the city was supplied by eight separate companies. About 55 per cent of the supply is from the Thames, 25 per cent from the Lea, and 20 per cent from springs and wells in the chalk. All river-water is filtered. The total population

* *Eng. News*, 1888, xx. p. 2.

supplied is about 6,000,000, and the rate of consumption is about 40 gallons per capita daily.

Notwithstanding the early existence of public water-supplies in a few cities, the general development of water-works was very slow in the first half of this century; for example, as late as 1864 there had been constructed in Germany but twenty-four water-works. During the last thirty years, however, the development in all civilized countries has been very great, and the rate of growth has constantly increased.

9. For many years the larger pipes were usually of wood, made by boring out logs to a diameter of 6 or 7 inches. Cast-iron pipes came into general use about 1800; and in 1820 the New River Company of London replaced its wooden mains with cast-iron ones at a cost of $1,500,000. At one time this company had about 400 miles of wooden pipe in use, and often as many as ten lines of pipe were laid side by side to form a single main.

When water first began to be supplied to each house it was thought quite impracticable to furnish a continuous supply. Instead, the water was turned on for only a few hours in the twenty-four, at which time the consumers were obliged to lay in their supply for the day. For sanitary reasons, and as a matter of convenience, the constant-supply system came into general use in spite of the many arguments against it. It was introduced in London in 1873, but as late as 1891, 35 per cent of the total supply was still on the intermittent system.

In Europe the question of quality has received as much attention as that of quantity. Great expense is borne to secure, if possible, water from springs or mountain streams, but where this is impracticable, efficient purification works are established. In the early part of this century some use was made in Paris of artificial filters for purifying the water from the Seine; but filtration on a large scale was first inaugurated by the Chelsea Company in London, which in 1829 started the first large sand filter similar to those now in such extensive use. In the last twenty-five or thirty years the use of such filters has rapidly extended until now it is a rare exception to find a European city using unfiltered surface-water.

10. **Development of Water-works in the United States.**—*Early Works.*—The first works in America for the supply of water to towns were those of Boston. They were built in 1652 and served to bring water by gravity from springs. The first instance where machinery was used was at Bethlehem, Pa., the works of which were put into operation June 20, 1754. In this case also the water was from a

spring, which is still in use as a water-supply. It was forced by a pump of lignum vitæ of 5-inch bore through hemlock logs into a wooden reservoir. Eight years later the builder of these works, Hans Christ. Christiansen, replaced the wooden pump by three iron ones of 4-inch bore and 18-inch stroke which were in use for seventeen years. The next works constructed were probably those at Providence, R. I., in 1772; and the next, those at Morristown, N. J., put into operation in 1791, and which still furnish water to the town.

The first use of the steam-engine was at Philadelphia in 1800. These curious engines were constructed largely of wood, even the boiler being partly of this material. The duty was 4,790,000 foot-pounds per 100 pounds of coal.* Steam was applied to New York's water-supply in 1804, these works having been inaugurated in 1799.

In the United States, as in Europe, wooden pipes were at first used, but it is stated by Chanute † that cast-iron pipes were used in Philadelphia as early as 1804, thus antedating by a few years their use in London.

Besides the works above mentioned some others were constructed at an early date, the total number in 1800 being 16. Important steps in advance were made by the construction, in 1822, of the enlarged works at Philadelphia and, somewhat later, of the gravity works of New York and Boston.

11. *Progress since 1850.*—The principal development in this country has taken place since 1850, and the improvements made have been very marked. Among these have been the perfection of cast-iron pipe; the improvements of pumping machinery, whereby a duty is now obtained greatly in excess of what was considered possible twenty-five years ago; the manufacture of the smaller pumps on a commercial scale, thus greatly reducing the cost to small towns; the adoption of direct-pumping systems for small towns, thus also in many cases greatly reducing first cost; and the development of the ground and artesian water-supplies in the Western States. The public water-supply has now come to be so much of a necessity that it is rare to find a village of 2000 inhabitants without its public supply.

The growth in the number of water-works since 1850 is shown by the following table taken from the "Manual of American Water-works" for 1891 and 1897. It gives the total number of water-works in existence at the end of various years, and the number built in each period.

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* Illustrated description in *Eng. News*, 1887, XVII. p. 247.

† Trans. Am. Soc. C. E., 1880, IX. p. 220.

Year.	Number of Works.	Number of Works Built in each Period.	Year.	Number of Works.	Number of Works Built in each Period.
1850	83	....	1875	422	179
1855	106	23	1880	598	176
1860	136	30	1885	1013	415
1865	162	26	1890	1878	865
1870	243	81	1896	3196	1318

The new works built between 1890 and 1896 were of course mainly for small towns, but a large amount of work has also been done each year in increasing the supplies for the larger cities. In 1880 the total population supplied was 11,809,231, while in 1890 it was 22,814,061, nearly one-half of the increase being due to the increase in population of cities already supplied in 1880. The total estimated cost of the works up to 1891 was $543,000,000; number of miles of mains 32,400, taps 2,213,000, and hydrants 220,000.

12. *Present Conditions and Necessities.*—As regards the improvement in the quality of water supplied not so much progress has been made as in increasing the quantity, and in this respect this country is far behind Europe. A large proportion of our largest cities use water taken directly from streams more or less polluted by sewage, and as yet few of these supplies are subjected to any purification process. The problem here is rendered especially difficult by reason of the enormous quantities of water used by American cities as compared with those of other countries.

From this statement of present conditions it is evident that the engineering work of the future lies principally in the development of new and better sources of supply, in providing increased quantities for our rapidly growing cities, and especially in the improvement of the quality of existing supplies. In the management of water-works, also, much needs to be done in the direction of waste prevention, both to reduce the immediate cost of operation and in many places to render it possible to install purification works at a reasonable expense.

VALUE AND IMPORTANCE OF A PUBLIC WATER-SUPPLY.

13. **Domestic Use.**—The most important use of a public water-supply is that of furnishing a suitable water for domestic purposes. The absolute necessity of a supply of some sort for such purposes in a large city is well appreciated, but the value of purity is, by many, not rated as high as it should be. The transmission of certain diseases

such as cholera and typhoid fever by polluted water is now universally recognized, and the value to a city of a pure supply when compared to one constantly polluted by sewage can scarcely be overestimated. Many examples of the benefits arising from the introduction of new or improved supplies are given in Chapter X.

A public supply of pure water is of great value not only in large cities, but in the smaller towns and villages. Too often a supply for a village is designed with almost exclusive reference to fire-protection, and little attention is paid to the quality of the water, the people expecting to depend on wells as before. As a rule, however, a good pure water is quite as much to be desired in this case as for a city supply, and, if provided, will in many cases be quite as fully utilized.

Another highly important function of a water-supply is that of furnishing the necessary flushing-water for a sanitary system of drainage. The most satisfactory and economical method yet found for disposing of the organic wastes of a community is by the water-carriage system. Such a sewerage system is manifestly of but slight value to the public at large without the coexistence of a public water-supply, as otherwise the necessary water for the flushing of closets—the most important function of a sewerage system—can be afforded by but few.

Besides furnishing an improved supply from the sanitary standpoint, a public works may often be made to furnish a water which for other reasons will be of greatly increased value to the domestic consumer; such as a soft water in place of a hard well-water—a point of very considerable importance to both domestic and commercial users.

14. Commercial Uses.—The commercial value of a good water-supply is appreciated when one considers the large number of manufacturing interests which require for their operation large quantities of suitable water. Such establishments as sugar-refineries, starch-factories, bleaching and dyeing houses, breweries, chemical works, and various other factories require an abundant water-supply, and in some cases a water of a high degree of purity. The question of water-supply indeed often determines the location of such factories. Large quantities are also used for operating elevators, for boiler purposes, and for many other uses that may be classed as commercial.

15. Public Uses.—The most important public use of a water-supply is perhaps in extinguishing fires. The economic value of a good fire-protection system is directly shown in the reduced rates of insurance which follow its introduction or improvement. Instead of distributing a heavy fire-loss among the people of a community through high rates of insurance it is assuredly much better economy to contribute to the maintenance of a public water-works, which at the same time provides a suitable water for other purposes. To permit of the establishment of certain classes of factories it is absolutely essential that an efficient fire-protection be furnished.

Other important public uses of a water-supply are in street-sprinkling and sewer-flushing, in furnishing water for public buildings, and for drinking and ornamental fountains. A real value exists in the improved appearance which may be given a city by the use of water in fountains and for lawns and public parks; and indeed all the benefits accruing from a good water-supply act indirectly to increase the desirability of a town for many purposes and to enhance the value of the property therein.

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PART I.

REQUIREMENTS AND RESOURCES.

A. QUANTITY OF WATER REQUIRED: SOURCES OF SUPPLY.

CHAPTER II.

QUANTITY OF WATER REQUIRED.

16. Nature of the Problem. — One of the first questions to be answered when a new or enlarged water-supply is under consideration is that relating to the quantity which will be required when the works are completed, and for a certain period in the future. In the nature of the case this problem can be solved only approximately. Since the total quantity consumed is sure to increase in the future, the chief effect of an error in the estimate will be to vary the date at which an enlargement of the capacity will be required; but even so, to secure the most economical construction it is necessary that as close an estimate be made as possible.

In estimating consumption there will arise two cases:

(1) The case of a town being supplied for the first time;

(2) The case of an enlargement of an old supply.

In the first case an estimate of the immediate future consumption must be made by a study of the consumption of towns of similar characteristics, taking into consideration the various modifying influences. In the second case the consumption is already known, and that for a few years in the future can be readily estimated. In both cases, estimates for long periods ahead, such as twenty or thirty years, are very uncertain. To be of any value they must be based upon a careful study of the circumstances affecting increase in population and the use of water.

Estimates of consumption should include not only the average quantity which will be required, but also the variation in the consumption, in order that the various parts of the works—the reservoirs, pumps, and distributing system—may be properly proportioned.

17. Consumption, How Stated.—Consumption is usually stated in terms of the average daily consumption per capita throughout the year on the basis of the total population of the town or city. In large cities the total population corresponds nearly to the number of consumers, but in small towns and villages only a small percentage of the inhabitants may be users, and the statistics for such places are of little value unless the number of takers or taps is also given.

The amount consumed is determined in various ways. Where pumps are used it is obtained by multiplying the number of strokes made by the pumps by the displacement of the plungers, no allowance ordinarily being made for slip. The resulting error will not usually exceed 2 or 3 per cent, and is not of great consequence in this connection, but occasionally, as in the case of leaky suction-mains or well-tubes, large quantities of air are pumped and the "slip" becomes very great. In gravity works, the water is more or less accurately measured by weirs, or by the known capacity of certain pipes or conduits, or is merely guessed at.

In whatever way determined, the total amount is stated as the consumption. It therefore includes all water supplied, whether used, or wasted, or lost through broken pipes or mains. Sometimes, also, it includes water used in the condenser of the pumping-engine in cases where it should be deducted.

18. Influences Affecting the Consumption per Capita.—One of these influences is the number of inhabitants in the town or city. This element affects the per capita consumption chiefly by affecting the extent to which use is made of private sources of supply. Thus in large cities the use of the public supply is almost a necessity, while in small towns and villages the private supplies may remain in use to a large extent long after the introduction of the public supply.

The nature of the industries of a town is a large factor in determining the amount of water used; also the wealth and habits of the people, and the extent to which water is used for fountains, watering of lawns, street-sprinkling, and other public purposes. Climate has also a very considerable influence, especially as to the amount used for sprinkling purposes and that which is wasted in winter to prevent freezing. It is probable, however, that the most important factors in determining the consumption is the degree of care taken to detect leakage or waste, and the fact as to whether the water is sold by measure or otherwise. Good quality, abundant quantity, and high pressure tend to increase the consumption by encouraging a more liberal use and often, at the same time, greater wastefulness.

In many cases the introduction of a new or an improved water-supply is followed by such an increase in consumption that the time comes sooner than expected when the new works are no longer adequate to supply the demand. When estimating the probable consumption under the second case, i.e., the enlargement of an old supply, it is necessary then that the figures relating to the old works be used with considerable caution. Important changes in the character of a city sometimes also occur, and with small towns such changes may take place very rapidly. These, however, can scarcely be predicted.

19. Consumption of Water for Various Purposes.—In order to make an intelligent application of data pertaining to the use of water, some knowledge is desirable of the consumption for various purposes. This information is especially useful in the design of works for places of peculiar characteristics, in the design of the different parts of a distributing system, and of separate supplies for different purposes. Unfortunately but little accurate information relating to the consumption of water for different purposes is to be had, as the use of meters for all consumers is of rare occurrence.

The different uses of water may conveniently be divided into four general classes: (1) Domestic use; (2) Commercial use; (3) Public use; (4) Loss and waste.

Probably the best analysis yet made of the subject of water-consumption for different purposes is that by Brackett,* and in the following discussion his paper has been freely drawn upon; other data are taken from various city reports.

20. *Domestic Use.*—The following table, mainly from Brackett, gives a good notion of the actual quantities used for domestic purposes and the variation in the consumption due to differences in the character of the population. The figures are from metered supplies and represent what may be considered as legitimate consumption, even though considerable water may have been wasted.

The consumption per capita is seen to vary from 6.6 to 59 gallons per day for the lowest and highest class of dwellings respectively; and the average for a town varies from 11.2 gallons for Fall River, a manufacturing city, to 44.3 gallons for Brookline, a wealthy suburb of

* Trans. Am. Soc. C. E., 1895, XXXIV. p. 185. See also Jour. New. Eng. W. W. Assn., June, 1904, p. 107.

Boston. From these data it would appear that for a metered supply the domestic use may easily vary from 15 to 40 gallons, but that an allowance of 20 to 30 gallons would in most cases be abundant.

TABLE No. 1.

CONSUMPTION PER CAPITA FOR DOMESTIC PURPOSES AS DETERMINED BY METER MEASUREMENTS.

City.	Number of Persons.	Consumption per Capita in Gallons.	Remarks.
Boston, Mass.....	1,461	59	Highest-cost apartment-houses in city.
" "	8,432	32	Moderate-class apartment-houses.
" "	1,844	16.6	Poorest-class apartment-houses.
" "	1,699	46.1	Boarding-houses.
Brookline, Mass..	4,140	44.3	Average of all dwellings supplied by meter.
Newton, Mass....	2,450	26.5	All houses supplied with modern plumbing.
" "	3,005	6.6	These families have but one faucet each.
Fall River, Mass..	170	25.5	The most expensive houses in the city.
" " " ..	70,000	11.2	Average of all.
Worcester, Mass..	90,942	16.8	Whole domestic consumption.
" " ..	187	23.4	Cedar Street, best class of houses.
" " ..	809	15.6	Austin Street, cheaper houses.
London, Eng......	8,183	25.5	Houses renting from $250 to $600 each, having bath and two water-closets.
" "	5,089	18.6	Middle class, average rental $200.
Yonkers, N. Y....	34,000	20.6	Average of all.
Madison, Wis.....	13,000	21.3	Total domestic and commercial use.

With an unmetered supply the domestic consumption and waste may be many times greater than the figures given above. In Boston the estimated actual domestic consumption, including waste, was in 1892 (for the Cochituate works) 62.24 gallons per capita out of a total of 94.93 gallons. In Philadelphia, a city having an unmetered service, meters were placed experimentally upon the services of twenty residences in different parts of the city. The consumption for four of these services averaged 149 gallons per head per day, the highest rate being 181 gallons. In several other cases the rate averaged from 40 to 60 gallons, while in some it was as low as 9 gallons. In 1893, 142 houses were inspected and the average consumption found to be 222 gallons per capita.

21. *Commercial Use.*—Under this head are included all uses for mechanical, trade, and manufacturing purposes. Large users of water for such purposes are office buildings and stores, hotels, factories, elevators, railroads, breweries, sugar-refineries, and a few other industries. In 1892 the consumption in Boston for various commercial purposes as determined mostly by meters was as follows:

Office-buildings and stores, gals. per head for total population.......	11.17
Steam-railroads, " " " " " "	2.26
Sugar-refineries, " " " " " "	1.70
Factories, " " " " " "	2.15
Breweries, " " " " " "	0.89
Steamers and shipping, " " " " " "	0.90
Elevators and motors, " " " " " "	2.95
Saloons, " " " " " "	1.16
Hotels, " " " " " "	1.62
Miscellaneous, " " " " " "	5.47
Total " " " " " "	30.27

Similar statistics for 1880 indicated a consumption of about 25 gallons per capita. At Syracuse, N. Y., in 1888–89, 7.2 gallons per capita were used in operating elevators and 23.2 gallons for other commercial purposes. In New York City the consumption for commercial purposes is about 24 gallons per capita. Mr. Brackett considers that 35 gallons per capita should be allowed in making provision for the future supply of Boston.

In smaller cities the consumption for commercial purposes would in many cases be much less, while in some it might be more. In Fall River, for example, in 1892 the commercial consumption was estimated at 2 gallons per capita, this low value being due to the fact that most of the factories at that place get their supply directly from the river. In Yonkers, N. Y., a fully metered town (population 34,000), the consumption for commercial purposes was, in 1897, 27.4 gallons per capita, the total being 102 gallons. Considering the above data, it is probably fair to estimate the consumption for commercial purposes at from 5 to 35 gallons per capita according to the nature of the town.

22. *Public Use.*—This includes the water used for schools and other public buildings, street-sprinkling, water-troughs and fountains, sewer-flushing and the flushing of water-mains, fire-extinguishment, and a few other occasional uses. Water for such purposes is seldom measured, but the amount is not likely to exceed on the average a few gallons per capita, although the rate of consumption is far from being uniform. In the following table is given the consumption for various public purposes in Boston for 1892, and in Fall River for 1899, the water being in both cases partly metered and partly estimated.

	Boston.	Fall River.
Public buildings, schools, etc., gals. per capita........	2.30	1.36
Street-sprinkling, " " "	1.00	1.02
Sewer-flushing, " " "	.10	.48
Water-troughs and fountains, " " "	.25	1.91
Fires, " " "	.10	.11
Blowing off dead ends, " " "	...	.33
Miscellaneous, " " "	...	.36
Total, " " "	3.75	5.57

In many places much more water is used for sprinkling purposes than the quantities given above. Estimates for a few places are as follows: In Minneapolis, in 1897, 5 gallons per capita; in Indianapolis, 3 gallons; Rochester, N. Y., 3 gallons; Newton, Mass., 4 gallons; Madison, Wis., 10 gallons.* Street-sprinkling is carried on for about half the year only, so that the actual rate of consumption is about double these figures. Lawn-sprinkling in public parks would add very little. Assuming an amount for this purpose equal to $\frac{1}{10}$ inch in depth per day, and allowing 10 acres for each 25,000 inhabitants, the average used would be equal to about 1 gallon per head per day for the period of two or three dry months.

For fire purposes the average consumption is very small, but at times the rate is very high. (See Art. 32.)

Few American cities use any considerable quantity of water for ornamental purposes, and it might be well to consider whether a part of the large amounts wasted in some of our cities might not be more advantageously used in adding to the attractiveness of public parks and squares by means of ornamental and drinking fountains. The amount of water used in some of the ornamental fountains in the European capitals is at times very large, but does not add greatly to the average consumption. In Paris the average is estimated at only about 2.4 gallons per capita daily, although there are many fountains using from 4 to 100 gallons per second. These, however, are allowed to play only at certain hours or on special occasions.

The total consumption for public purposes may finally be estimated at from 3 to 10 gallons per head, averaging perhaps 5 gallons, the amount depending largely on the item of street-sprinkling.

23. *Loss and Waste.*—The enormous quantities of water (150 to 300 gallons per head per day) used by some of the large cities of the United States, when compared with the foregoing data from metered supplies, indicate that a very large percentage of the water furnished is lost through leakage or is wasted by the consumer. The chief causes of such waste are bad plumbing, leaky mains, waste to prevent freezing, and willful or careless waste. The waste by the domestic consumer has already been considered under domestic consumption. With metered supplies, water may still be badly wasted by the consumer, but such being paid for at regular rates, it must be considered as legitimate consumption. But when all services are metered and a liberal allowance is made for public uses, there is still a large amount of water apparently furnished which is not accounted for.

~~This discrepancy or loss is due to three causes: errors in meters,~~

* Boston Met. Dist., 1901, 2.13 gal. See table in Jour. New. Eng. W. W. Assn., June, 1904, p. 126.

This discrepancy or loss is due to three causes: errors in meters, errors in estimating the pumpage due to the slip of the pumps, and actual loss through leaks and breakages. Meters, when old, will tend to register less than the true amount, especially when measuring small quantities; furthermore, the actual amount pumped is nearly always less than that figured from plunger displacement, and to correct this error an insufficient allowance, or no allowance at all, may be made. Both these errors act to increase the apparent loss. Probably their combined effect will rarely be less than 5 per cent of the total amount pumped, and may easily reach 10 per cent. The actual loss is, therefore, often considerably less than the apparent loss.

In the following table * are given data showing the amount of water unaccounted for in certain cities where all or nearly all water used is metered. The use for public purposes has been taken into account so that the amount unaccounted for represents closely the leakage and errors of measurement.

The towns of Milton and Belmont, Mass., belong to the Boston Metropolitan district, and receive their water through Venturi meters. All consumers are also metered. The water unaccounted for amounts in these places to from 2000 to 5000 gallons per mile of pipe.

City or Town.	Population 1900.	Total Consumption Gallons per Consumer.	Per cent of Taps Metered.	Unaccounted for.		
				Per cent.	Gallons per Consumer.	Gallons per Day per Mile of Pipe.
Ware, Mass.	8,263	44.0	100.0	39.8	17.5	11,200
Wellesley, Mass.	5,072	50.0	100.0	41.5	20.8	3,450
Yonkers, N. Y.	47,930	89.0	100.0	40.7	45.7	23,340
Fall River, Mass.	104,860	40.5	96.0	21.5	8.5	10,000
Worcester, Mass.	118,420	68.0	94.5	46.5	31.6	20,800
Brockton, Mass.	40,063	36.0	90.0	33.8	12.2	6,200
Woonsocket, R. I.	28,204	28.6	86.7	23.0	6.6	4,370

24. Leakage from mains has been directly determined in several cases. Tests of comparatively new pipe systems indicate a leakage of from 500 to 1200 gallons per day per mile, and one engineer specifies a maximum allowable leakage of 60 to 80 gallons per mile per inch of diameter of pipe.† Certain tests of pipes in several German and Dutch cities showed leakages of less than 300 gallons per mile.‡ A

* Jour. New Eng. W. W. Assn., June, 1904, p. 132.

† Trans. Am. Soc. C. E., 1897, XXXVIII. p. 30.

‡ *Jour. f. Gasbel. u. Wasservers.*, 1894, p. 722.

test of a 24-inch main by Mr. Brush * showed a leakage of 6400 gallons per day per mile, under a pressure of 110 pounds per square inch. In large systems, cases of breakages of 4- and 6-inch mains have occurred which have remained long undiscovered, the water flowing away through adjacent sewers at rates as high as 100,000 gallons per 24 hours. In 1902 the amount supplied to Stoneham, Mass., was reduced from 800,000 gallons per day to 330,000 gallons by the repair of four large leaks in the street mains, which had been discovered by special investigation. During the same year eight leaks in the Boston works were found to be wasting about 650,000 gallons per day.

Pipe-leakage is likely to increase as the system gets older, on account of the loosening of joints through settlement, increased leakage of valves, etc. As a general estimate Mr. Kuichling † uses the values of 2500 to 3000 gallons per mile of pipe. This is equivalent to from 3 to 10 gallons per capita, the population per mile of pipe usually ranging from about 300 to 1000.

Considering these various facts, the total amount of water lost or unaccounted for in metered supplies may be placed at from 15 to 30 gallons per capita.

25. Total Consumption per Capita. — Recapitulating the above estimates for various purposes, we have, as reasonable extreme and average values for those supplies having a fairly good meter system :

Use.	Gallons per Capita. Daily.		
	Minimum.	Maximum.	Average.
Domestic	15	40	25
Commercial	5	35	20
Public	3	10	5
Loss	15	30	25
Total	38	115	75

As it will seldom occur that for any given place the conditions are all favorable for a minimum or a maximum use for all purposes, the above totals are to be considered as much more extreme figures than the separate items.

For the Boston Metropolitan district the result of a careful analysis of data by Brackett places these figures as follows: Domestic, 25 gal-

* Trans. Am. Soc. C. E., 1888, IX. p. 89.

† Trans. Am. Soc. C. E., 1897, XXXVIII. p. 19.

lons; commercial, 23.5 gallons; public, 7 gallons; loss and waste, about 65 gallons.*

TABLE NO. 2.

CONSUMPTION OF WATER IN AMERICAN CITIES AND TOWNS IN 1890 AND 1905.

City.	Population. 1900.	Population per Tap. 1890.	Per cent of Taps Metered. 1890.	Consumption per Inhabitant Daily. 1890.	Per cent of Taps Metered. 1905.	Consumption per Inhabitant Daily. 1905.
Chicago	1,698,600	7.1	2.5	140	...	200
Philadelphia	1,293,700	6.1	0.3	132	1.0	230
St. Louis	575,200	11.8	8.2	72	7.0	92
Boston	560,900	6.6	5.0	80	5.0	151
Cleveland	381,800	8.7	5.8	103	68	137
Buffalo	352,400	6.3	0.2	186	3	324
San Francisco	342,800	9.9	41.4	61	21	96
Cincinnati	325,900	8.5	4.1	112	12	130
Detroit	285,700	5.1	2.1	161	9	188
Milwaukee	285,300	11.1	31.9	110	80	91
Louisville	204,730	11.9	5.9	74	8	81
Minneapolis	202,720	16.5	6.3	75	47	76
Providence	175,600	9.4	62.4	48	86	68
Indianapolis	169,160	35.6	7.6	71	10	82
Kansas City	163,750	...	...	...	38	73
St. Paul	163,065	12.7	4.2	60	38	56
Rochester	162,600	5.4	11.4	66	41	88
Toledo	131,820	18.6	9.4	72	70	75
Columbus, O.	125,560	11.5	6.4	78	76	110
Worcester, Mass.	118,420	8.9	89.4	59	95	75
Fall River, Mass.	104,860	14.9	74.6	29	97	42
Memphis, Tenn.	102,320	11.9	3.7	124	20	100
Lowell, Mass.	94,970	9.2	22.9	66	69	58
Atlanta, Ga.	89,870	20.0	89.6	36	100	65
Dayton, Ohio	85,333	20.0	3.8	47	70	70
Nashville, Tenn.	80,870	14.9	0 8	146	52	148
Camden, N J.	75,940	...	...	131	3	155
Yonkers, N. Y.	47,930	12.0	82.4	68	99	115
Newton, Mass.	33,587	5.5	67.4	40	86	58
Aurora, Ill.	24,147	8.2†	19.3	40.7	36.†	56†
Madison, Wis.	19,164	11.0	31.0	40	97	46
Ashland, Wis.	13,074	9.9†	2.8	90	...	81
Champaign & Urbana, Ill.	14,826	7.3†	2.5	43†	...	45
Chippewa Falls, Wis.	8,094	7.4†	6.6	13.8	...	100
Middleborough, Mass.	6,885	11.7	24.0	21	47	38
Beloit, Wis.	10,436	10.2†	10.0†	64†	...	130
Foxborough, Mass.	3,266	8.7	34.0	44.0	46	63
Clinton, Ill.	4,452	4.1†	3.0	27.0	1.1†	99†
Shenandoah, Ia.	3,573	15.5†	15.5†	39†	...	35
Melrose, Mass.	12,962	4.2	1.7†	71†	3	112

40

26. In Table No 2 are given data concerning consumption and the use of meters in various cities for 1890 and 1905, complied mainly from the Manual of American Water-works for 1890, and from a paper by

* Jour. New Eng. W. W. Assn., June, 1904, p. 127.

† 1895.

Bailey containing statistics for 136 large cities.* The very considerable increase in consumption in nearly every city during the period from 1890 to 1905 is noteworthy. In some cases this increase is evidently much beyond any legitimate increase in demand. The great increase in the use of meters is also noteworthy.

For cities above 25,000 inhabitants the size has no apparent relation to the consumption. This fact is more clearly shown by the average consumption for groups of cities of different size. Mr. Kuichling † finds for 100 of the largest cities in the United States and Canada the following averages for 1895:

For Cities of a Population of	Consumption per Capita.
1,000,000 and more	106 gallons.
600,000 to 300,000	122 "
300,000 to 100,000	106 "
100,000 to 50,000	105 "
50,000 to 30,000	105 "

The large value for the second group is due to the high consumption of 220 gallons for Pittsburg. For towns smaller than the above the consumption is generally lower, partly on account of a less commercial use and partly because the water is used by only a portion of the community.

In a general way the effect upon consumption of the ratio of population to taps is observable for the various cities, but too many other elements enter to enable any definite relation to be traced. The great irregularity in consumption among the large cities, and the enormous quantities used by some, can be explained only on the supposition that a large part of the water is wasted and lost. The effectiveness of meters in preventing very high rates of consumption is clearly brought out by the table; for with two exceptions, no city having 20 per cent of its taps metered has a consumption appreciably above 100 gallons.

From statistics of the consumption for 1900 in 136 cities having a population exceeding 25,000 the relation of consumption to meters is roughly given by the following averages: ‡

Per cent of Taps Metered.	Average Consumption. Gallons per capita.
Less than 10	153
10 to 25	110
25 to 50	104
More than 50	62

27. Increase in Consumption.— For many years past there has been a large and steady increase in the consumption of water. This is due chiefly to the more general use of water and to an increase in the

* *Eng. News*, 1901, XLV. p. 279.

† Trans. Assn. C. E. of Cornell University, 1898, p. 10.

‡ *Eng. News*, 1901, XLV. p. 279.

number of fixtures in the houses supplied; but where no restriction has been imposed upon the use of water the waste has increased even faster than the legitimate use, so that in many cases the consumption has become enormously high.

To exhibit the general tendency the consumptions per capita for several large cities for the period from 1875 to 1900–05 have been plotted in Fig. 3. The curves for the cities of Chicago and Philadelphia

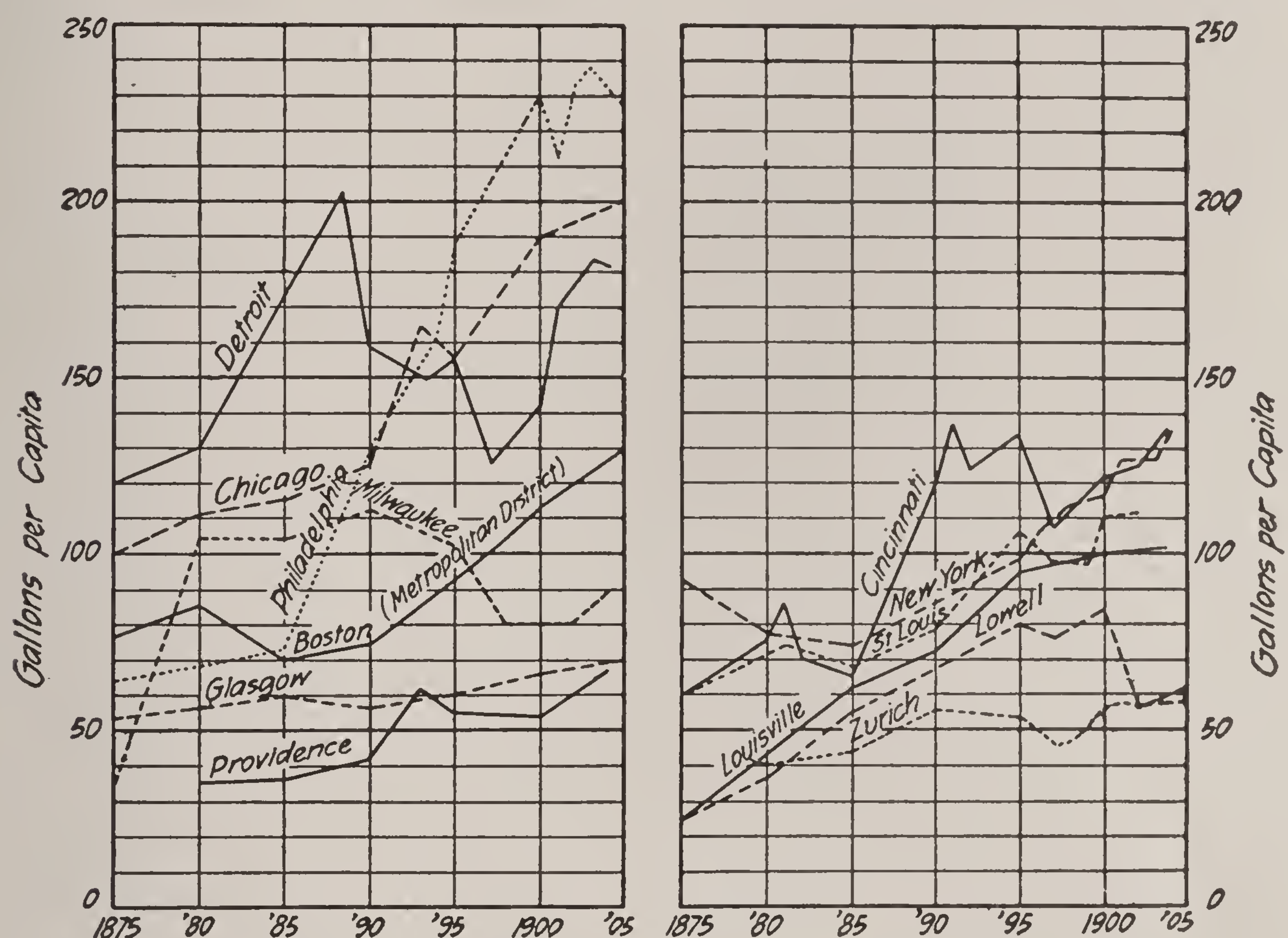

FIG. 3.—VARIATION IN YEARLY RATES OF CONSUMPTION.

show in what manner the unrestricted use of water is likely to raise the consumption. Omitting such cases of excessive rates of increase, there still remains a marked tendency towards an increased consumption of water. With originally low rates of consumption this increase is large even with well-metered cities, such as Providence, for example, with 82 per cent metered. This is also well shown in the figures of Table No. 2. Of the ten largest cities having over 50 per cent of taps metered in 1905, all but three showed a considerable increase in consumption, the average rate for these cities increasing from 65 gallons in 1890 to 78 gallons in 1905. The city of Milwaukee (Fig. 3) is a good example of the restraining effect of meters in a large city. About 80 per cent of the taps were metered in 1905.

Some of the cities, such as Boston and St. Louis, have good systems of inspection, and the consumption, though not excessive, is yet increasing at quite a high rate.

It would therefore appear that even with the best systems the per-capita consumption of water is likely to continue to increase for some time to come. In case the use of water is already restricted, it would not in general be safe to estimate the amount of this increase at less than 10 or 15 gallons for the next decade. Where few meters are used at present, the consumption could in many cases be greatly reduced by their introduction, or by a better inspection system.

The difficulty of estimating future consumption is illustrated by the case of Boston. In the investigation for the Metropolitan district, made in 1894, it was estimated that 100 gallons per capita would be sufficient for thirty years to come. As a matter of fact the consumption in the district increased from 83 gallons in 1893 to 129 gallons in 1905.

28. **Variations in Consumption.**—The foregoing articles have discussed only the average consumption throughout the year. There will now be considered the variations which occur in the consumption from time to time.

For the design of the different parts of the works it is desirable to know the monthly, the daily, and the hourly variations. The variations for periods of one month or more are of use in questions pertaining to large storage-reservoirs, while those for short periods of a few days or hours are of use in the design of pumps, service-reservoirs, and mains. For example, if no storage exists between the pumps and the consumer, then the pumps must be designed to furnish water at the maximum possible rate of consumption, while with a certain amount of storage they may be designed with only sufficient capacity to supply water at the maximum daily rate or at the maximum weekly rate. Likewise with no storage the source of supply, whether surface-water or ground-water, must have a capacity sufficient at all times to supply water at the maximum rate. With more or less storage the capacity of the source can be more or less reduced.

29. *Monthly Variations.*—In nearly all cases the rate of consumption reaches a maximum in the summer owing to the use of water for street- and lawn-sprinkling. This high rate usually extends over two or three months. A secondary maximum often occurs in the winter, due to the waste of water to prevent freezing, but the use of meters will largely prevent excessive variations from this cause. In extreme cases, however, the winter consumption may be very high. For example, during the severe winter of 1898–99 service-pipes quite generally froze in many places in the Northwestern States, and in some of these towns the waste of water to prevent further freezing raised the daily consumption to 300 or 400 gallons per capita for several weeks.

The occurrence of a large fire at such a time would be likely to prove disastrous. Such a contingency should, however, be met by using a more ample margin of safety in the depth at which the pipes are laid, and need only be considered to a slight extent as a possible element in causing high consumption.

The monthly variations in consumption for several places are illustrated by the curves of Fig. 4; and further data relating to monthly rates are given in Tables No. 3 and 3a.

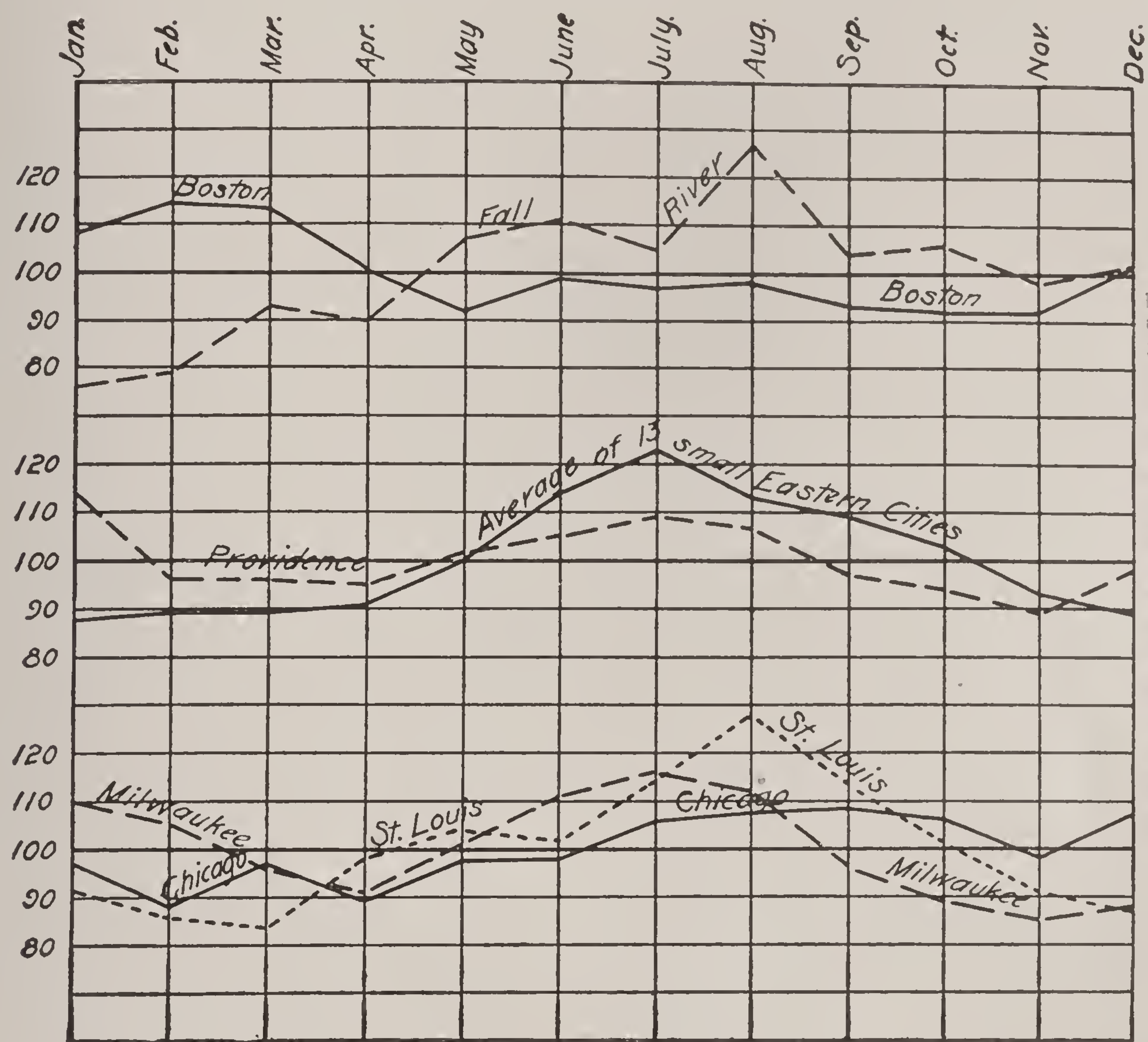

FIG. 4.—RATIOS OF MONTHLY TO AVERAGE CONSUMPTION.

From the diagrams and table it may be concluded that the maximum monthly rate will seldom exceed 125 per cent of the average, it being in fact much below this figure for most places represented. The diagram further shows that excessive consumption is likely to continue for two or three consecutive months, averaging for this longer period a rate of 110 to 115 per cent of the yearly average.

30. *Daily Variations.*—The maximum daily rate is usually estimated at about 150 per cent of the average. In the tables very considerable differences are to be noted in the ratios for different places, these being caused by a variety of conditions, some accidental and

TABLE NO. 3.

MAXIMUM MONTHLY AND DAILY RATIOS EXPRESSED AS PERCENTAGES OF AVERAGE CONSUMPTION.

City.	Ratio of Maximum Monthly to Average Consumption	Ratio of Maximum Daily to Average Consumption.	City.	Ratio of Maximum Monthly to Average Consumption.	Ratio of Maximum Daily to Average Consumption.
Chicago . . .	108	116	Louisville . . .	127	135
Philadelphia .	110	122	Columbus . .	107	157
Boston. . . .	114	119	Fall River. . .	115	. . .
Cincinnati . .	124	153	Dayton	118	178
Cleveland . .	114	146	Newton . . .	125	143
Buffalo . . .	. . .	168	Pawtucket . .	111	153
Detroit . . .	117	150	Woonsocket,R.I.	122	155
Milwaukee . .	113	. . .	Marquette, Mich.	139	194

TABLE NO. 3A.*

MAXIMUM MONTHLY, WEEKLY AND DAILY RATIOS FOR MASSACHUSETTS CITIES, EXPRESSED AS PERCENTAGES.

Averages for Six Years, 1895–1900.

City or Town.	Population, 1900.	Average Daily Consumption, per Capita, Gallons.	Percentages of Maximum to Average Consumption.		
			Monthly.	Weekly.	Daily.
Worcester (less than 6 years).	118,421	58	117	128	165
Fall River.	104,863	36	121	130	150
Lowell	94,969	79	117	130	150
Cambridge	91,886	81	113	138	180
Lynn and Saugus	73,597	67	116	125	177
Lawrence	62,559	55	111	134	164
New Bedford	62,442	96	113	126	151
Brockton	40,063	30	134	164	232
Salem	35,956	67	114	119	178
Taunton	31,036	46	116	127	147
Gloucester	26,121	32	129	142	193
Waltham (5 years only). . .	23,481	76	115	135	188
Brookline	19,935	85	124	160	184
Hyde Park and Milton . . .	19,822	42	146	147	166
Wakefield and Stoneham . .	15,487	53	124	127	182
Newburyport	14,478	41	114	128	157
Woburn	14,254	73	123	145	218
Beverly	13,884	70	140	163	222
Marlborough	13,609	37	119	119	220
Milford and Hopedale . . .	13,463	61	121	136	158
Peabody	11,523	89	114	127	155
Attleborough	11,335	36	130	154	245
Framingham	11,302	36	122	143	194
Gardner	10,813	62	125	128	169
Abington and Rockland . .	9,816	39	138	167	233

* From Mass. Bd. Health Report, 1900, p. 613.

some constant. For the larger cities the ratio of 150 per cent appears to be a fair maximum, but for the smaller cities the ratio is frequently over 200 per cent. Generally speaking, the lower the average consumption the greater the variation. The maximum daily rate will usually occur in the month of maximum consumption, and a rate considerably above the average for the month will obtain for several consecutive days. Thus where the maximum daily consumption is 150 per cent of the average, the maximum weekly consumption is likely to be about 130 per cent of the average, but for longer periods of time the rate will approach the monthly maximum.

31. *Ordinary Hourly Variations.*—If there were no waste or leakage, the consumption during several hours of the night would be almost nothing and the relative variations in consumption throughout the 24 hours would be very great. Whatever leakage exists is nearly constant and tends therefore to decrease these variations. During the summer, when the monthly rate is high, the hourly rate is also likely to be high, as the excessive use of water at that time of the year is largely due to lawn- and street-sprinkling, which usually occurs at a time of day when the consumption for other purposes is large. This results in a very high hourly rate. To prevent this excessive rate many towns have regulations requiring the sprinkling of lawns to be done at special hours when the demand for other purposes is somewhat lessened. The consumption in the winter, although it may be great, is more uniform throughout the 24 hours, as the waste at this time of year will be the greatest at night. In small cities the demand is likely to be more irregular than in large cities.

Measurements made in Boston in August, 1893, gave, for the Mystic works, the following rates of consumption for different portions of the day, expressed as gallons per head per day.

1 to 4 A.M.	40.8 gallons	1 to 4 P.M.	98.2 gallons
4 to 7 "	58.6 "	4 to 7 "	79.5 "
7 to 10 "	103.8 "	7 to 10 "	61.9 "
10 A.M. to 1 P.M. ...	93.0 "	10 P.M. to 1 A.M.	52.9 "
Average	73.6 gallons		

The maximum rate for 3 hours was thus 103.8 gallons, or 141 per cent of the average; and from 7 A.M. to 7 P.M. the rate was 127 per cent of the average for the day. Referring to Table No. 3 and assuming the variation on the day of greatest draught to be the same as here given, the maximum draught from 7 to 10 A.M. for the year would then be 141 per cent of 119 per cent = 168 per cent of the daily average for the year. The large consumption from 1 to 4 A.M. must have been mostly waste.

In Detroit the maximum hourly demand in 1894 and in 1895 was 178 per cent of the average yearly rate.

Mr. Coffin found for Attleboro, Mass. (population 7577 in 1890), a rate for the maximum month of 122 per cent of the average yearly rate, maximum week 134 per cent, maximum day 155 per cent, maximum hour of maximum day 333 per cent, maximum two continuous hours 312 per cent, and minimum hour 45 per cent. The average rate

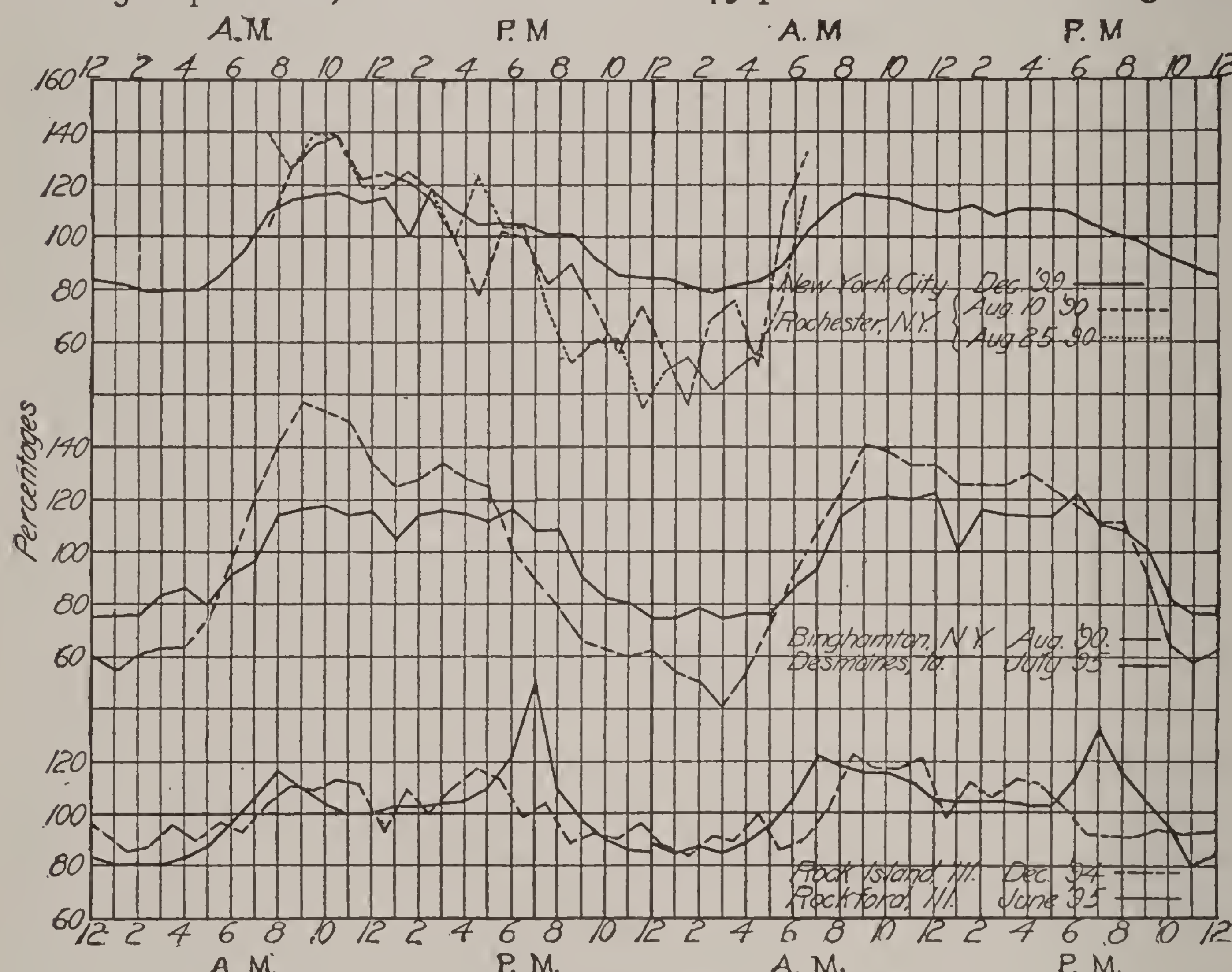

FIG. 5.—HOURLY VARIATIONS IN CONSUMPTION.

from 10 A.M. to 3 P.M. for three days in the month of maximum consumption was 230 per cent of the average.*

In Fig. 5 are plotted, for two days each, the hourly rates of consumption, expressed as percentages of the average hourly rate for the entire day, for the cities of New York City,† Rochester,‡ N. Y., Binghamton,‡ N. Y., Des Moines,§ Ia., Rockford,‖ Ill., and Rock Island,¶ Ill. For the city of Rockford the high consumption during sprinkling hours, 6 to 8 P.M., is notable, and for all places the large consumption during the night. The total per-capita consumption of these places was for 1895 approximately as follows: New York City, 100 gallons; Rochester, 71 gallons; Binghamton, 135 gallons; Des

* Trans. Am. Soc. C. E., 1897, XXXVIII. p. 27.

† Report of J. R. Freeman on the water-supply of New York City, 1900.

‡ Ogden. Sewer Design (New York, 1899). § *Eng. News*, 1896, XXXV. p. 130.

‖ Reports of city officers, 1895. ¶ Report by D. W. Mead, 1895.

Moines, 43 gallons; Rockford, 90 gallons; and Rock Island, 200 gallons. It will be noted that, in general, those places having the largest consumptions show the smallest percentage variation throughout the day. This is due to the excessive leakage and waste which occurs in these places, and which is nearly uniform.* If the maximum ratios of the diagrams be multiplied by the maximum daily ratio of, say, 150 per cent, there results for the maximum hourly ratios for the entire year the values 175, 210, 180, 238, 220, and 183 per cent, respectively. Regarding rates for longer periods than one hour it is to be noted from the diagram that a rate nearly equal to the maximum is likely to continue for 4 or 5 hours.

To illustrate the effect of temperature and precipitation upon the daily consumption and its variation, four diagrams of hourly consumption for Detroit are given in Fig. 5a.† Curve I represents the effect of extreme cold weather; curve II that of hot dry weather; curve III average conditions; and curve IV Sunday consumption.

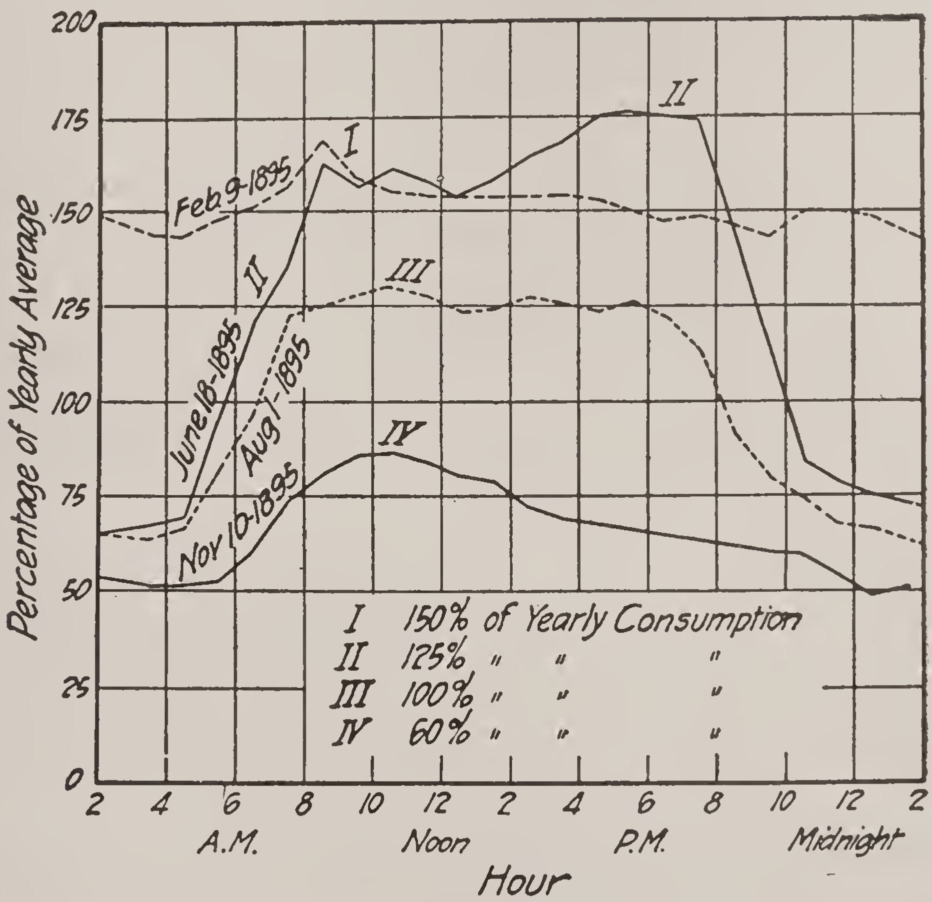

FIG. 5a. — HOURLY VARIATIONS IN CONSUMPTION, DETROIT, MICH.

32. *Consumption for Large Fires.*—Large fires occur but seldom, and in most of the statistics already given, especially those relating to hourly rate, it is safe to say that nothing more than an ordinary fire has been involved, such as would require much less than the maximum

* This point is well brought out by Mr. J. R. Freeman in his report on the water-supply of New York City, in which he shows graphically that the hourly rates of consumption of New York, Brooklyn, Fall River, and Woonsocket differ by nearly a constant quantity, although the average daily consumptions are widely different. (See *Eng. Record*, 1900, XLII. p. 103.)

† Trans. Am. Soc. C. E., 1901, XLVI. p. 413.

rate of supply. The consumption for large fires must then be considered in addition to the rates given above.

The maximum rate of fire consumption in gallons per capita per day for a town or city of average character may be taken equal to $\frac{1000}{\sqrt{x}}$, where x = population in thousands. This is based on Kuichling's estimate of the required number of fire-streams,* and assumes 250-gallon streams.

If the average consumption is 100 gallons per capita, then the fire rate in per cent of the average will be as follows for different-sized cities:

Population.	Rate of Fire Consumption in Percentage of Average, when Average equals 100 Gallons per Day.
1,000	1000 per cent.
5,000	447 " "
10,000	316 " "
50,000	141 " "
100,000	100 " "
200,000	71 " "
300,000	58 " "
500,000	45 " "

For other average values of the daily consumption the percentages would vary accordingly, being greater for smaller consumptions. In the case of small cities the fire rate is evidently the principal factor to be considered; in large cities it is of much less relative importance. The duration of the above rate of fire consumption may be several hours; it has been estimated by Freeman at about six hours as a maximum for the full number of streams.

33. *Maximum Hourly Rate.*—The chance of a large fire occurring at the same time as the maximum consumption for other purposes is exceedingly remote, so that in obtaining the probable hourly maximum some reduction may be made in the figure obtained by combining both maxima.

The maximum hourly rate for a city of 50,000 inhabitants, with 100 gallons per capita as the average consumption, may, for example, be estimated about as follows:

Maximum daily ratio = 175 per cent of average.

Max. hourly ratio of maximum daily = 150 per cent of 175 per cent
= 262 per cent of average.

Fire consumption . . . . . = 141 " " " "

Total . . . . = 403 " " " "

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* Trans. Am. Soc. C. E., 1897, XXXVIII. p. 16. For further discussion of this subject see Chapter XXVIII.

This total may be reduced to, say, 375 per cent of the average, or 375 gallons per day, as the maximum rate. It would not, however, be safe to assume a much lower rate, as the average daily for an entire month is likely to be 130 per cent of the average, which would give for the ordinary hourly maximum $130 \times 150 = 195$ per cent. Adding the fire demand, the maximum becomes 336 per cent, or 336 gallons per day.

34. **Consumption in European Cities.**—The consumption of water in European cities is much less than in American cities. This is due in part to the more general use of meters in Europe, but it is also undoubtedly true that water is used less lavishly and wastefully there than here. Moreover, in the United States much more water is lost by leakage, the pipes usually being much larger and in many cases probably not so well laid. It is believed, however, that a considerable part of the difference is due to a greater legitimate demand in this

TABLE NO. 4.

CONSUMPTION OF WATER IN EUROPEAN CITIES.

City.	Estimated Population.	Consumption per Capita Daily. Gallons.	City.	Estimated Population.	Consumption per Capita Daily. Gallons.
England, 1896–97: *			France, 1892 (Bechmann):		
London	5,700,000	42	Bordeaux	252,654	58
Manchester	849,093	40	Toulouse	148,220	26
Liverpool	790,000	34	Nantes	125,000	13
Birmingham	680,140	28	Rouen	107,000	32
Bradford	436,260	31	Brest	70,778	3
Leeds	420,000	43	Grenoble	60,855	264
Sheffield	415,000	21	Other countries, 1892–96 (Bechmann):		
Nottingham	272,781	24			
Brighton	165,000	43	Naples	481,500	53
Plymouth	98,575	59	Rome	437,419	264
Germany, 1890 (Lueger):			Florence	192,000	21
Berlin	1,427,200	18	Venice	130,000	11
Breslau	330,000	20	Zurich	80,000	60
Cologne	281,700	34	Geneva	70,000	61
Dresden	276,500	21	Amsterdam	515,000	20
Düsseldorf	144,600	25	Rotterdam	240,000	53
Stuttgart	139,800	26	Brussels	489,500	20
Dortmund	89,700	78	Vienna	1,365,000	20
Wiesbaden	62,000	20	St. Petersburg	960,000	40
France, 1892 (Bechmann):			Bombay	810,000	61
Paris	2,500,000	53	Sydney	423,600	38
Marseilles	406,919	202	Buenos Ayres	680,000	34
Lyons	401,930	31			

* Compiled, except the figures for London, by Hazen. *Eng. News*, 1899, XLI. p. 111.

country; a demand caused in some cases by a higher commercial consumption and in general by a larger domestic use due to the less economical habits of the people and to the use of a larger number of fixtures. In Table No. 4 is given the consumption per capita for several cities in various European countries.

The use of water for public purposes in seven German cities varied from 1 to 12 gallons per capita in 1888–1890, this being from 2 per cent to 33 per cent of the total consumption. In Berlin 2.5 per cent is used for street-sprinkling, 3 per cent for sewer-flushing, and 7 per cent for fountains. In Dresden 3.7 per cent of the entire consumption is used for public fountains.* In Paris 35 per cent, or 20 gallons per capita, is used for street-washing.

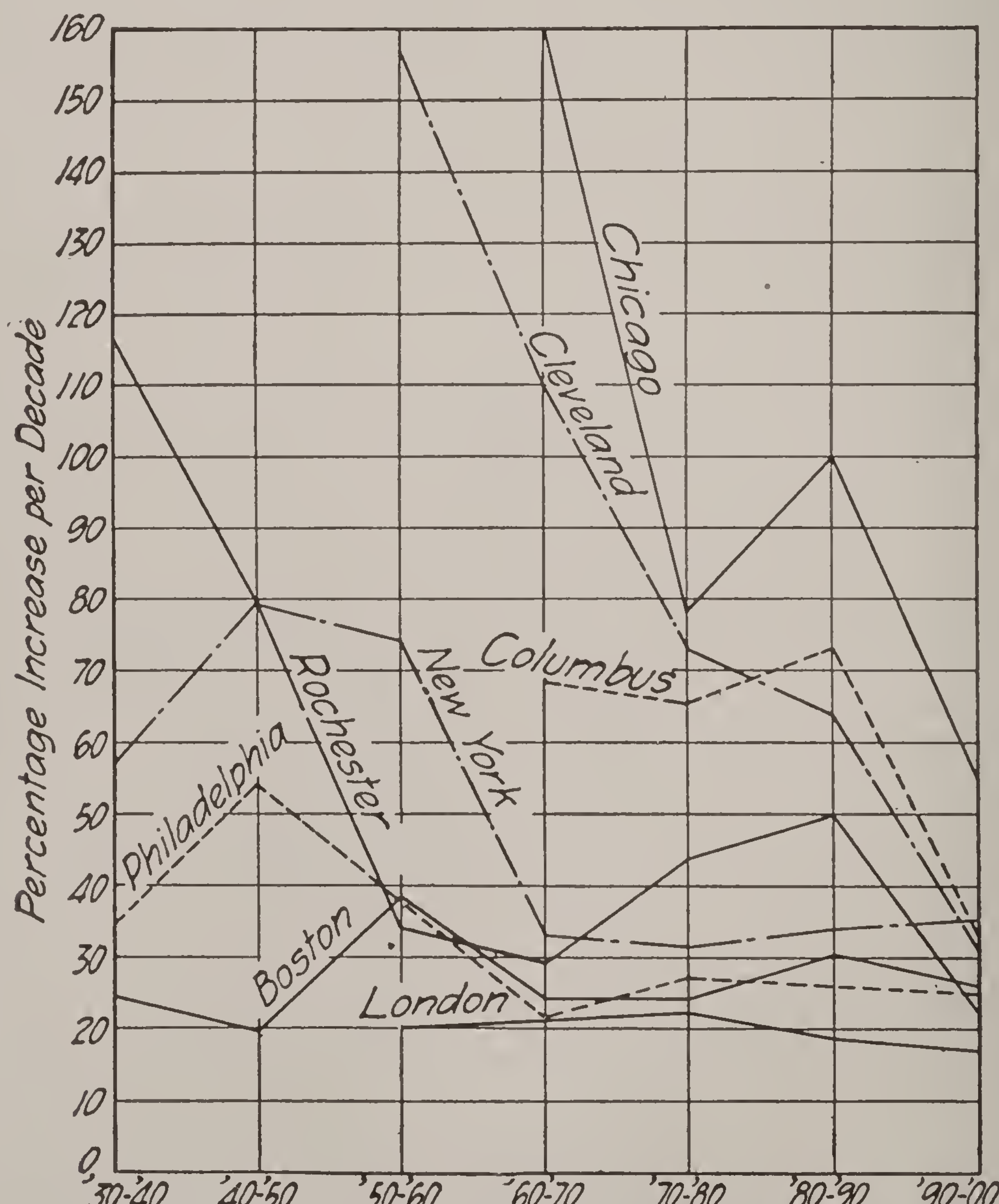

FIG. 6.—PERCENTAGE GROWTH OF CITIES.

35. **Growth of Cities.**—A necessary factor in any estimate of future consumption is that of future population. The rate of growth of different cities is exceedingly various, but of any one city it is likely to be

* Handbuch der Ingenieurwissenschaften, p. 69.

fairly constant for several years, or at least will vary but slowly. The older and the larger the city the more uniform the rate of growth, and, barring national disasters, a fairly close estimate can be made for two or three decades in the future. In the case of many American cities the rate is still undergoing large variations, and predictions are very uncertain.

For a city with a steady rate of growth the percentage added each decade or shorter period is very nearly constant; and to estimate the future population it is only necessary to apply this constant percentage successively for as many periods as desired. If the percentage is changing, then a varying rate must be used, which can only be predicted by a study of the changes in the rates in past years and a knowledge of such local conditions as are likely to affect the city's growth. To facilitate such estimates the percentage increase for each decade should be plotted, and any marked tendency to change can then be allowed for in extending the curve forward.

In Fig. 6 are plotted such percentages for several cities of differing characteristics. The percentages for London are remarkably constant,

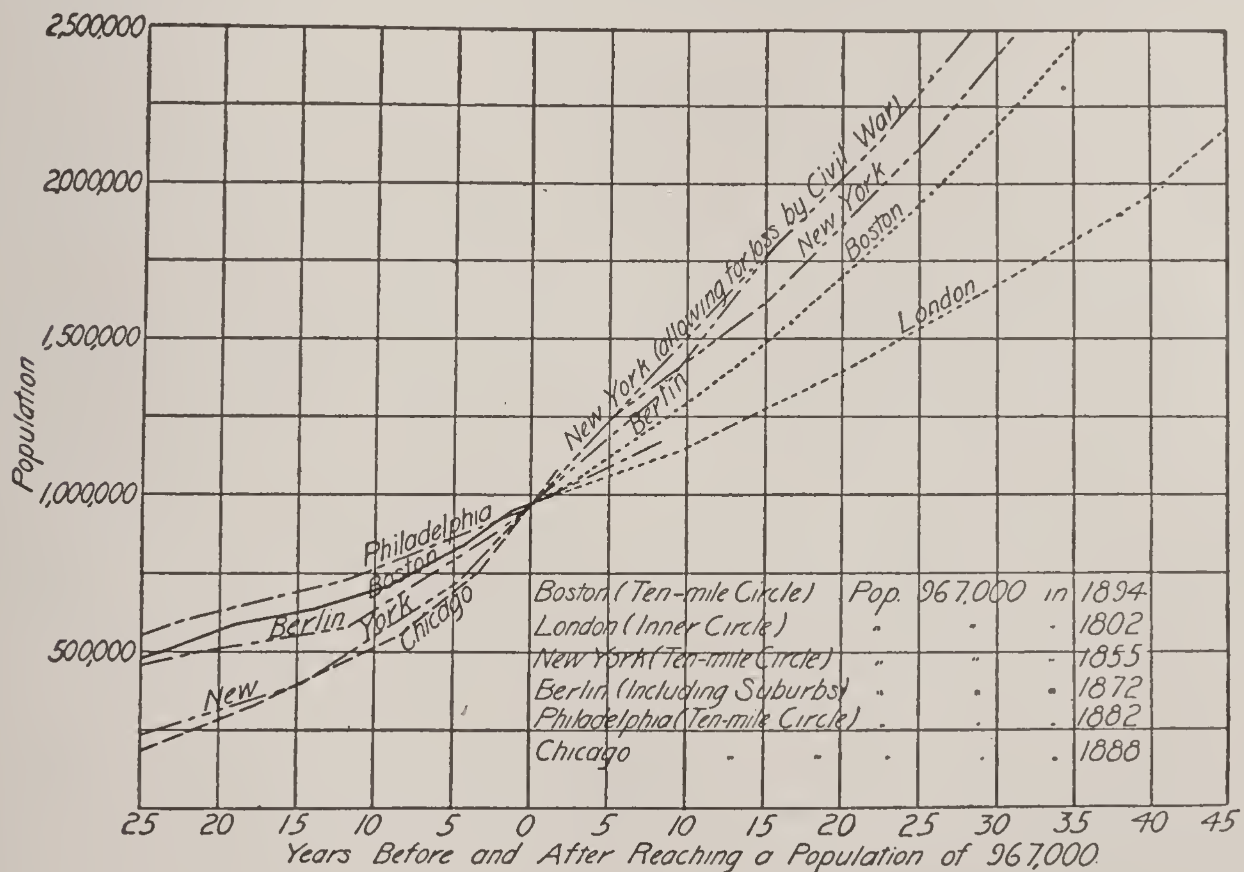

FIG. 7.—POPULATION CURVES PLOTTED WITH REFERENCE TO BOSTON, MASS.

and in estimates for the future a rate of 20 might reasonably be assumed. Several of the other cities have reached a nearly uniform rate, while in some the rate is still likely to undergo great changes. In estimating the population of London for forty years in the future the Royal Commission in 1893 used the percentage for the decade 1881 to 1891, a value of 18.2. The data for Boston, New York, Philadelphia, and Chicago are as compiled by Brackett in Appendix No. 1 of the Report of the State Board of Health of Massachusetts upon a Metropolitan Water-supply. They represent in each case the population within a 15 mile radius.

36. Another method of estimating future population is to study the growth of various larger cities from the period at which their population equaled the present population of the city in question; and, taking account of differing characteristics, to deduce therefrom the probable future population required. This method was used as an aid in predicting the future population of Boston in the report above referred to, and the diagram employed is reproduced in Fig. 7. It exhibits the curves for several cities, so plotted as to intersect at the point corresponding to a population of 967,000, the population of Boston in 1894.

An objection to this method of estimation is that it is based upon a comparison of rates of growth of cities of widely varying characteristics, and of rates relating to very different periods of time. Thus the growth of Boston in 1900 is compared with that of New York in 1860, when industrial conditions were materially different from those at the present time.

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CHAPTER III.

SOURCES OF SUPPLY.

37. Classification.—The sources of water-supply may be divided into the following classes according to the general source and the method of collection:

A. Surface-waters:
   1. Rain-water collected from roofs, etc.
   2. Water from rivers.
   3. Water from natural lakes.
   4. Water collected in impounding reservoirs.

B. Ground-waters:
   5. Water from springs.
   6. Water from shallow wells.
   7. Water from deep and artesian wells.
   8. Water from horizontal galleries.

Each of the above sources except the first and last are at present furnishing many cities in the United States with a more or less satisfactory water.

38. Quality of Water from Various Sources.—The kind of water which a region can furnish depends on its climatic, geologic, and topographic features. Much good water has been obtained from small streams in the rougher portions of the United States where sites for reservoirs can readily be found and where collecting areas are sparsely populated; but in a large portion of the country such a source of supply is impracticable or undesirable, and in these localities we find that the ground-water supplies have been more largely developed. Many supplies drawn from lakes and rivers are also in use in all parts of the country, but until some method of purification is generally adopted they will not be as a rule very satisfactory. These sources must, however, continue to furnish a large and increasing number of cities with water as the supplies from the first-mentioned source become more and more fully utilized.

Ground-waters are as a rule of better quality from a sanitary point of view than surface-waters, but in many cases they will not be altogether satisfactory until processes for the removal of iron and of hardening impurities are adopted.

39. Utilization of the Various Sources.—The following table gives the number of water-works obtaining their supply from the various sources indicated, and the percentage of the total number supplied from each source. Under Northeastern States are included Pennsylvania, New Jersey, and all to the north and east; North Central includes all others to the north of the Ohio River and east of the Mississippi River; Southeastern, all remaining States east of the Mississippi; and Western, all west of it.*

TABLE NO. 5.

SOURCES OF WATER-SUPPLIES IN THE UNITED STATES.

Source.	No. of Works.					
	Northeastern States.	Southeastern States.	North Central States.	Western States.	Total Number.	Per cent of Grand Total.
Surface-waters:						
Rivers	336	116	117	256	825	24.6
Lakes	129	6	62	24	221	6.6
Impounding reservoirs	135	20	12	46	213	6.3
Combinations	15	1	2	3	21	0.6
Total	615	143	193	329	1280	38.2
Ground-waters.						
Shallow wells	130	41	310	380	861	25.7
Artesian wells	39	59	98	145	341	10.1
Springs	300	72	27	103	502	14.9
Galleries and tunnels	9	0	12	13	34	1.0
Combinations	33	8	22	21	84	2.5
Total	511	180	469	662	1822	54.2
Surface- and ground-waters.						
Rivers and ground-waters	92	16	37	61	206	6.1
Lakes and ground-waters	11	0	16	4	31	0.9
Imp. reservoirs and ground-waters	9	1	0	7	17	0.5
Total	112	17	53	72	254	7.6
Grand total	1238	340	715	1063	3356	100.0

The number of filtered supplies in 1896 was as follows:

Surface-waters	179
Ground-waters	23
Surface- and ground-waters	29
Total	231

* From *Eng. News*, 1898, XL. p. 9, and Manual Am. W. W., 1897.

In Europe a much larger proportion of the public supplies is derived from ground-water sources than is the case in this country. In Germany, for example, in 1884, of the total population having a public water-works the following percentages drew their supply from the various sources indicated: *

River- and other surface-water	27.9 per cent.
Spring-water	13.8 " "
Other ground-water	58.3 " "

In France, out of a total population of about 12 millions living in cities of over 5000 inhabitants, the following percentages were, in 1892, supplied with water from the sources indicated: †

Rivers	14 per cent.
Springs	23 " "
Other ground-water	14 " "
Combinations	49 " "

* *Jour. f. Gasbel u. Wasservers.*, 1884, p. 411.
† Bechmann. Distribution d'eau (Paris, 1899), II. p. 330.

CHAPTER IV.

THE RAINFALL.

40. The rainfall being manifestly the source of all water-supply, whether caught as it flows over the surface or first allowed to percolate into the ground to furnish water for wells and springs, it is desirable to commence the discussion of the quantity available from the different sources with a study of the rainfall. The yield of a given source is the product of several factors, of which the rainfall is but one; and in many cases it is quite as easy or even easier to estimate the value of this product directly as to determine it from a consideration of the several factors. In other cases, however, this cannot be done, and to enable the data already collected regarding the various elements to be intelligently used in the solution of new problems, a careful study of each of these elements is necessary.

41. **Measurement of Rainfall.**—The amount of rainfall is expressed in inches of depth upon a horizontal surface, snowfall being reduced to

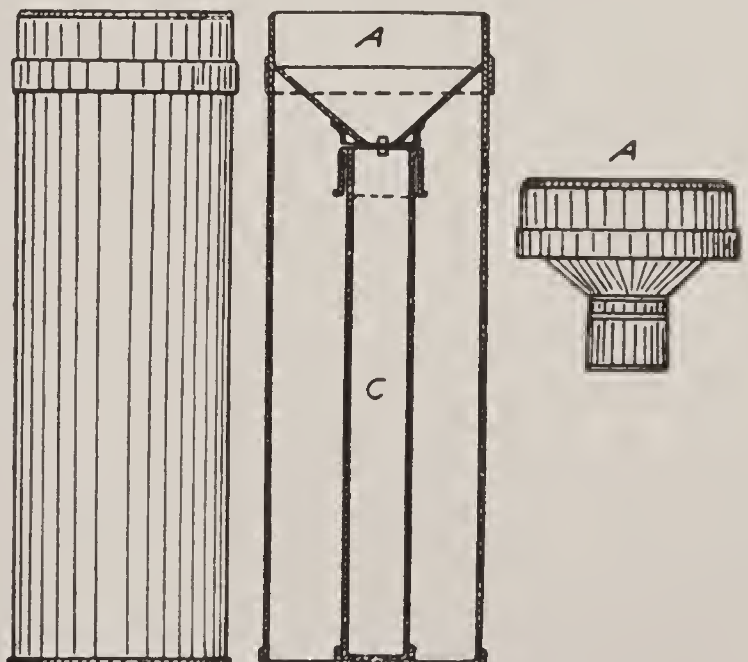

FIG. 8.—ORDINARY RAIN-GAUGE.

its equivalent amount of rainfall. The ordinary rain-gauge used by the Weather Bureau is illustrated in Fig. 8. The diameter of the receiver A is 8 inches, and the entire height of the instrument is 2 feet. The rim is beveled to a sharp edge and is accurately circular. The water which falls into the receiver is conveyed into the collecting-tube C, of one-tenth the cross-section of the receiver, and the amount of water so collected is determined by a measuring-stick of cedar. In this way small rainfalls can be readily measured. Large rainfalls overflow into the outer cylinder, which is also used as a collector for snow.

While the actual measurement is thus simple, the collection of the correct amount of water is not easy. It is found that the amount of water collected depends largely upon the location of the gauge. Variations as great as 50 per cent have been observed, due to differences in location in regard to buildings and other objects, and to the elevation of the gauge above the ground. In general the greater the elevation of the gauge the less the amount of water collected. The reason for this has been quite conclusively shown to be due to the greater velocity of the wind at the greater elevation, less water being collected the stronger the wind.*

The errors of collection due to wind eddies caused by buildings, trees, etc., are of much greater importance than those due to elevation, and to avoid these the gauge should be located some distance from all disturbing objects and not much above the ground. In cities, the best place is on the roofs of flat buildings, and this is the location usually selected by the Weather Bureau. Such locations, though free from disturbances caused by other buildings, are not as trustworthy as is desirable, and it is estimated by the Bureau that the amounts registered by its gauges are from 5 to 10 per cent too small.†

Besides inaccuracies due to exposure, there are slight inaccuracies in the measurement of small rainfalls in dry weather due to evaporation from the gauge, and very considerable errors in the measurement of snowfalls.

With the ordinary rain-gauge it is impracticable to determine rates of rainfall for short periods of time, the records usually obtained from these gauges being merely the total amounts of rainfall for each twenty-four hours. For estimating flood-volumes from small areas, however, it is important to know the rate of rainfall for much shorter periods than one day. For this purpose self-recording gauges are essential, that is, gauges which give a continuous record of the rainfall or a record taken at such short intervals as to be for all practical pur-

* See reference (2), p. 50.

† Bulletin D, Weather Bureau, 1897, p. 9.

poses continuous. Various forms have been devised, some weighing the water, others recording by volume.*

42. **Rainfall Statistics** for a large number of stations can now be readily obtained from the monthly reports of the Weather Bureau. Since 1888 observations relating to excessive rainfalls have been made with self-recording rain-gauges, the number of stations provided with such gauges in 1900 being about seventy. The data of importance in connection with water-supply questions are the mean yearly rainfall, the deviation from this in dry years, the monthly rainfall, and finally the maximum depth of rain falling in a single day or less.

43. **Mean Annual Rainfall.**—The mean annual rainfall and the principal drainage areas of the United States are shown in Fig. 9.† The maximum rainfall is seen to be along a narrow belt of the North Pacific coast, where it considerably exceeds 60 inches. Towards the interior the amount rapidly falls off, and between the Sierras and the Rocky Mountains it ranges from 5 to 15 inches. East of the Rockies there is a gradual increase eastward and southward to a maximum along the Gulf of 60 inches, and from 40 to 50 inches on the Atlantic coast.

44. **Secular Variations in the Rainfall.**—The question of a gradual change in the yearly rainfall is one the solution of which would doubtless require data covering several centuries. The rainfall for a particular locality may average considerably below the mean for many years, after which may follow, perhaps, an equally long period of surplus. In an analysis of several records extending over many years it was found that during an 83-year period at New Bedford, Massachusetts, the averages for 10-year periods were as high as 16 per cent above the mean and 11 per cent below; for 60-year periods the extremes were, at St. Louis, 17 per cent and 13 per cent, and at Cincinnati, 20 per cent and 17 per cent. For a 25-year period the extreme variations were 10 per cent for both New Bedford and St. Louis.‡

The variations or cycles above referred to, that extend over several years, are in some cases very marked, but they are very erratic and as yet quite incapable of being predicted. In Fig. 10 are plotted what are called progressive averages of precipitation for three sections of the country, and the actual precipitation for Madison, Wisconsin, for a number of years. The progressive averages for each section are found by first averaging the yearly rainfalls for three or four stations; then

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* For a description of various forms of self-recording gauges see references (7) and (9), p. 51.

† From a paper by John C. Hoyt in Trans. Am. Soc. C. E., 1907, LIX. p. 431.

‡ Bulletin D.

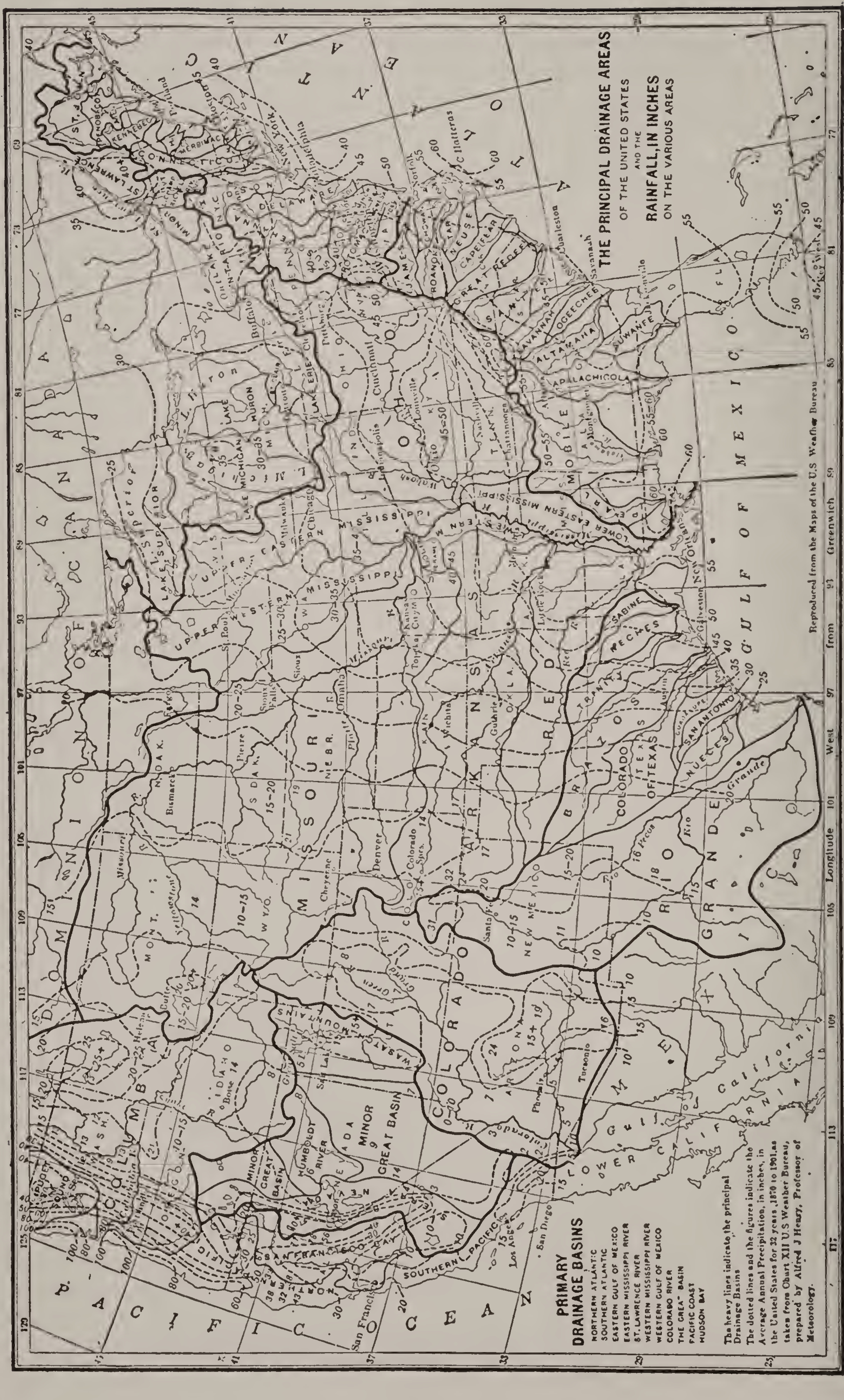

FIG. 9.— MEAN ANNUAL RAINFALL AND DRAINAGE AREAS OF THE UNITED STATES.
(From Trans. Am. Soc. C. E., vol. LIX.)

these averages are further modified to give a smoother curve by the formula

$$c' = \frac{a + 4b + 6c + 4d + e}{16},$$

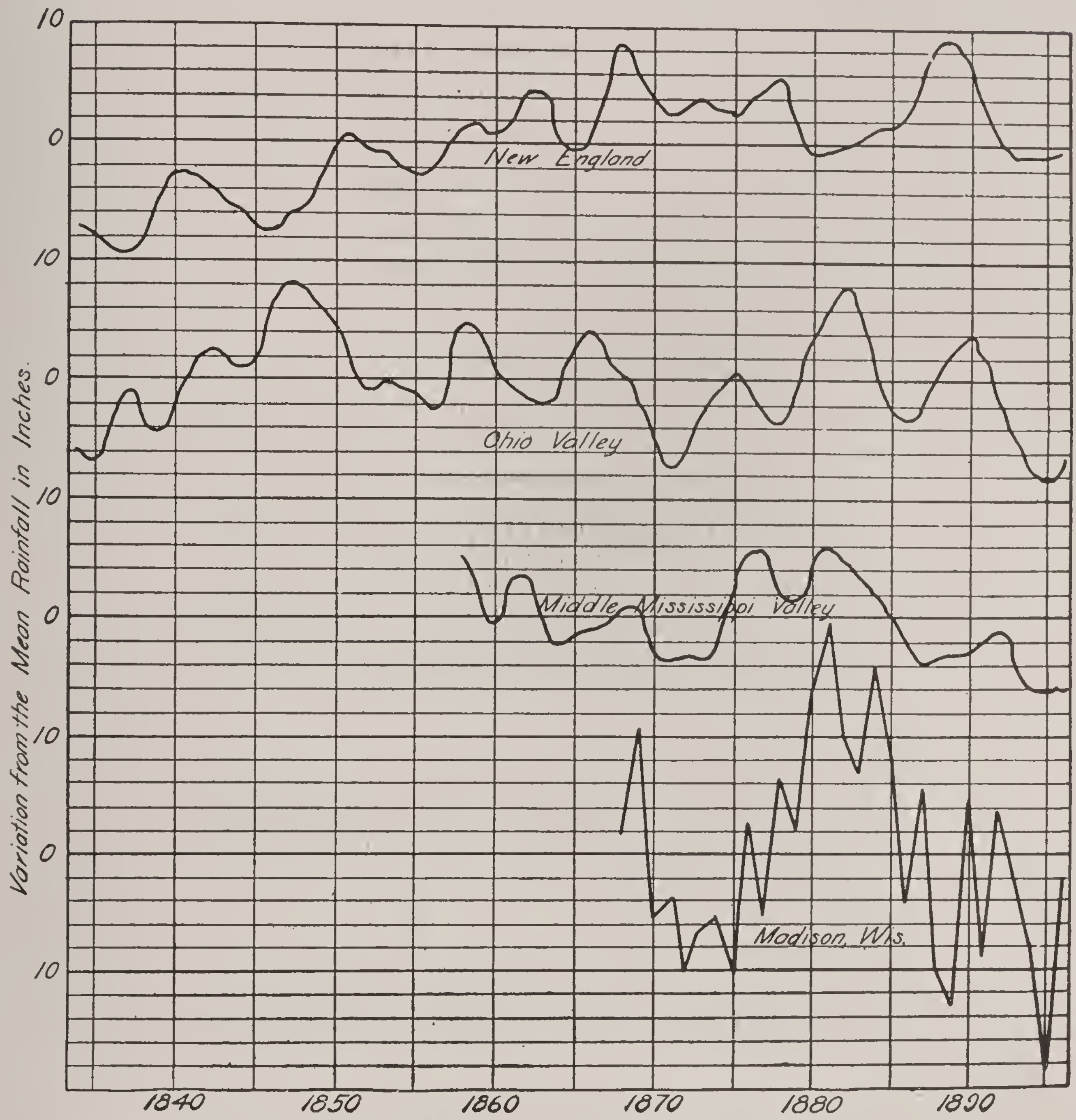

FIG. 10.—SECULAR VARIATIONS IN THE RAINFALL.

where a, b, c, d, and e are the rainfalls for successive years, and c' is the progressive average for the middle year. In this way any gradual change in the rainfall can be more clearly brought out. In the diagram the ordinates represent inches above or below the mean. The gradual increase for a long period of time in the rainfall at the stations representing New England is very striking, although this is shown by other records to be quite local in extent. Other changes for considerable lengths of time are to be noted in the diagram, and it is clearly to be seen that a record covering twenty or thirty years is of no value in studying the question of secular variation. The diagram for

Madison, Wisconsin, is of course very rough, but shows the same general variation as that just above it. If the portion of the curve for the years 1880 to 1895 alone be considered, a very rapid and persistent decrease in the rainfall would be noted.

45. **Mean Monthly Rainfall.**—The monthly distribution of the rainfall is of great importance in all questions relating to the utilization of water for power purposes or for the supply of cities. The rain falling in the summer months, when vegetation is using a maximum of water and evaporation is rapid, is of but little value for supplying water to the streams. It is the winter and spring rains which must largely be relied upon to fill reservoirs and to raise the low ground-water to its normal level.

Fig. 11 * shows graphically the mean monthly distribution of the rainfall for several stations representative of the different sections of the

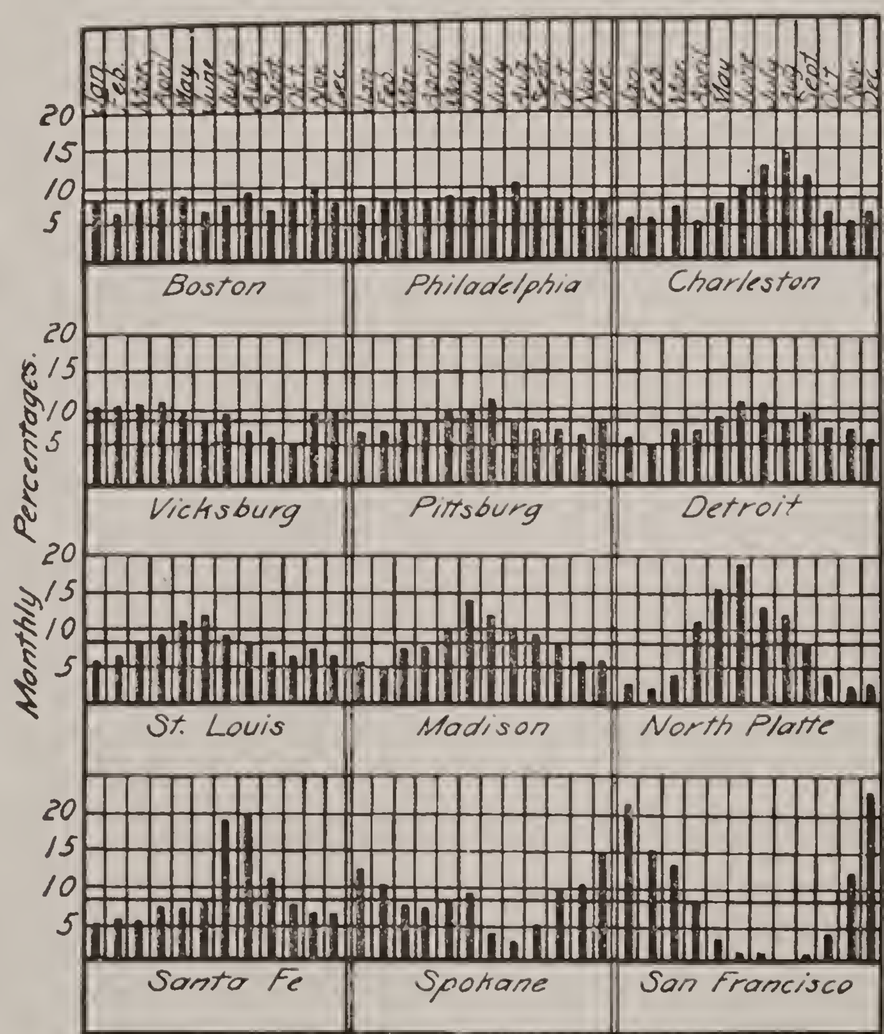

FIG. 11.—MONTHLY VARIATIONS IN RAINFALL.

country. The ordinates represent the percentage of the total yearly rain falling in the month.

In the eastern and southern parts of the country the distribution is quite uniform, the variation here being greatest along the south Atlantic coast, as shown in the diagram for Charleston. As we go farther north and west to Detroit, Madison, and North Platte, a great change

* Bulletin D.

takes place, a larger and larger percentage of the rain falling in the summer months. This is a very advantageous distribution for vegetation, but a very poor one for furnishing surplus water. The diagram for Santa Fé is typical of New Mexico and Arizona, and that for Spokane of the northern plateau. The distribution along the entire Pacific coast is very similar to that at San Francisco.

Numerical data relating to the distribution of the yearly rain are given in Table No. 6.

46. Minimum Yearly Rainfall.—In Table No. 6 are given, for several representative stations, the mean yearly rainfall; the proportion of the yearly rain falling during the six months from June to November, inclusive; the percentage of the mean yearly rain which fell in the driest year covered by the records; the percentages for the two driest consecutive years, and likewise for the three driest consecutive years; and the number of years of records from which the data have been collected. The records close with 1896.

TABLE NO. 6.

GENERAL RAINFALL STATISTICS FOR THE UNITED STATES.

Station.	Mean Yearly Rainfall.	Per cent of Summer and Autumn Rain to Mean Yearly.	Per cent Driest Year to Mean Year.	Per cent Two Driest Years.	Per cent Three Driest Years.	Number of Years' Record.
North Atlantic:						
Boston	45.4	50	60	70	80	79
New Haven	45.8	52	74	78	82	45
New York	44.7	52	62	77	80	61
Philadelphia	42.3	52	70	75	80	72
Washington	42.9	51	69	71	74	45
South Atlantic:						
Wilmington	53.7	61	75	80	81	26
Charleston	49.1	61	48	55	62	89
Augusta	48.0	50	81	88	87	27
Jacksonville	54.1	65	74	77	83	27
Key West	38.2	70	54	61	73	49
Gulf and Lower Mississippi:						
Shreveport	48.2	43	67	75	75	25
Montgomery	52.5	42	76	80	83	24
Mobile	62.6	51	68	75	78	26
New Orleans	60.3	52	64	75	77	26
Galveston	47.7	58	50	65	72	26
Nashville	50.2	46	67	73	83	32
Vicksburg	52.7	43	70	74	83	42
Ohio Valley:						
Pittsburg	36.6	53	70	78	85	54
Cincinnati	42.1	50	60	72	71	62
Indianapolis	42.2	51	59	76	82	27
Louisville	47.2	48	74	81	85	25
Cairo	42.6	47	62	75	81	25

TABLE NO. 6.—*Continued.*

GENERAL RAINFALL STATISTICS FOR THE UNITED STATES.

Station.	Mean Yearly Rainfall.	Per cent of Summer and Autumn Rain to Mean Yearly.	Per cent Driest Year to Mean Year.	Per cent Two Driest Years.	Per cent Three Driest Years.	Number of Years' Record.
Lake Region:						
Marquette	32.3	58	69	75	81	33
Detroit	32.5	56	65	72	79	46
Cleveland	36.6	54	71	74	81	41
Duluth	30.7	63	65	81	88	26
Upper Mississippi Valley:						
St. Louis	40.8	52	55	65	75	60
Davenport	33.3	58	56	68	73	26
Chicago	34.0	54	66	80	86	30
Milwaukee	31.0	55	66	74	73	53
Madison	33.2	58	39	58	68	28
La Crosse	30.7	65	57	78	79	24
St. Paul	28.2	63	53	54	75	39
The Plains:						
Omaha	31.4	63	57	63	70	27
Dodge City	19.8	62	51	58	73	22
North Platte	18.1	61	56	67	72	22
Denver	14.3	48	59	71	77	27
Cheyenne	12.7	55	39	62	75	27
The Plateau:						
Yuma	2.8	39	25	50	46	21
Phœnix	7.1	49	52	88	90	13
Tucson	11.7	65	44	79	80	19
Santa Fé	14.6	69	53	63	66	37
Carson City	12.1	25	57	63	72	19
Salt Lake City	18.8	39	55	64	74	29
Spokane	18.6	38	73	84	84	15
Walla Walla	15.4	38	46	81	86	27
Pacific Coast:						
Astoria	77.0	33	64	68	77	34
Portland	46.2	31	67	70	79	27
Red Bluff	24.4	23	52	64	58	25
Sacramento	19.9	16	42	67	84	47
San Francisco	23.4	17	51	73	78	47
Los Angeles	17.2	15	33	48	59	24
Fresno	9.3	18	61	65	74	19
San Diego	9.7	18	30	54	61	47

By an examination of data relating to stream-flow it is found that the months from June to November are the six months in which the rainfall has in general the least direct effect upon stream-flow. The percentages of the yearly rain falling in these months have therefore been given in the table.

In England it was formerly the practice in designing water-works to assume as the mean rainfall for the three driest years $83\frac{1}{3}$ per cent of the mean, but further investigation led to the adoption of 80 per cent as a more reliable figure. Similar ratios have also been made use of to a considerable extent in this country, but from the above table it is evident that in many localities it would not be safe to use over 75 per cent, or even less. The very low percentages for some of the stations must be taken as an indication of what may occur at any point in a comparatively wide territory in each case. For example, at Madison, Wisconsin, the rainfall in 1895 was but 39 per cent of an average, and this was both preceded and followed by years of low rainfall, thus giving the very low percentages of 58 and 68 for the two and three driest years. This extreme drought was very local, but it shows what may happen at rare intervals in that part of the country.

The lowest percentages for the one, two, and three driest years, with the exception of a few extreme cases, are about the same over a large portion of the United States and may reasonably be placed at about 60, 70, and 75 per cent for the East and South, with a reduction to 50, 60, and 70 per cent respectively for the Northwest and plains region. For the Rocky Mountain region and the Pacific coast the figures would in many places be much lower, but the conditions are here so varied that a general statement would be of no value.

47. **Maximum Rates of Rainfall.**—In estimating the maximum flood-volumes of small streams—a matter of very great importance in the design of dams and reservoir embankments—it is desirable to know the maximum rates of rainfall for periods of a few hours or a single day.

In Table No. 7 are presented data compiled from the *Monthly Weather Review* relating to excessive rainfalls. The records cover the period from 1871 to 1906, and all rainfalls are represented which exceeded in amount 5 inches in twenty-four hours, and, from 1894 to 1906, all those which equaled or exceeded 2 inches in one hour. As far as possible, the same storm is represented but once for any one State, although records may have been received from several stations; and furthermore each storm is counted as a one-day storm or a two-day storm, but not both. A one-day storm is one in which all the rain falls in a meteorological day, that is, from 8 P.M. to 8 P.M., and in a two-day storm all the rain falls within two such days. A one-day storm may therefore have fallen in a few hours, and likewise a two-day storm, so that the figures given do not necessarily represent the maximum rates. However, by taking the maximum from among a great many records the figures thus found for the one- and two-day storms will approximate the maximum for 24 and 48 hours. The one-hour rates are well determined. The number of times a rainfall has exceeded the given amounts is an indication of the frequency of heavy storms and also to some extent, of the reasonableness and reliability of the maximum figure. Those States having the highest maximum rates are those where heavy rainfalls are the most frequent.

TABLE NO. 7.

MAXIMUM RAINFALLS.

State.	No. of Stations.*	Hourly Rate.		One-day Storms.		Two-day Storms.	
		Maximum.	No. of Storms = or > 2 in.	Maximum.	No. of Storms = or > 5 in.	Maximum.	No. of Storms = or > 5 in.
Northern and Central States:							
Maine	18	...	...	5.61	1	5.21	2
New Hampshire and Vt.	41	3.00	1	7.41	1	6.79	5
Massachusetts	78	2.29	2	6.60	2	8.22	8
Connecticut and R. I.	29	4.49	3	7.40	2	10.30	9
New York	76	3.35	7	10.10	8	10.04	10
New Jersey	59	2.58	6	8.73	9	10.40	12
Pennsylvania	84	3.20	10	8.37	10	9.03	6
Delaware	7	...	...	...	...	6.79	1
Maryland	25	4.64	1	7.00	5	14.75	3
Virginia and D. C. (since 1898)	37	3.00	9	7.70	6	6.85	9
West Virginia	39	2.20	1	5.49	2	7.00	1
Tennessee	41	4.00	4	6.57	16	9.67	13
Kentucky	43	2.90	5	7.02	5	8.62	3
Missouri	96	4.74	25	8.00	16	9.60	17
Ohio	142	3.32	11	5.55	2	8.06	7
Indiana	45	2.71	12	7.00	8	10.00	6
Illinois	63	4.36	9	9.08	17	9.35	14
Michigan	86	3.40	4	5.50	1	6.34	4
Wisconsin	64	3.61	4	6.94	5	10.15	3
Minnesota	71	3.30	7	7.20	5	7.80	6
Iowa	78	3.90	11	8.22	12	9.70	6
Kansas	82	3.47	16	8.23	15	8.40	17
Nebraska	69	3.10	11	12.00	13	10.69	8
North and South Dakota	87	3.65	8	7.70	6	6.55	4
Colorado	79	3.08	6	6.20	1	7.39	4
South Atlantic and Gulf States							
North Carolina	56	3.43	17	9.14	34	13.00	26
South Carolina	48	3.00	12	10.82	11	13.22	23
Georgia	62	3.45	28	10.38	26	11.52	31
Florida	34	4.10	48	10.70	37	13.14	40
Alabama	52	3.60	11	9.00	28	10.00	22
Mississippi	51	3.63	12	9.60	24	10.60	24
Louisiana	54	4.12	26	22.27	64	16.55	34
Texas	81	4.33	40	13.93	43	14.78	33
Arkansas	48	3.45	7	11.00	18	9.10	20
Oklahoma (since 1898)	...	2.66	2	...	...	...	...
Pacific Coast:							
California	279	8.67	1	11.50	16	22.40	33
Oregon and Washington	94	...	...	7.12	8	10.40	24

The curves of Fig. 12 based on the data of Table No. 7 show approximately the maximum rainfalls which may be expected for

* Since 1894 the records are from those stations having self-recording rain gauges.

different lengths of time. The curve for the Northern and Central States is somewhat exceeded in a few States, but for most of them it represents rainfalls but little greater than those which have already been observed, and which may occur again at any time. This curve gives a rate of 4 inches for one hour, 8 inches for 24 hours, and 10 inches for 48 hours. The curve for the South Atlantic and Gulf States represents the maximum recorded rainfalls for all the States of this group except Louisiana, for which the records far exceed those of any other

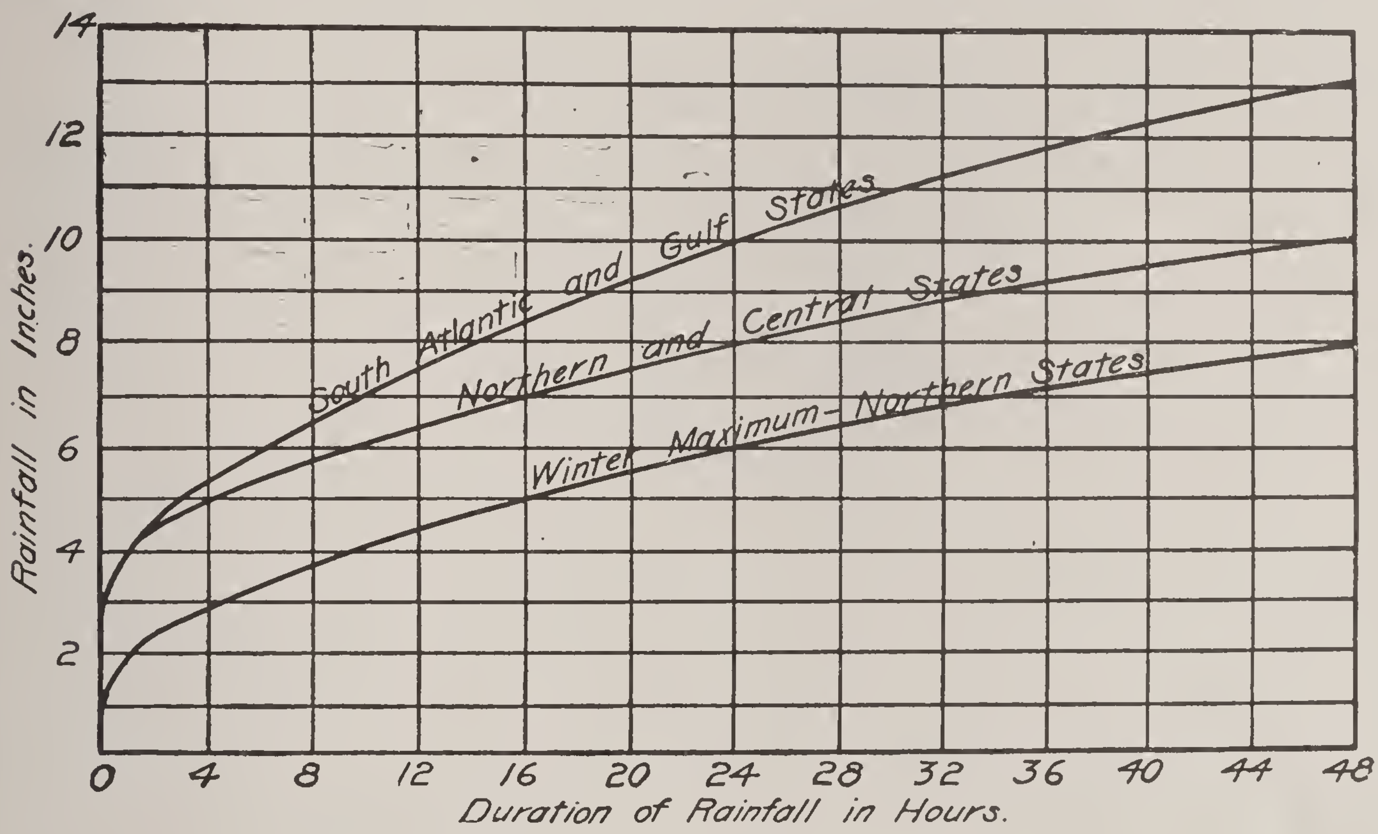

FIG. 12.—MAXIMUM PROBABLE RAINFALLS.

State. For the Pacific coast very high records are also noted at some stations.*

Of especial interest to the hydraulic engineer are the rains which occur while the ground is frozen and covered with snow. A study of the data shows that in all those States where such conditions could obtain, the maximum rates of rainfall for the winter months are considerably below those for the summer months. They are approximately given by the lower curve of the diagram. The melting of snow during an extensive rain may increase the total equivalent by one or two inches, thus giving about the same total as the summer curve.

48. Extent of Great Rain-storms.—That excessive rainfalls are of sufficient extent to cover areas of such size as are ordinarily considered in water-supply problems is made evident by the statistics of a few

* For rates for shorter intervals of time than one hour reference should be made to various works on sewer design and to papers relating thereto. Among these see references (6) to (10), p. 53.

great storms. In October, 1869, a great storm occurred in the eastern part of the United States, with its maximum intensity in Connecticut. A careful analysis of the records made by Mr. James B. Francis * shows the areas covered by different depths of rain to have been as follows:

Depth of Rain.	Area Covered.
6 inches or more	24,431 square miles.
7 " " "	9,602 " "
8 " " "	1,824 " "
9 " " "	1,046 " "
10 " " "	519 " "
11 " " "	179 " "

The following are some of the maximum rates observed in this storm:

4.00	inches in	2	hours.
4.27	" "	3	"
5.86	" "	18.5	"
7.15	" "	24	"
8.90	" "	30	"
8.44	" "	42	"

The maximum recorded rainfall was 12.35 inches, at Canton, Conn., all of which probably fell in about 48 hours.

The winter storm in New England of February, 1886, was also very extensive, it being estimated that 4 inches or more fell over an area of 7600 square miles, 6 inches or more over an area of 3000 square miles, and 8 inches or more over an area of 750 square miles.† It is probably true that some of the most violent storms of the Western mountain region, the so-called "cloud-bursts," are of much greater intensity than any represented in the table, but they are very local in extent.

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* Trans. Am. Soc. C. E., 1878, VII. p. 224.

† *Eng. News*, 1886, XV. p. 216.

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CHAPTER V.

EVAPORATION AND PERCOLATION.

49. Relation of Evaporation and Percolation to Stream-flow and to Ground-water.—Of the rain which falls, a part passes off immediately into the streams and forms what may be called the flood-flow; a part is evaporated directly from the surface of the ground, from water-surfaces, and from the leaves of vegetation; and a portion percolates into the ground. Of the last portion a part is caught by vegetation, passed upwards and evaporated or transpired from the leaves (an insignificant portion being retained by the plant), and a part passes on downwards and laterally, sooner or later finding its way to the surface again in the form of springs, which constitute the source of the dry-weather flow of streams.

The total flow of a stream is then, in general, equal to the rainfall less the evaporation; the flood-flow is equal to the rainfall less the percolation and evaporation; and finally the dry-weather flow may be considered as equal to the deep or more permanent percolation. To enable stream-flow data to be used in the most intelligent way in making estimates, it is therefore desirable to have a knowledge of the laws of evaporation and percolation and of the relative amounts which take place under different conditions. Furthermore, as the percolating water constitutes the ground-water from which many supplies are drawn, this knowledge is of first importance in a study of ground-water sources.

The subject of evaporation naturally divides itself into two parts: evaporation from water-surfaces and evaporation from land-surfaces. The former is of importance in studying the run-off from watersheds having considerable areas of lakes and ponds, and in taking account of the evaporation from reservoirs. Considerable reliable information exists relating to this part, and the application thereof is easy and direct. Evaporation from land-surfaces is, however, much more difficult to determine, since the conditions affecting it are so varied and indeterminate; it is therefore only possible to give figures which indicate in a general way the effects of some of the conditions.

EVAPORATION FROM WATER-SURFACES.

50. **Influences Affecting Evaporation.**—The evaporation from the free surface of water takes place at a rate depending upon the temperature of the water at the surface, and upon the quantity of vapor already in the air immediately adjacent to it. The former varies not only with the air temperature, but with the depth, nature, and extent of the body of water, and with the extent to which it is exposed to wind and sun. The latter depends upon the amount of moisture in the air generally, and also to a large extent upon the action of wind in removing the accumulated vapor from above the water. For any given locality the evaporation will vary closely with the variations in mean air temperature, but for different localities variations in humidity will cause it to be very different even though the temperatures are the same.

51. **Experiments on Evaporation from Water-surfaces.**—Owing to the difficulty of duplicating conditions of humidity and temperature it is evident that determinations of evaporation from small shallow vessels are of little use in arriving at an estimate of the evaporation from large bodies of water. The best results have been obtained by the use of comparatively large vessels placed in a considerable body of water, such as a lake or reservoir. Even in this case the variation in temperature between the water outside and inside the vessel will at times be several degrees, and it is, moreover, difficult to eliminate the effect of the sides of the vessel in protecting the water-surface from the wind.

52. *Experiments at Boston.*—The most extensive experiments of this character which have been made in the United States are those which were carried out by Desmond FitzGerald at the Chestnut Hill reservoir of the Boston water-works.*

In Table No. 8 are given the mean monthly evaporations as deduced from these experiments. For the summer months they are the means for ten years of observations, while for the winter months they are deduced from special experiments on the evaporation from snow and ice.

The maximum daily evaporation was 0.57 inch, the mean temperature of the water being 70°.7 F. in the reservoir and 68°.8 in the tank. Experiments on snow and ice indicated an evaporation of about 0.02

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* See references (2) and (5), p. 65.

inch per day from snow and 0.04 inch from ice. The maximum yearly evaporation was 43.63 inches and the minimum 34.05, or a variation of 11 per cent above and 13 per cent below the mean.

TABLE NO. 8.

MEAN MONTHLY EVAPORATIONS AT CHESTNUT HILL RESERVOIR, BOSTON, MASS.

Month.	Evaporation, Inches.	Per cent of Yearly Evaporation.	Month.	Evaporation, Inches.	Per cent of Yearly Evaporation.
January	0.96	2.4	July	5.98	15.2
February	1.05	2.7	August	5.50	14.0
March	1.70	4.3	September	4.12	10.4
April	2.97	7.6	October	3.16	8.1
May	4.46	11.4	November	2.25	5.7
June	5.54	14.2	December	1.51	3.9

Total for the year = 39.20 inches. Mean temperature = 48°.6.

53. *Experiments at Rochester.*—A similar series of experiments have been carried on since 1891 by Mr. Kuichling at the Mt. Hope reservoir of the Rochester water-works.* The results of these experiments are given in Table No. 9. They are the means of observations covering from two to eight years.

TABLE NO. 9.

MEAN MONTHLY EVAPORATIONS AT MOUNT HOPE RESERVOIR, ROCHESTER, N. Y.

Month.	Evaporation, Inches.	Per cent of Yearly Evaporation.	Month.	Evaporation, Inches.	Per cent of Yearly Evaporation.
January	0.52	1.5	July	5.47	15.8
February	0.54	1.6	August	5.30	15.4
March	1.33	3.9	September	4.15	12.0
April	2.62	7.6	October	3.16	9.1
May	3.93	11.4	November	1.45	4.2
June	4.94	14.3	December	1.13	3.2

Total for the year = 34.54 inches. Mean temperature = 47°.8.

54. *Other Experiments.*—Experiments by J. J. R. Croes on the Croton River in 1865–1870 for eight to ten months of the year gave, as filled out by Mr. FitzGerald † for the winter months, an average annual evaporation of 39.64 inches.

Of foreign experiments, those made by Mr. Charles Greaves at Lee Bridge, England, are probably the most extensive.‡ They embrace fourteen years of observations, and were carried out by means of a

* See the various annual reports of the Executive Board of Rochester, N. Y.

† Trans. Am. Soc. C. E., 1886, XV. p. 617.

‡ Proc. Inst. C. E., XLV. p. 19.

floating slate tank, 3 feet square and 12 inches deep, placed in the river Lee. The results are given in Table No. 10. The maximum yearly evaporation was 26.933 inches and the minimum 17.332, the variations being 31 per cent above and 16 per cent below the mean.

TABLE NO. 10.

MEAN MONTHLY EVAPORATIONS AT LEE BRIDGE, ENGLAND.

Month.	Evaporation, Inches.	Per cent of Yearly Evaporation.	Month.	Evaporation, Inches.	Per cent of Yearly Evaporation.
January	0.755	3.6	July	3.443	16.7
February	0.603	2.9	August	2.849	13.8
March	1.065	5.2	September	1.606	7.8
April	2.098	10.2	October	1.056	5.1
May	2.753	13.4	November	0.669	3.3
June	3.142	15.2	December	0.574	2.8

Total for the year = 20.613 inches.

55. **Calculated Evaporations from Water-surfaces.**—In Table No. 11 are given calculated evaporations deduced from readings of dry- and wet-bulb thermometers at various Signal Service stations in 1887 and 1888, supplemented and controlled by observations at several stations by means of the Piche evaprometer.* The results indicate at least the relative conditions of temperature and humidity at the various stations, and are therefore indicative of the relative evaporation. They are believed by Mr. Russell, the officer in charge, to represent approximately the evaporation from surfaces of ponds, lakes, and reservoirs. The results thus obtained for Boston, New York, and Rochester agree quite well with the observations already quoted.

PERCOLATION, AND EVAPORATION FROM LAND-SURFACES.

56. **Influences Affecting Evaporation and Percolation.**—Evaporation from the ground depends upon the moisture contained therein, upon the temperature, and upon the nature of the vegetation or other soil-covering. The moisture present in the ground depends in turn upon the rainfall, and the ability of the soil to receive and retain the percolating water. The greater the rainfall the greater the evaporation, but evaporation is relatively much more constant than the rainfall. It therefore follows that the difference, or the stream-flow, is more variable than either, and as the rainfall increases the percentage flowing off will increase.

* *Monthly Weather Review*, Sept. 1888, p. 235.

TABLE NO. 11.

CALCULATED MONTHLY EVAPORATION IN THE UNITED STATES.

Stations and Districts.	Jan., 1888.	Feb., 1888	March, 1888.	April, 1888.	May, 1888.	June, 1888.	July, 1887.	August, 1887.	Sept., 1887.	October, 1887.	Nov., 1887.	Dec., 1887.	Year.
New England:													
Eastport	0.9	1.4	1.5	2.4	2.5	2.7	2.2	2.9	2.5	2.6	2.2	1.4	25.2
Portland	1.0	1.2	1.8	2.6	1.8	3.3	3.8	3.9	3.4	3.0	2.5	1.4	29.7
Manchester	0.9	1.6	2.2	3.3	3.8	5.0	4.1	3.3	2.5	2.8	2.4	1.4	33.3
Northfield	0.8	1.0	1.5	2.3	2.5	3.4	3.5	2.7	2.3	1.8	1.1	1.0	23.9
Boston	1.2	1.6	2.2	3.4	3.1	4.7	4.4	4.0	3.5	2.7	2.2	1.4	34.4
Nantucket	1.1	1.1	1.2	1.5	1.8	2.1	3.3	3.8	3.4	2.7	1.8	1.8	25.6
Wood's Holl	0.5	0.8	1.8	2.4	1.8	2.7	2.7	2.4	2.7	1.2	0.8	0.5	20.3
Block Island	1.1	1.1	1.2	2.0	1.8	2.6	2.5	3.1	2.8	2.6	1.8	1.4	24.0
New Haven	1.1	1.6	1.8	2.7	2.7	4.1	3.7	3.8	3.1	3.2	2.4	1.6	31.8
New London	1.5	1.3	1.5	2.6	2.8	4.0	3.4	3.9	3.2	3.1	2.4	2.1	31.8
Middle Atlantic States:													
Albany	0.9	1.2	1.6	3.3	3.9	4.5	5.0	4.7	3.2	3.0	2.1	1.4	34.8
New York City	1.8	1.4	2.0	3.4	3.3	4.6	5.0	5.2	4.3	4.1	3.3	2.2	40.6
Philadelphia	1.6	2.1	2.5	4.4	4.0	5.7	5.7	5.2	4.3	4.0	3.3	2.2	45.0
Atlantic City	1.2	1.6	1.5	2.4	1.8	3.6	2.9	3.3	2.4	1.8	1.2	1.5	25.2
Baltimore	2.0	2.2	2.8	5.1	4.7	5.6	6.0	5.0	4.4	4.3	3.6	2.4	48.1
Washington City	1.8	1.7	2.5	4.2	3.8	6.0	5.4	4.9	4.1	4.2	4.5	2.5	45.6
Lynchburg	2.6	2.7	3.4	5.2	4.5	5.6	4.7	4.3	3.3	3.4	3.2	2.6	45.5
Norfolk	1.8	1.6	2.3	3.5	3.2	4.2	4.6	3.7	3.7	2.9	2.3	1.8	35.6
South Atlantic States:													
Charlotte	2.6	2.6	4.3	6.4	4.5	5.8	4.0	4.0	4.6	4.0	3.6	2.6	49.0
Hatteras	1.8	1.6	1.6	2.5	2.2	3.0	3.3	4.1	3.8	3.2	2.6	1.6	31.3
Raleigh	2.0	1.8	2.6	3.8	4.1	5.4	4.2	3.2	3.0	2.7	2.4	1.8	37.0
Wilmington	2.4	2.2	2.7	3.3	3.3	4.3	4.3	3.1	3.9	3.4	2.8	2.7	38.4
Charleston	2.5	2.5	3.5	3.7	3.9	4.4	4.5	4.8	4.2	4.0	3.2	2.5	43.7
Columbia	2.2	2.3	2.6	4.8	4.3	5.4	4.2	3.8	4.2	3.4	3.6	2.4	43.2
Augusta	3.0	2.6	3.4	5.3	4.8	5.0	4.8	4.5	5.1	4.1	3.6	3.1	49.3
Savannah	3.3	2.8	4.1	4.7	4.3	4.6	4.2	4.7	3.4	3.6	3.5	2.8	46.0
Jacksonville	2.9	2.6	3.8	4.3	4.6	5.3	5.0	4.7	3.8	3.6	3.0	2.1	45.7
Florida Peninsula:													
Titusville	3.5	2.6	3.3	3.8	3.8	4.3	3.8	4.3	4.0	4.1	3.6	3.1	44.2
Cedar Keys	3.3	2.8	4.0	4.6	4.5	5.1	5.0	5.5	4.5	4.1	3.5	2.6	49.5
Key West	3.8	3.7	3.8	4.5	4.4	4.8	5.1	5.1	4.7	4.3	3.8	3.6	51.6
Eastern Gulf States:													
Atlanta	2.7	2.6	4.0	6.2	4.7	5.0	4.5	4.7	5.8	4.6	4.2	2.5	51.5
Pensacola	2.9	2.8	4.1	4.0	4.3	4.6	5.0	5.4	5.2	4.5	3.6	2.4	48.8
Mobile	2.6	2.5	2.8	3.5	3.7	4.0	4.1	4.6	4.6	4.1	3.4	2.2	42.1
Montgomery	3.5	3.3	5.1	6.5	5.9	5.8	4.3	4.5	5.7	4.6	4.3	3.1	56.6
Vicksburg	2.1	2.5	3.6	5.1	5.7	4.8	4.0	5.0	4.7	3.4	4.0	2.2	47.1
New Orleans	2.8	2.8	4.1	3.8	4.2	4.1	4.1	4.3	4.4	4.6	3.7	2.5	45.4
Western Gulf States:													
Shreveport	1.6	2.1	3.0	4.8	4.9	4.2	4.9	5.2	5.0	4.1	3.4	2.4	45.6
Fort Smith	2.2	2.7	3.5	5.3	4.4	4.6	5.6	4.6	4.7	5.9	3.9	2.2	49.6
Little Rock	2.1	2.8	3.5	5.5	4.8	4.1	5.4	5.9	5.8	5.2	4.3	2.3	51.7
Corpus Christi	1.4	1.6	3.3	3.0	3.2	3.9	4.4	4.3	4.3	4.1	3.0	2.3	38.8
Galveston	1.6	2.8	3.2	2.9	4.3	4.2	5.3	5.2	5.2	4.7	4.2	2.4	46.0
Palestine	2.1	3.0	3.3	4.2	4.3	4.5	5.8	4.6	4.8	4.4	4.0	2.1	47.1
San Antonio	2.4	3.3	4.1	3.8	4.0	4.5	6.6	5.8	5.2	5.4	4.2	3.1	52.4

TABLE NO. 11.—*Continued.*

CALCULATED MONTHLY EVAPORATION IN THE UNITED STATES.

Stations and Districts.	Jan., 1888.	Feb., 1888.	March, 1888.	April, 1888.	May, 1888.	June, 1888.	July, 1887.	August, 1887.	Sept., 1887.	October, 1887.	Nov., 1887.	Dec., 1887.	Year.
Rio Grande Valley:													
Rio Grande City....	2.7	3.5	3.5	3.6	4.5	4.6	6.9	7.0	5.2	4.9	3.6	3.1	53.1
Brownsville	1.8	2.6	2.9	3.0	3.5	3.9	4.0	4.1	3.3	3.0	2.6	2.3	37.0
Ohio Valley and Tennessee:													
Chattanooga........	2.0	3.3	3.3	5.3	3.7	4.3	4.3	5.0	5.4	4.0	3.9	1.9	46.4
Knoxville..........	2.4	2.6	3.4	5.0	3.5	4.2	4.9	5.0	4.9	4.1	3.8	2.1	45.9
Memphis...........	2.1	2.3	3.1	5.9	5.3	4.8	4.9	5.4	5.5	4.2	4.1	2.4	50.0
Nashville...........	1.9	2.1	3.2	5.9	5.0	5.1	5.5	6.3	5.9	4.0	3 3	1.9	50.1
Louisville..........	1.7	2.1	2.8	5.6	5.4	5.8	6.8	7.4	6 4	4.9	3.8	2.1	54 8
Indianapolis........	1.3	1.4	2.2	4.6	4.8	5.7	7.7	6.9	5.2	4.1	3.1	1.6	48.6
Cincinnati..........	1.8	1.8	2.6	4.9	5.2	6.4	6 5	6.6	6.1	4.7	3.3	2 1	52 0
Columbus...........	1.6	2.0	2.3	4.5	4.8	5.8	6.9	6.4	5 1	4 0	2.6	1.8	47.8
Pittsburg...........	1.4	1.9	2.2	3.8	4.2	5.4	6.6	5.6	4.9	3.4	2.8	2.3	44.5
Lower Lake Region:													
Buffalo.............	0.8	1.1	1.3	2.2	3.3	3.9	4.9	5.2	3.9	2.8	1.9	1.6	32 9
Oswego.............	0.6	1.0	1.1	2.2	2.8	3.8	3.9	4.0	3.6	2.7	2.2	1.0	28.9
Rochester..........	0.5	1.1	0.9	2.6	3.8	4.9	4.6	4.1	3.8	2.6	2.2	1.3	32.4
Erie................	1.0	1.4	1.4	2.7	3.7	4.6	5.5	4.8	3.1	2.5	1.9	1.2	33.8
Cleveland..........	1.1	1.4	1.5	2.9	3.3	4.4	5.2	4.9	3.8	3.4	2.4	1.4	35.7
Sandusky...........	0.8	1.4	1.5	3.2	3.7	4.6	5.4	5 4	3.7	3.4	2.2	1.3	36.6
Toledo..............	0.9	1.1	1.5	3.5	3.8	4.6	6.0	6.4	3.7	3.4	2.4	1.3	38.6
Detroit	0.8	1.1	1.6	3.0	4.1	4 8	5.9	5.2	3.4	2.8	2.0	1.3	36.0
Upper Lake Region:													
Alpena..............	0.7	0.6	0.9	1.6	2.1	3.6	3.8	3.7	2.8	2.2	1.5	0.8	24.3
Grand Haven.......	0.5	0.7	1.3	2.6	3.1	3.8	4.7	3.8	2.7	2.6	1.7	1.1	28.6
Lansing.............	0.6	1.2	1.4	2.7	2.8	4.0	4.3	3.9	2.4	1.9	1.4	1.0	27 6
Marquette..........	0.8	0.8	0.9	1.7	2.4	3.3	3.4	3.3	3.1	2.2	1.3	1.3	24.5
Port Huron.........	0.6	1.0	1.1	2.6	3.0	3.8	4.6	4.2	3 2	2.5	1.7	1.0	29.3
Chicago............	1.0	1 2	1.8	3.2	3.3	4.8	5.4	5.3	4.1	3.2	2.3	1.2	36.8
Milwaukee..........	0.5	1.0	1.1	2.4	2.6	3.8	4.8	3.7	3.4	2.9	1.9	0.9	29.0
Green Bay..........	0.5	0.6	0.8	1.7	2.5	4.1	5.6	4.2	3.0	2.4	1.9	0.9	28.2
Duluth..............	0.5	0.5	0.6	1.5	2.4	2.5	3.9	3.4	3.0	2.5	1.2	1.0	23.0
Extreme Northwest:													
Moorhead..........	0 2	1.4	0.5	2.1	3.6	3.8	3.7	3.3	3.5	2.4	1.3	0.5	26.3
Saint Vincent.......	0.3	0.3	0.5	1.8	3.8	3.9	3.1	2.6	2.6	2.0	0.9	0.3	22.1
Bismark............	0.4	0.6	0.6	3.0	4.3	4.1	5.6	4.2	4.0	2.6	1.2	0.4	31.0
Fort Buford........	1.4	0.7	0.6	3.0	4.7	5.0	6.2	4.9	4.8	3.0	1.7	0.5	35.5
Fort Totten........	0.2	0.3	0.4	2.2	4.6	3.8	4.2	3.7	3.7	2.3	1.4	0.4	27.2
Upper Mississippi Valley:													
Saint Paul..........	0.7	0.7	2.2	2.0	2.3	4.1	5.0	3.7	2.8	2.4	1.5	0.7	28.1
La Crosse..........	0.4	1.2	1.4	3.3	3 5	4.4	5.4	4.7	3.0	3.0	1.8	0.8	32.9
Davenport..........	0.5	1.0	1.8	3.8	3.4	4.6	6.9	6.2	4.4	3.0	2.3	1.1	39.0
Des Moines.........	0.6	1.0	1.5	3.7	3.1	4.2	6.6	4.7	4.1	3.3	2.3	0.9	36 0
Dubuque............	0.7	1.0	1.4	2.2	2.9	4.2	6.2	4.8	3.3	2.8	1.8	0.9	33.2
Keokuk.............	0.8	1.1	2.1	4.2	3.7	4.3	7.0	6.8	5.0	3.8	2.9	1.2	42.9
Cairo...............	1.6	2.1	2.9	5.8	4.4	4.3	5.6	6.5	5.1	4.5	3.8	2.3	48.9
Springfield, Ill......	0.8	1.1	2.0	4.6	3.8	4.3	5.4	6.5	4.5	3.5	2.9	1.4	40.8
St. Louis...........	1.3	1.6	2.5	5.5	4.7	5.0	7.5	8.0	5.9	4.9	3.9	1.4	52.2

TABLE NO. 11.—*Continued.*

CALCULATED MONTHLY EVAPORATION IN THE UNITED STATES.

Stations and Districts.	Jan., 1888.	Feb., 1888.	March, 1888.	April, 1888.	May, 1888.	June, 1888.	July, 1887.	August, 1887.	Sept., 1887.	October, 1887.	Nov., 1887.	Dec., 1887.	Year.
Missouri Valley:													
Lamar	1.1	1.6	2.4	4.4	3.8	4.0	6.0	4.6	3.7	3.6	2.9	1.5	39.6
Springfield, Mo	1.1	1.7	2.4	5.0	4.8	4.0	5.0	3.4	3.4	3.5	3.1	1.4	38.3
Leavenworth	0.9	1.5	2.3	4.6	4.5	5.0	6.3	4.5	4.0	3.9	2.7	1.4	41.6
Topeka	1.1	1.2	2.0	4.0	4.1	4.1	6.3	3.5	3.2	3.0	2.2	1.4	36.1
Omaha	0.8	1.5	1.4	4.4	3.8	5.2	6.2	5.2	4.3	4.3	3.0	1.4	41.7
Crete	0.7	1.1	1.2	3.5	3.3	4.5	5.6	4.7	3.8	3.6	2.4	1.1	35.5
Valentine	1.2	1.6	1.8	5.0	3.2	5.3	6.9	5.0	5.2	3.8	3.3	1.5	43.8
Fort Sully	0.6	0.9	1.3	4 4	4.1	5.2	7.7	4.9	5.7	3.6	2 8	0.7	41.9
Huron	0.3	0.7	0.8	3.7	3.7	4.1	5.7	4.2	4.1	3.1	2.4	0.7	33.0
Yankton	0.4	1.4	1.2	3.3	3.1	4.4	4.6	3.7	2.9	3.0	2.2	0.8	31.0
Northern Slope:													
Fort Assiniboine	0.8	1.2	1.2	3.8	4.1	4.2	6.8	5.5	4.8	3.5	2.5	1.1	39.5
Fort Custer	0.6	1.5	1.3	5.4	6.8	4.9	9.6	8.0	6.1	3.4	2.9	1.5	52.0
Fort Maginnis	1.1	1.4	1.1	3.3	3.2	4.6	6.8	4.6	3.8	2.8	2.0	1.1	35.8
Helena	1.1	3.6	2.1	6.1	4.3	5.5	7.2	7.7	6.4	4 3	3.0	2.1	53.4
Poplar River	0 4	0.8	0.8	2.7	4.9	5.7	6.0	4.8	4.4	2.5	1.7	0.7	35.4
Cheyenne	3.3	5.7	4.0	8.2	5.2	10.4	8.0	7.7	8.6	5.8	6.1	3.5	76.5
North Platte	0.8	1.8	1.8	5.4	3.9	6.9	6.0	4.8	3.7	2.8	2.3	1.1	41.3
Middle Slope:													
Colorado Springs	3.0	3.3	4.1	6.7	5.6	4.3	6.7	7.2	6.8	4.6	4.2	2.9	59.4
Denver	2.8	3.7	3.5	7.6	5.8	10.5	8.3	8.5	6.1	4.9	4.2	3.1	69.0
Pike's Peak	2.1	1.3	1.5	2.1	1.8	1.9	3.0	4.0	3.0	2.3	2.8	1.0	26.8
Concordia	1.3	2.8	1.8	4.8	4.3	5.7	7.3	5.2	4.3	4.5	3.4	1.8	47.2
Dodge City	1.4	2.4	2.8	4.1	4.6	7.4	8.3	6.6	5.5	5.2	4.2	2.1	54.6
Fort Elliot	1.3	1.9	3.2	5.1	5.4	8.2	7.6	6.2	5.4	4.7	4.2	2.2	55.4
Southern Slope:													
Fort Sill	1.6	2.0	2.6	3.8	4.0	4.4	4.8	7.5	5.1	4.2	4.1	2.0	46.1
Abilene	1.8	1.7	3.1	4.2	5.0	5.8	9.5	7.5	6.2	4.5	3.4	1.7	54.4
Fort Davis	5.4	5.7	6.7	8.5	11.0	12.0	11.4	9.0	5.9	5.2	5.7	4.9	91.4
Fort Stanton	3.9	3.9	5.2	7.3	9.5	10.9	9.4	11.6	3.9	4.0	3.6	3.8	76.0
Southern Plateau:													
El Paso	4.0	3.9	6.0	8.4	10.7	13 6	9.4	7.7	5.6	5.2	4.6	2.9	82.0
Santa Fé	3.0	3.4	4.2	6.8	8.8	12.9	9.2	9.8	6.6	6.7	5.7	2.7	79.8
Fort Apache	2.6	3.0	3.6	6.8	9.4	9.1	7.1	6.7	5.3	5.2	4.1	2.6	65.5
Fort Grant	5.2	4.8	6.4	9.2	10.2	13.8	12.4	10.5	9.0	7.9	7.2	4.6	101.2
Prescott	1.4	2.8	3.6	5.4	6.2	8.1	6.6	6.5	4.7	4.9	3.6	2.2	56.0
Yuma	4.4	5.2	6.6	9.6	9.6	12.6	11.0	10.2	8.2	8.2	5.5	4.6	95.7
Keeler	3.0	4.6	6.3	8.7	9.3	11.9	12.8	13.9	10.6	8.8	5.9	4.8	100.6
Middle Plateau:													
Fort Bidwell	0.8	1.8	1.8	4.6	5.2	4.0	8.8	8.1	5.0	4.6	2.4	1.3	48.9
Winnemucca	0.9	2.8	6.2	9.1	9.3	10.1	11.5	12.0	9.9	6.6	3.7	1.8	83.9
Salt Lake City	1.8	2.7	3.6	7.2	6.9	8.9	9.2	10.7	9.6	6.5	5.0	2.3	74.4
Montrose	1.8	2.7	3.7	6.2	7.0	11.1	10.2	8.3	6.9	5.2	3.4	2.0	68.3
Fort Bridger	1.6	2.5	2.7	4.3	4.3	6.5	7.7	6.8	5.6	4.2	5.2	4.7	56.1
Northern Plateau:													
Boisé City	1.6	2.5	3.8	6.1	6.5	6.6	10.0	9.2	7.4	5.2	3.2	1.8	63.9
Spokane Falls	0.7	1.7	2.7	4.4	5.4	4.4	7.7	6.4	3.8	2.5	1.7	1.4	42.8
Walla Walla	1.1	2.9	3.6	6.2	7.7	5.7	9.9	7.9	5.1	3.4	1.8	2.4	57.7

TABLE NO. 11.—*Continued.*

CALCULATED MONTHLY EVAPORATION IN THE UNITED STATES.

Stations and Districts.	Jan., 1888.	Feb., 1888	March, 1888.	April, 1888.	May, 1888.	June, 1888.	July, 1887.	August, 1887.	Sept., 1887.	October, 1887.	Nov., 1887.	Dec., 1887.	Year.
North Pacific Coast:													
Fort Canby	1.2	1.1	1.8	2.1	2.8	2.3	1.8	2.9	1.8	1.8	1.5	0.9	21.1
Olympia	1.3	1.2	1.8	2.5	4.1	3.3	3.2	3.1	2.4	1.5	1.3	1.1	26.8
Port Angeles	1.0	0.9	1.8	1.8	2.5	2.1	2.1	1.8	1.5	1.2	1.3	1.1	19.1
Tatoosh Island	1.2	1.1	1.8	1.4	1.8	1.8	1.4	1.4	1.4	1.6	1.8	1.4	18.1
Astoria	1.1	1.0	1.6	2.1	3.0	2.7	3.0	2.9	2.6	2.3	1.8	1.2	25.3
Portland	0.9	1.1	2.4	3.4	5.0	3.2	5.4	4.2	3.4	2.7	1.8	1.2	34.7
Roseburg	1.2	1.6	2.7	3.9	4.7	3.5	5.4	4.7	5.0	3.2	1.7	1.6	39.2
Middle Pacific Coast:													
Red Bluff	3.0	4.6	5.4	6.1	7.0	6.9	11.0	10.7	10.1	10.5	5.9	3.6	84.8
Sacramento	1.8	3.1	3.7	4.3	4.2	5.6	5.9	5.6	6.5	7.3	3.9	2.4	54.3
San Francisco	2.7	2.7	3.3	3.1	2.8	3.1	2.4	2.5	3.3	5.0	2.8	3.0	36.7
South Pacific Coast:													
Fresno	1.8	2.8	3.0	5.6	6.0	7.0	9.1	10.2	7.6	6.7	3.8	2.2	65.8
Los Angeles	2.3	2.0	2.8	3.4	3.0	3.8	3.2	3.5	3.1	4.1	3.0	3.0	37.2
San Diego	2.9	2.7	2.5	2.7	3.3	2.8	3.2	3.3	2.9	4.3	3.2	3.7	37.5

If the soil is very coarse or sandy, percolation will be rapid and large, and the water will soon escape beyond the reach of vegetation. This will result in a small evaporation, a large percolation, and consequently a large and steady stream-flow. If the soil is very fine, or is hard and impervious, both percolation and evaporation will be small and stream-flow large and irregular. Topography also greatly affects evaporation by affecting percolation. The maximum evaporation will occur where the soil is sufficiently porous and level to receive the water and to retain it within the reach of vegetation.

57. Effect of Vegetation or Other Soil-covering.—As showing the influence of vegetation on evaporation, Risler's widely quoted table of the daily consumption of water by various crops during the growing season is here given:

Crop.	Consumption of Water in Inches per Day.
Meadow-grass	0.134 to 0.267
Oats	0.140 " 0.193
Indian corn	0.110 " 0.157
Clover	0.140 " .....
Vineyard	0.035 " 0.031
Wheat	0.106 " 0.110
Rye	0.091 " .....
Potatoes	0.038 " 0.055
Oak trees	0.038 " 0.035
Fir trees	0.020 " 0.043

These figures indicate that the grain crops will consume from 10 to 15 inches of water during the growing season. Grasses require still more per day, and for an entire summer season will consume 30 or 40 inches, if furnished. Baldwin Latham found that Italian rye-grass would under suitable conditions consume from 100 to 200 inches per year, if supplied.*

From this it is seen that the demands of vegetation during the growing season are very great, and until these are satisfied very little is left to replenish the ground-water or to add to stream-flow. The large amount consumed by grasses and grains as compared to forest trees is to be noted. Other experiments indicate less difference in favor of forest trees, but there is little doubt that forests require less water than crops. Fernow gives as the ratios of the evaporation from different surfaces relative to that from a water-surface the following: sod 1.92; cereals 1.73; forest 1.51; mixed 1.44; water 1.00; bare soil 0.60.† Forests not only consume less water than crops, but, what is of more importance, they promote regularity of stream-flow by retarding the surface-flow and so increasing the percolation as well as delaying and decreasing the flood-flow.

The effect of a covering on the ground-surface is shown by experiments of Eser. Calling the evaporation from bare ground 100 per cent, the evaporation from ground covered with 1 cm. of sand was 33 per cent; when covered with 5 cm. of straw 10 per cent; with 5 cm. of forest leaves from 11 to 15 per cent; and with grass growing thereon 243 per cent.‡

The effect of vegetation, or other covering, upon the percolation is of course the reverse of its effect upon evaporation. Experiments by Wollny on bare soils 20 inches deep showed that for six months, May to October, the percolation was, for sand 65 per cent, for loam 33 per cent, and for peat 44 per cent of the rainfall. With grass growing thereon the percolation was 14.0, 1.3, and .8.7 per cent, respectively, nearly all of which occurred in October.§

58. **Experiments on Evaporation and Percolation.**—Greaves carried out in connection with his experiments quoted on page 57 many experiments on soils. Two slate tanks were filled, one with ordinary soil and sodded over, the other with fine sand. All the rain either evaporated, or percolated through the soil. The average results for fourteen years were as follows:

	Soil.	Sand.
Rainfall	25.72 inches	25.72 inches
Percolation	7.58 "	21.41 "
Evaporation	18.14 "	4.31 "

* Proc. Inst. C. E., LXXIII. p. 236.

† Bulletin No. 7, Forestry Div., Dept. Agriculture.

‡ Handbuch der Ingenieurwissenschaften; Der Wasserbau, I. Abt., I. Hälfte, p. 34.

§ *Ibid.*, p. 38.

Experiments with uncropped soils 5 feet deep by Gilbert and Laws at Rothamsted, England, from 1870 to 1890, gave the average results shown in Table No. 12. The average percolation through 40 inches of soil was 15.16 inches, and through 20 inches was 14.38 inches. The evaporation was much more uniform than the percolation, it varying from 11.89 to 21.74 inches, while the percolation varied from 3.94 to 24.38 inches. The soil was of a rather heavy character. The areas were $\frac{1}{1000}$ acre in extent, inclosed by cast-iron boxes, and the drainage-water was collected and measured.*

TABLE NO. 12.

EVAPORATION EXPERIMENTS AT ROTHAMSTED, ENGLAND.

Month.	Rainfall. Inches.	Evaporation. Inches.	Percolation. Inches.	Month.	Rainfall. Inches.	Evaporation. Inches.	Percolation. Inches.
January	2.51	0.45	2.06	July	3.03	2.26	0.77
February	2.04	0.60	1.44	August	2.45	1.95	0.50
March	1.74	0.88	0.86	September	2.86	2.11	0.75
April	2.21	1.53	0.68	October	3.20	1.70	1.50
May	2.28	1.69	0.59	November	3.03	0.98	2.05
June	2.52	1.92	0.60	December	2.42	0.61	1.81

Average yearly rainfall .... 30.29 inches.
" " evaporation .... 16.68 "
" " percolation .... 13.61 "

In Table No. 13† are given the results of various European experiments on percolation through various depths of soil and under various conditions. The data are of value as indicating the relative percolations under different conditions. The actual figures must, however, be used with caution, as in most if not all cases the experiments were so conducted that all the precipitation either percolated or evaporated. In actual drainage-areas a portion reaches the stream by running over the surface, most of the flood-flow being thus derived.

59. Evaporation as Determined from Stream-flow.—The average evaporation from large areas as determined by subtracting stream-flow from rainfall can be readily obtained for several watersheds from the data given in Chapter VI. Within the range covered by the data, the mean annual evaporation for the Atlantic coast region obtained in this way is given approximately by the formula $E = 12 + \frac{1}{4}R$, where E = annual evaporation, and R = annual rainfall in inches. Vermeule suggests as a general formula $E = (15.50 + .16R)(.05T - 1.48)$, in which T = mean annual temperature in degrees Fahrenheit.‡

* Proc. Inst. C. E., cv. p. 31.

† From Lueger. Die Wasserversorgung der Städte, p. 203.

‡ Geolog. Survey N. J., 1894, III. p. 76.

TABLE NO. 13.

EXPERIMENTS ON PERCOLATION (LUEGER).

Place.	Depth of Soil in Inches.	Kind of Soil.	With or Without Vegetation.	Rainfall in Inches.	Percolation in per cent of Rainfall.				
					Spring.	Summer.	Fall.	Winter.	Year.
Abbotshill.....	36	Sandy loam	With	26.0	30.3	1.7	54.1	83.9	42.3
Holmfield	36	Dolomitic soil	With	24.7	24.9	7.7	22.8	30.3	19.6
Lee-Bridge....	36	Sand	Without	25.7	......	......	.....	......	83.2
"	36	Loamy sand	With	25.7	......	......	......	......	26.6
Rothamsted. ..	40	Loam and clay	Without	31.0	......	......	......	......	43.4
Calèves........	47	Clay	With	39.8	......	......	......	......	28.0
Görlitz	49	Clay	Without	25.7	36.1	29.3	26.5	19.0	28.1
"	49	Loam	Without	25.7	52.4	45.6	28.6	29.9	41.0
"	49	Sandy loam	Without	25.7	49.7	42.4	27.9	37.7	40.5
Tharand }	49	Clay	With	29.0	59.0	21.3	20.9	84.4	40.8
Moholz }	49	Loam	With	29.0	89.7	36.0	32.9	92.0	58.7
Erlangen..	25	Sand	Without	......	......	......	......	......	43.0
Salzburg.......	25	Sandy loam and clay	With	30.2	......	......	......	......	33.9
"	25	Sandy loam and clay	Without	30.2	......	......	......	......	64.2
Oberdöbling....	50	Loam	Without	25.8	43.3	41.0	24.4	32.0	32.8

60. Amount of Percolation over Large Areas.—The proportion of the stream-flow that is derived from the ground-water or from the percolation varies greatly for different watersheds. Even where the subsoil is very porous it is usually the case that the surface-soil is more or less clayey and during heavy rains much water will flow off over the surface. Long Island furnishes an example of conditions very favorable to percolation, the amount obtained directly from wells at Brooklyn being in 1894 two-thirds of the total yield of the watersheds drawn from. This is equivalent to about 500,000 gallons per day per square mile, equal to 10.5 inches or 28.5 per cent of the rainfall. The total percolation must have been at least 12 inches, or three-fourths the total yield. Experience in Holland in collecting water from sand-dunes indicates that from 30 to 50 per cent of the rainfall is available in the ground-water. Conditions are, however, seldom so favorable as at these places.

An approximate estimate of the total amount of percolation which is useful in adding to stream-flow or to ground-water supplies may be made for any particular region by subtracting the flood-flows of a stream from its total flow, providing the storage afforded by small ponds and lakes is insignificant. In small streams the flood-flows increase and decrease so rapidly that it is not difficult to eliminate in this way most of the surface-flow.

The results of an analysis of this kind of the daily flow of the Perkiomen, Neshaminy, and Tohickon, made from data given in the reports of the Philadelphia Water Bureau, show that the annual run-off, excluding the flood-flows, usually amounts to from 5 to 8 inches. This is equivalent to from 25 to 35 per cent of the total run-off and 12 to 18 per cent of the rainfall.

LITERATURE.

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3. Depth of Evaporation in the United States. *Monthly Weather Review*, September, 1888.
4. Harrison. On the Subterranean Water in the Chalk Formation of the Upper Thames and its Relation to the Supply of London. Proc. Inst. C. E., 1890–91, CV. p. 2.
5. FitzGerald. Rainfall, Flow of Streams, and Storage. Trans. Am. Soc. C. E., 1892, XXVII. p. 253.
6. Fernow. Relation of Evaporation to Forests. Bull. No. 7, Forestry Div., U. S. Dept. Agr. *Eng. News*, 1893, XXX. p. 239.
7. Vermeule. Report on Water-supply. Geological Survey of New Jersey, 1894, III. A valuable and exhaustive discussion of the subject of rainfall, evaporation, ground-storage, and stream-flow as applied to New Jersey.
8. Handbuch der Ingenieurwissenschaften, Band III, Abt. 1, 1. Hälfte, and Lueger, Wasserversorgung der Städte, contain many references to the foreign literature of this subject.
9. Kimball. Evaporation Observations in the United States. *Eng. News*, 1905, LIII. p. 353.

CHAPTER VI.

FLOW OF STREAMS.

61. General Methods of Procedure.—When a stream is under consideration as a source of water-supply, the peculiarities of its flow—the minimum, maximum, and total flow for various periods of time—are among the first things to be determined. The most accurate as well as the most direct method of determining these is by means of a series of gaugings extending over several years, which, to be of the greatest value, should include periods of high flood and periods of drought. A long series of gaugings is, however, seldom available at the time when a source must be decided upon, but by establishing gauges at the earliest possible moment much valuable information may be had by the time detailed designs are required. This applies especially to the case of a city seeking an additional supply.

Where gaugings are not to be had, or where they are very limited in extent, as close an estimate as possible must be made from a comparison with other streams whose flows are known, taking into account as far as may be the differences in rainfall, climate, and in the various characteristics of the different watersheds. Where such differences are great this method will give results only roughly approximate, but still much better than mere guesses and quite sufficient in many cases to determine the availability of a given source. Where, however, the margin is close, and in problems pertaining to the detailed design, a more accurate knowledge is greatly to be desired. It can be obtained only by means of gaugings.

62. Influences Affecting Stream-flow.—All streams derive their supply ultimately from the rainfall, and, in general, the amount of the run-off is equal to the rainfall less the evaporation. In the last chapter the various influences affecting evaporation and percolation were discussed, and it only remains to consider how the variations in these factors go to affect stream-flow.

Whatever augments evaporation decreases stream-flow, and by the same amount. Thus a watershed with a large percentage in grass will yield a less amount than one with rocky and barren hillsides; one with a large percentage of water-surface, less than one with a small percentage. Again, the higher the temperature the greater the evaporation and the less the stream-flow. An increased rainfall will also increase the evaporation, but the relative increase in evaporation will be less than that in the rainfall; hence the larger the rainfall the greater the percentage flowing off. The distribution of the rainfall throughout the year also affects greatly the evaporation and consequently the stream-flow.

The effect of large percolation is to make the run-off more uniform; but where the water is held for the use of vegetation by a porous soil, a large percolation may result in a decreased total flow. Steep, rocky hillsides will give a large per cent of the rainfall to the streams, but the flow will be very irregular; flat grass-lands will give little or nothing to the streams during the season of growth. Again, the winter climate has, through its effect on percolation, an important influence on the regularity of the flow. When the ground is frozen, little water goes to replenish the ground-storage during the melting of the snow, but if the soil is open to winter rains and snows, much water will be furnished through percolation to increase the summer flow, while the spring floods will be correspondingly reduced. This effect of climate is well illustrated in Fig. 15, page 82, in the curves for the Connecticut and Savannah rivers.

It is thus seen that temperature, topography, vegetation, and soil, as well as the amount of rainfall, are important factors to be considered in a study of the flow of a stream.

It should here be noted that from any given area of watershed a portion of the percolating water is likely to escape to a lower point of the valley before coming to the surface. The amount lost in this way, although usually insignificant, is sometimes very large. This question, together with methods of utilizing such water, is discussed in subsequent chapters; for the present it will be assumed that this portion is so small in amount that it may be neglected.

63. Units of Measure.—Rainfall is expressed in inches in depth, and the rate in inches per hour or per twenty-four hours; and for comparative purposes stream-flow is often likewise expressed, meaning thereby inches in depth over the entire watershed. For other purposes the flow is usually expressed in cubic feet, or cubic feet per square mile of watershed, and the rate of flow in cubic feet per second, or cubic feet per second per square mile. The foot and second units are also convenient to use in all hydraulic formulas, but in matters pertaining to storage and distribution the gallon unit is in common use, and rates are expressed in gallons per minute and gallons per twenty-four hours.

For convenience in computations relative to rainfall and flow of streams, the following table is inserted.

TABLE NO. 14.

VOLUMES AND RATES OF FLOW IN FEET AND SECONDS CORRESPONDING TO GIVEN VOLUMES AND RATES OF RAINFALL IN INCHES AND HOURS.

Depth in Inches.	Cubic Feet per Square Mile.	Inches per Hour.	Cubic Feet per Second per Square Mile.	Inches per 24 Hours.	Cubic Feet per Second per Square Mile.
0.1	232,320	0.1	64.5	1	26.9
0.2	464,640	0.2	129.0	2	53.8
0.3	696,960	0.3	193.5	3	80.7
0.4	929,280	0.4	258.1	4	107.5
0.5	1,161,600	0.5	322.6	5	134.4
0.6	1,393,920	0.6	387.1	6	161.3
0.7	1,626,240	0.7	451.7	7	188.2
0.8	1,858,560	0.8	516.2	8	215.1
0.9	2,090,880	0.9	580.7	9	242.0
1.0	2,323,200	1.0	645.3	10	268.9

One inch of rain = 2,323,200 cu. ft. per sq. mile.
One inch per hour = 645.33 cu. ft. per sec. per sq. mile.
One inch per 24 hours = 26.89 cu. ft. per sec. per sq. mile.
One cubic foot = 7.4805 U. S. gallons.
One cubic foot per sec. = 646,300 gallons per day.

64. **Divisions of the Subject.**—The question of the flow of streams naturally divides itself into three parts:

First, the minimum flow of the stream.

Second, the maximum or flood flow.

Third, variations in the flow through successive months and years.

The first information is necessary in case a stream is under consideration for which but little storage is obtainable, or in answer to the question whether it is practicable to draw directly from the stream without storage. The second is of great importance in the design and execution of all river work, and especially in determining the size of waste-weirs. The third determines the supplying capacity of the water-shed and the size of impounding reservoirs.

MINIMUM FLOW.

65. The dry-weather flow of streams is maintained entirely from ground- and surface-storage; and as facilities for such storage vary in

different watersheds, so will the minimum flow vary. Surface-storage, if consisting of large areas of shallow lakes and ponds, acts to decrease greatly the total flow of a stream on account of the great evaporation, while at the same time it usually increases the minimum flow.

In Table No. 15 on page 70 are given the minimum flows of several streams in different localities. For streams in the northern Atlantic-coast States these and other statistics indicate that for watersheds of less than 200 square miles in area the minimum flow varies from nearly zero to about 0.2 cubic foot per second per square mile, averaging 0.10 or 0.12. For large streams the minimum is rarely less than 0.10, and in some cases is as high as 0.30, the latter figure being about the minimum flow for the Connecticut with an area of 10,234 square miles, and for the Merrimack with 4599 square miles of watershed. In the upper Mississippi valley the minimum flow is much less, as indicated by the data for the Rock, the Illinois, and the Des Plaines rivers. Streams in this locality of several hundred square miles of watershed are likely to have a minimum of zero, while still further west this applies to streams of thousands of square miles of catchment-area.

MAXIMUM OR FLOOD FLOW.

66. General Considerations.—The maximum rate at which the waters from great storms will pass down a stream is affected largely by the steepness of the slopes, by the size and shape of the drainage-area, and by the distribution of the branches. Small areas will have larger maximum rates of flow than large areas, other things being equal, as the former are affected by short rainfalls of high rates, while in the latter case the maximum flows are caused by rains of longer duration but of less intensity. For a like reason streams with steep slopes will have a higher maximum rate than those with flat slopes.

The evaporation which takes place during a flood is of so little importance that its effect may be neglected. Percolation absorbs large portions of heavy rains if the ground is dry, but such rains are quite as apt to occur with the ground already soaked or even frozen, so that in the extreme case, which is the one that must be considered, percolation is of small moment.

Of much greater importance in distributing the run-off over a long interval of time, and so reducing the maximum rate, is the surface storage of natural lakes and ponds and of those created by the inundation of large flats bordering the stream. The effect of this last factor may be sufficient to reduce the flood-flow to one-half or one-fourth that of a stream with a narrow valley.

67. Data of Maximum Rates of Flow. — In Table No. 15 are given data concerning the maximum flow of streams taken mainly from Water-supply Paper No. 147, U. S. G. S., by E. C. Murphy. Much detailed information concerning floods is given in this paper and in other Water-supply Papers of the Survey. A great variation is observable in the table, due partly to the varying rates of rainfall, but largely to the differing characteristics of the streams. Nevertheless the data will be of some assistance in estimating probable maximum floods.

TABLE NO. 15.

MINIMUM AND MAXIMUM FLOW OF STREAMS.

Stream.	Place.	Drainage area; square miles.	Minimum Flow; cu. ft. per sec. per sq. mile.	Maximum Flow; cu. ft. per sec. per sq. mile.	Authority.*
	Northeastern United States:				
Skinner Creek	Mannsville, N. Y.	6.40	...	124.20	(23)
Coldspring Brook	Massachusetts	6.43	...	48.40	"
Croton River, S. Branch	New York	7.80	...	73.90	"
Woodhull Reservoir	Herkimer, N. Y.	9.40	...	77.80	"
Stony Brook	Boston, Mass.	12.7	...	121.00	"
Great River	Westfield, Mass.	14.0	...	71.40	"
Swartswood Lake	New Jersey	16.0	...	68.00	"
Williamstown River	Williamstown, Mass.	16.5	...	34.00	"
Croton River, W. Branch	New York	20.5	0.020	54.40	"
Beaverdam Creek	Altmar, N. Y.	20.7	...	111.00	"
Trout Brook	Centerville, N. Y.	23.0	...	50.60	"
Wautuppa Lake	Fall River, Mass.	28.5	...	72.00	"
Pequest River	Huntsville, N. J.	31.4	...	19.30	"
Sawkill	New Jersey	35.0	...	228.60	"
Whippany River	Whippany, N. J.	37.0	...	61.62	"
Cuyadutta Creek	Johnstown, N. Y.	40.0	...	72.40	"
West Canada Creek	Motts Dam, N. Y.	47.5	...	34.10	"
Pequannock River	New Jersey	48.0	...	115.00	(12)
South Fork	Croyole Tp., Pa.	48.6	...	215.00	(6)
Sauquoit Creek	New York Mills, N. Y.	51.5	...	53.40	(23)
Rockaway River	Dover, N. J.	52.2	...	43.00	"
Oneida Creek	Kenwood, N. Y.	59.0	...	41.20	"
Flat River	Rhode Island	61.0	...	120.75	(2)
Camden Creek	Camden, N. Y.	61.4	...	24.10	(23)
Nine Mile Creek	Stittville, N. Y.	62.6	...	124.90	"
Wissahickon Creek	Philadelphia, Pa.	64.6	0.232	43.50	"
Sandy Creek	Allendale, N. Y.	68.4	...	87.70	"
Rock Creek	Washington, D. C.	77.5	...	126.30	(3)
Sudbury River	Framingham, Mass.	78.0	0.036	44.2	(5)
Hockanum River	Connecticut	79.0	...	78.10	(23)
Nashua River	Massachusetts	84.5	...	71.04	"
Independence Creek	Crandall, N. Y.	93.2	...	66.50	"
Deer River	Deer River, N. Y.	101	...	78.10	"
Wanaque River	New Jersey	101	...	66.00	"
Tohickon Creek	Point Pleasant, Pa.	102	0.002	112.50	"
Fish Creek, E. Branch	Point Rocks, N. Y.	104	...	80.50	"
Nashua River	Massachusetts	109	...	104.53	(2)

* See references at end of chapter.

TABLE NO. 15. — *Continued.*

MINIMUM AND MAXIMUM FLOW OF STREAMS.

Stream.	Place.	Drainage area; square miles.	Minimum Flow; cu ft. per sec. per sq. mile.	Maximum Flow; cu. ft. per sec. per sq. mile.	Authority.*
Sandy Creek, N. Branch . .	Adams, N. Y.	110	. . .	67.30	(23)
Scantic River, N. Branch . .	Connecticut	118	. . .	51.80	"
Ramapo River	Mahwah, N. J.	118	. . .	105.09	"
Rockaway River	Boonton, N. J.	119	. . .	40.70	(11)
Paulinskill River	New Jersey	126	0.13	54.	(11)
Patuxent River	Laurel, Md.	137	0.124	31.20	(23)
Neshaminy Creek	Pennsylvania	139	0.009	97.60	(18)
Oriskany Creek	Colemans, N. Y.	141	. . .	55.80	(23)
Perkiomen Creek	Frederick, Pa.	152	0.39	105.4	(18)
Mohawk River	Ridge Mills, N. Y.	153	. . .	46.40	(23)
Ramapo River	Pompton, N. J.	160	0.140	66.10	"
Fish Creek, W. Branch . .	McConnellsville, N. Y. . .	187	. . .	32.70	"
Pawtuxet	Rhode Island	190	. . .	56.9	(2)
Salmon River	Altona, N. Y..	221	. . .	27.60	(23)
Black River	Forestport, N. Y.	268	. . .	39.00	"
South Branch	New Jersey	276	. . .	100.	(12)
Croton River	Croton Dam, N. Y.	339	0.150	74.90	(11)(2)
Great River	Westfield, Mass.	350	. . .	151.40	(23)
East Canada Creek	Dolgeville, N. Y.	356	. . .	24.70	"
Moose River	Agers Mill, N. Y.	407	. . .	31.00	"
Stony Creek	Johnstown, Pa.	428	. . .	70.00	"
West Canada Creek	Middleville, N. Y..	518	. . .	24.90	"
Farmington River	Connecticut	584	. . .	41.70	"
Monocacy River	Frederick, Md.	665	0.116	29.80	"
Passaic River	Little Falls, N. J.	773	0.190	24.20	"
North River	Port Republic, Va.	804	0.220	29.80	"
Passaic River	Dundee, N. J.	823	. . .	43.38	"
North River	Glasgow, Va.	831	0.180	44.80	"
Raritan River	Boundbrook, N. J.	879	0.140	59.30	"
Potomac, N. Branch	Cumberland, Md.	891	0.022	22.80	"
Black River	Lyons Falls, N. Y.	897	. . .	46.00	"
Schoharie Creek	Fort Hunter, N. Y.	948	. . .	44.00	"
Genesee River	Mount Morris, N. Y. . . .	1,070	0.094	39.20	"
Mohawk River	Little Falls, N. Y.	1,306	. . .	21.83	(23)
Greenbrier River	Alderson, W. Va.	1,344	. . .	41.60	"
Black River	Carthage, N. Y.	1,812	. . .	21.20	"
Chemung River	Elmira, N. Y.	2,055	. . .	67.10	(14)
Androscoggin River	Rumford, Me.	2,220	. . .	25.00	(23)
Mohawk River	Rexford, N. Y.	3,384	. . .	23.10	"
Kennebec River	Waterville, Me.	4,410	. . .	25.20	"
Hudson River	Mechanicsville, N. Y. . . .	4,500	. . .	15.50	"
Merrimac River	Lawrence, Mass.	4,553	0.31	23.40	"
Potomac River	Dam, No. 5, Md.	4,640	0.78	22.20	"
Delaware River	Lambertville, N. J.	6,500	. . .	53.80	"
Susquehanna River	Northumberland, Pa. . . .	6,800	. . .	17.50	"
Connecticut River	Holyoke, Mass.	8,660	. . .	21.10	"
Connecticut River	Hartford, Conn.	10,234	0.51	20.30	"
Potomac River	Maryland	11,043	. . .	42.60	"
Potomac River	Great Falls, Md.	11,427	. . .	41.20	"
Susquehanna River	Harrisburg, Pa.	24,030	. . .	18.90	"

* See references at end of chapter.

TABLE NO. 15.—*Continued.*

MINIMUM AND MAXIMUM FLOW OF STREAMS.

Stream.	Place.	Drainage area; square miles.	Minimum Flow; cu. ft. per sec. per sq. mile.	Maximum Flow; cu. ft. per sec. per sq. mile.	Authority.*
	Southeastern United States:				
Coosawattee River	Carters, Ga.	532	0.588	31.86	"
Etowah River	Canton, Ga.	604	0.405	31.50	"
Tuckasegee River	Bryson, N. C.	662	0.603	58.23	"
Little Tennessee River	Judson, N. C.	675	0.408	85.24	"
Broad River	Carlton Ga.	762	0.344	38.22	"
Catawba River	Catawba, N. C.	1,535	0.553	53.10	"
Yadkin River	Salisbury, N. C.	3,399	...	31.60	"
Tallapoosa River	Milstead, Ala.	3,840	...	18.23	"
Broad River	Alston, S. C.	4,609	...	28.44	"
Black Warrior River	Tuscaloosa, Ala.	4,900	...	27.89	"
Savanna River	Augusta, Ga.	7,294	...	42.50	"
Tennessee River	Chattanooga, Tenn.	21,418	...	20.80	"
	Central United States:				
Des Plaines River	Riverside, Ill.	630	0.0	21.4	(15)
Rock River	Rockford, Ill.	6,500	0.016	...	(13*b*)
Mississippi River	St. Paul, Minn.	6,0853	...	19.70	(23)

68. **Formulas for Flood-flows.** — Various formulas have been proposed for expressing the maximum flow of a stream, some involving only the rainfall and area, while others attempt to take account also of the slope and shape of the watershed. Obviously any formula which does not involve the last two factors is not of general application, although it may give good results for a particular class of streams. In applying such a formula to other streams due allowance must be made for the differing conditions.

Among the most widely known of this class of formulas is that given by Fanning and recommended by him as applicable to average New England and Middle-State basins. It is

$$Q = 200\frac{M^{\frac{5}{6}}}{M}, \qquad (1)$$

in which Q = discharge in cubic feet per second per square mile, and M = area in square miles. Thus the total discharge is made to vary according to $M^{\frac{5}{6}}$. The discharges given by this formula have been materially exceeded in some cases, especially for the smaller watersheds, and have been reached by floods caused by rains much below

* See references at end of chapter.

the maximum. It gives a value of Q which appears to increase somewhat too slowly with decrease in area.

Another formula derived from measurements of streams of flat slopes in the upper Mississippi valley is that proposed by Cooley * and is

$$Q = 180 \frac{M^{\frac{2}{3}}}{M}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

It is intended to represent those floods occurring with comparative frequency, as once in six or ten years. Several other formulas are given by Mr. Cooley in the paper referred to.

It may be here noted that the waste-weirs of the dams of the Boston Water-works are designed to carry a flood-volume at the rate

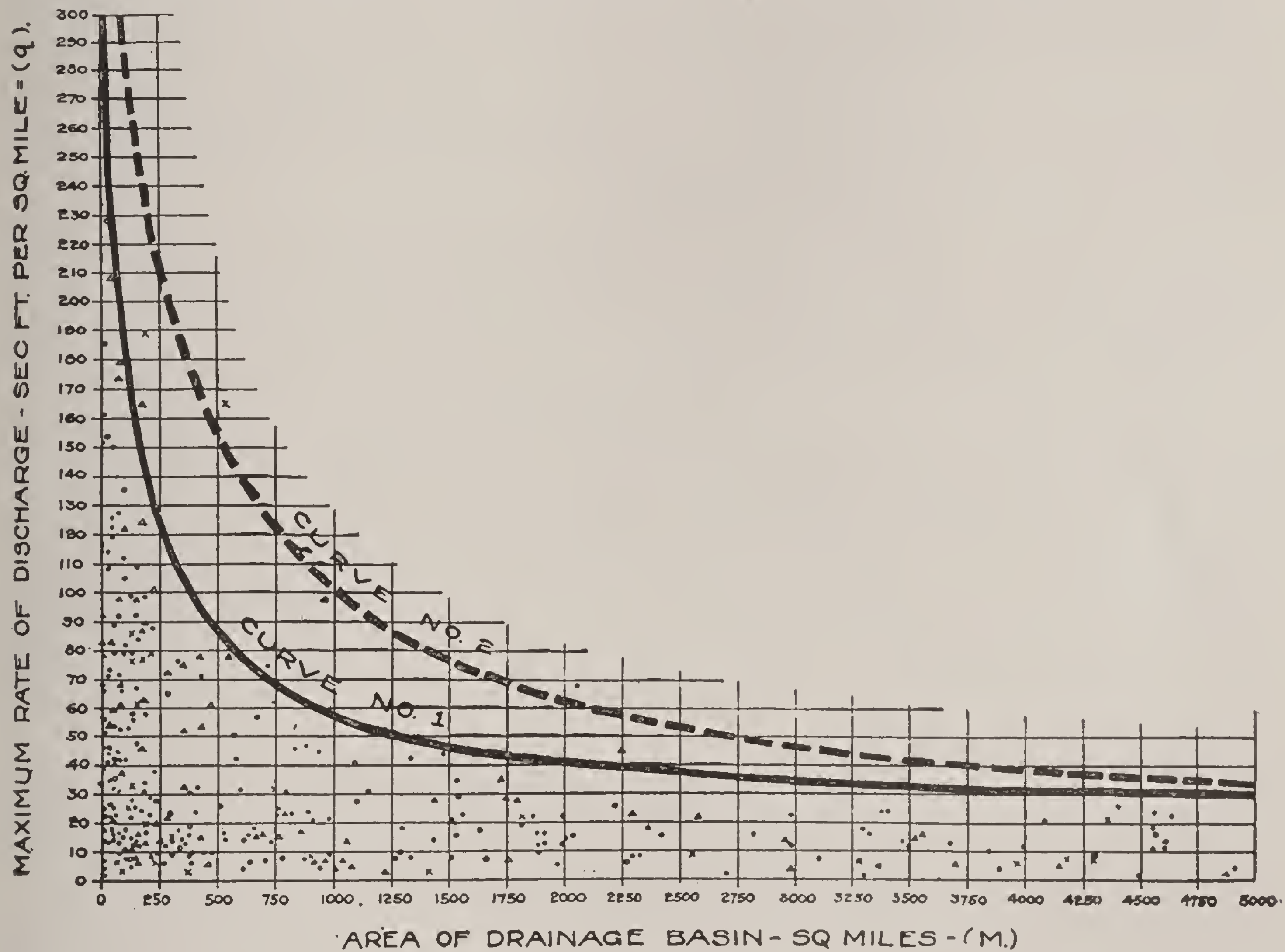

FIG. 12a. — RELATION BETWEEN FLOOD DISCHARGE AND DRAINAGE AREA (KUICHLING).

of 6 inches per 24 hours, or 161 cubic feet per second per square mile. The watersheds are from 20 to 75 square miles in area.

The relation between flood-flow and drainage area is well shown in Fig. 12a, from Kuichling's report on the Water-supply for the New York Barge Canal.† On this diagram are plotted the

* Jour. West. Soc. Engrs., 1896, I. p. 306.

† Report of State Engineer on Barge Canal, 1901, p. 844. This paper contains a discussion of many formulas.

data of Table No. 15 and numerous other data relating to American and European rivers. Curve No. 1 represents floods which may occur occasionally and curve No. 2 those which may occur but rarely.

Mr. Murphy, in Water-supply Paper No. 147 already quoted, plots in a similar manner data for streams in the northeastern United States obtaining a curve whose equation is

$$Q = \frac{46,790}{M + 320} + 15. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

This gives somewhat lower values than Mr. Kuichling's "Curve No. 1."

69. **Rational Method of Estimating Flood-flow.** — A more rational method than by the use of a formula, and one which is applicable to any area, has been proposed by some engineers, most recently by Mr. Chamier in a paper before the Institution of Civil Engineers.* It is based upon the following principles:

Assuming a uniform rate of rainfall, the flow of a stream will increase rapidly until a sufficient time has elapsed for water to reach the point in question from all parts of the drainage-area. After this, with a continuation of the rain, the increase in flow will be much slower, being now due to the increase in the percentage flowing off. The maximum flood will then probably be produced by the greatest possible rainfall of a duration corresponding to the length of time above mentioned. Rates of rainfall of long duration are far from uniform, but irregularity in the rate within the time required for the concentration of the water to the point of discharge will have little effect on the maximum rate of flow.

The elements to be determined in this method are then: (1) the length of time required for the water from the most remote part of the watershed to reach the point of discharge; (2) the maximum rate of rainfall of a duration equal to this time; and (3) the percentage flowing off. The maximum rate of flow will then be equal to this rate of rainfall multiplied by the percentage as above found.

(1) *The Time Required.* — In estimating this time the maximum distance to be covered can be obtained from a good map of the area in question, making due allowance for the sinuosities of the smaller channels. The velocities of flow are more difficult of estimation. In

* Proc. Inst. C. E., 1898, CXXXIV. p. 313. Abstracted in *Engineering Record*, 1899, XXXIX. p. 163.

channels of considerable size they may be roughly estimated by Kutter's formula, the slopes and high-water cross-sections being known. For smaller branches the velocity will usually range from two to four miles per hour, though in some cases it may be considerably higher. On lateral slopes, before reaching well-defined channels, the water will move at a slow velocity, estimated by Mr. Chamier at from one-half to one mile per hour. In any case an approximate estimate of velocity in the various channels can be made by a few direct observations during moderately high water.

A good notion of the total time required for the concentration of the water may also be obtained by observing the time which elapses from the beginning of a sudden storm until the maximum effect is felt at the point of discharge. Where the country is level and the drainage-channels far apart, or where there is large surface storage, this would be the most reliable method of estimating the time.

If the element of time is correctly determined, the effects of size and shape of area, of slopes, and of surface storage will all have been taken account of to a very large degree.

(2) *The Rate of Rainfall.*—The time having been determined, the corresponding rainfall may be taken from the data of the last chapter. To the estimated rainfall, if occurring in the winter, may be added a maximum of about 2 inches for melting snow.

(3) *The Percentage Flowing Off.*—The total flood-discharge of a stream at times when the ground is previously soaked or frozen will usually vary from 50 to 75 per cent of the rainfall. The percentage running off during the rise of a stream will, however, be considerably less than the total percentage for the entire flood, on account of the effect of pondage and the greater percolation during the first part of the storm. In the paper already referred to Mr. Chamier estimates the percentages for different conditions as follows:

For flat country, sandy soil, or cultivated land, 25 to 35 per cent.

For meadows and gentle declivities, absorbent ground, 35 to 45 per cent.

For wooded hill-slopes and compact or stony ground, 45 to 55 per cent.

For mountainous and rocky country or non-absorbent surfaces, as frozen ground, 55 to 65 per cent.

70. Diagram for Flood-flows.—The diagram Fig. 13 gives directly the rate of flood-flow corresponding to various percentages and various values of the time required for the concentration of the flood-waters as determined under (1). It is constructed on the basis of rainfalls according to the middle curve of Fig. 12, page 51, or the lower curve with 2 inches added for snow. For small areas in which the maximum flow is normally reached in three or four hours, an allowance of 1 inch for snow would be sufficient.

In making use of this diagram it should be remembered that it is based on very excessive and rare rainfalls, and therefore the flood-flows

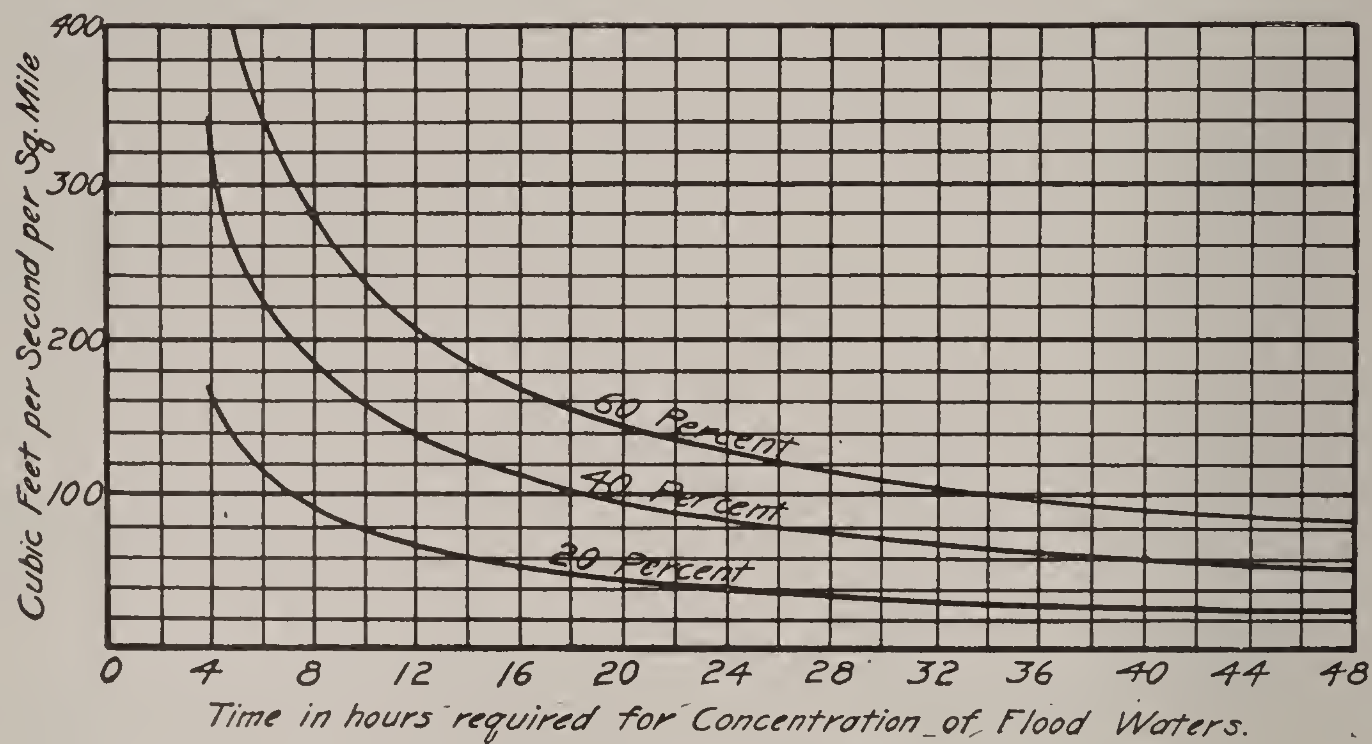

FIG. 13.—FLOOD-VOLUMES.

resulting will be the extraordinary floods which may occur perhaps once or twice in a century. Such, however, must be provided for in designing waste-weirs where inadequate dimensions would endanger the lives of the population in the valley below. For structures where a failure would mean a property loss only, it would often be more economical to provide for ordinary floods only, in which case a less rate of rainfall should be adopted according to local conditions.

71. **Example.**—To illustrate the use of the foregoing method, let it be required to estimate the flood-flow of a certain stream of a drainage-area of 50 square miles, with steep side slopes and a long valley. The length of the valley is, say, 15 miles, and actual length of the main channel 25 miles, with 5 miles of smaller channels reaching to the farthest part of the area. The time required for the water to get to the small channels may be one hour, to flow the 5 miles two hours, and the 25 miles eight hours, or a total of eleven hours. The summer rainfall to be expected in this time is, from Fig. 12, p. 51, about 6 inches, or at the rate of about 13 inches in 24 hours. The percentage may be taken at 50, giving therefore a rate of flow of $6\frac{1}{2}$ inches per 24 hours, or 174 cubic feet per second per square mile. Or, using the diagram of Fig. 13, we find that for a time value of 11 hours and a percentage of 50 the flood-flow is about 180 cubic feet per second per square mile.

For a stream of the same area but having a watershed nearly circular, the distance would be reduced to perhaps one-half the above, and the time to six hours, corresponding to a rainfall of 5 inches or a rate of $\frac{5}{6}$ inch per hour, which, with a percentage of 50, would give the high rate of 270 cubic feet per second per square mile.

72. Some Great Floods.—On March 30 and 31, 1889, there occurred a great storm over a large part of Pennsylvania and New York which caused very high floods in many streams. The great Johnstown disaster was one of the results of this storm. It was caused by floods in the South Fork of the Connemaugh River, a stream with a drainage-area of 48.6 square miles and a length of 10 miles. An estimate made by an investigating committee of the American Society of Civil Engineers* placed the maximum rate of flow at about 215 cubic feet per second per square mile. The rainfall amounted to from 6 to 8 inches on May 30 and 31, it being estimated that for several hours rain fell at the rate of $\frac{2}{3}$ inch per hour. The estimated rate of flow would be equal to one-half of this.

This same storm caused a flood in the Chemung River at Elmira, N. Y., a stream with a watershed of 2055 square miles, which was estimated by Mr. Collingwood at 138,000 cubic feet per second or 67.1 cubic feet per second per square mile.† The rainfall varied from 6 to nearly 10 inches, averaging about 7, the larger portion falling in 12 hours or less. The extreme length of the watershed is about 50 miles; the slopes are moderate, and it would probably require at least 24 hours for the maximum flood-point to be reached. On this basis the maximum flow of 67.1 cubic feet per second per square mile, or 2.5 inches per 24 hours, would be 35 per cent of the average rate of rainfall for this length of time.

On Feb. 6, 1896, great floods were caused in New Jersey by a rain of about 3.7 inches (most of which fell in 24 hours) and the simultaneous melting of snow estimated equal in amount to about 0.6 inch of rain, making a total of 4.3 inches. Of this, from 2.5 to 3 inches was discharged as flood-flow, the remainder being absorbed by the ground.‡ The total run-off was therefore about two-thirds or 66 per cent. The percentage for the first half of the flow would be perhaps 50. The effect of the snow was at first to retard the flow, but later to greatly increase it, thus virtually concentrating a large part of the precipitation into a few hours. The Raritan River, with a catchment area of 879 square miles, reached its maximum flow in about 16 hours, the rate being 68 cubic feet per second per square mile. If 3 inches represent the precipitation during this time, the flow would then be estimated, according to the method of Art. 69, at $3 \times .50 = 1.5$ inches in 16 hours, or 2.25 inches in 24 hours, equal to 61 cubic feet per second per square mile. The Passaic with a drainage-area of 822 square miles reached its maximum in 44 hours, and yielded only 22 cubic feet per second per square mile, the slowness of the rise and low maximum rate being due to extensive flats along the river. Estimating this in a similar manner, the rainfall would be the total amount, or 4.3 inches, and the flow equal to $4.3 \times .50 \times \frac{24}{44} \times 27 = 31.6$ cubic feet per second per square mile. These two examples serve to show that, while all estimates of flood-flows will be only roughly approximate, yet the method given will lead to more rational results than the application of any formula.

Data of the rise and flow of other New Jersey streams during this flood are given on the next page. They well illustrate the importance of taking account of features of a watershed other than mere extent of area.

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* Trans. Am. Soc. C. E., 1891, XXIV. p. 431.

† Report New York State Engineer, 1894, p. 387.

‡ See Report Geological Survey of New Jersey, 1896.

The decrease in flow with increase in the time required to reach a maximum is quite regular, with the exception of the Pequest, a stream having very large surface storage.

Stream	Approximate No. of Hours from Beginning to Maximum Flow.	Discharge, Cubic Feet per Second per Square Mile.	Drainage-area, Square Miles.
Pequannock	7	115	48
South Branch	8	113	67
Whippany	10	84	38
Wanaque	11	99	73
Pequest	15	13	158
Raritan	16	68	879
Pompton	16	65	285
Rockaway	16	43	118
Ramapo	24	54	160
Passaic	44	22	879

The flood on the Sudbury River caused by the great rain-storm in New England in February, 1886, is described by Mr. FitzGerald in a paper before the American Society of Civil Engineers.* The total rainfall, including snow equivalent to 2 inches of rain, from 7 P.M., Feb. 10, to midnight, Feb. 13, was estimated at 6.64 inches, of which about 5.08 inches flowed off, or about 75 per cent. The maximum rain in 24 hours was about 3 inches, and the maximum rate of flow was 1.54 inches per 24 hours, or about one-half that of the rainfall. The size of the drainage-area is about 78 square miles, and the topography an average for New England watersheds. On a neighboring stream of only 6.4 square miles of watershed the maximum rate of flow was 1.801 inches per 24 hours. The two rates were thus nearly the same, as the heavy rainfall was of sufficient duration to affect fully the larger watershed.

TOTAL FLOW FOR VARIOUS PERIODS OF TIME.

73. **Statistics of Stream-flow.**—Valuable records of run-off are available for a number of streams in the Eastern States that have been used or considered as a source of supply, but aside from these the information is meager. The most valuable of the available data are summarized in Table No. 16, in which are given the average yearly, the minimum yearly, and the seasonal flows, with corresponding rainfalls.

The characteristics of the various watersheds are briefly as follows: The Sudbury, Cochituate, and Mystic have been for many years the sources of Boston's water-supply. The Sudbury watershed is hilly and has steep slopes, but contains some large swamps; the Cochituate watershed is flat and sandy, while the Mystic is of an intermediate character. All have a very considerable percentage of forest area. The Connecticut River has a rugged watershed with about half the area

* Trans. Am. Soc. C. E., 1891, XXV. p. 253.

TABLE NO. 16.

STATISTICS OF THE FLOW OF STREAMS.

Stream.	Area Drained, Square Miles.	Years.	Average Yearly.			Year of Minimum Flow.			Average for Dec.-May.			Average for June-Nov.			Authority.*
			Rain, Inches.	Flow, Inches.	Per cent.	Rain, Inches.	Flow, Inches.	Per cent.	Rain, Inches.	Flow, Inches.	Per cent.	Rain, Inches.	Flow, Inches.	Per cent.	
Sudbury	75.2	1875–97	45.77	22.22	48.6	32.78	11.19	34.1	22.98	17.52	76.0	22.61	4.70	20.8	(19)
Cochituate	18.87	1863–96	47.08	20.33	43.2	31.20	9.76	31.3	22.97	14.87	64.7	24.10	5.46	22.7	"
Mystic	26.9	1878–96	43.79	19.96	45.6	31.22	9.32	29.8	22.11	15.12	68.4	21.66	4.84	22.4	"
Connecticut	10,234	1871–85	44.69	25.25	56.5	40.02	18.25	45.6	20.13	17.95	89.1	24.56	7.30	29.7	(13*a*)
Croton†	338.0	1870–94	48.38	24.57	50.8	38.52	14.54	37.8	23.39	17.81	76.1	24.99	6.76	27.0	(11), (17)
Upper Hudson	4,500	1888–96	39.7	23.36	59.0	33.49	17.46	52.2	18.2	16.23	89.0	21.5	7.13	33.0	(14)
Genesee	1,060	1894–96	39.82	12.95	32.5	31.00	6.67	21.5	19.58	10.20	52.2	20.24	2.75	13.6	"
Passaic	822	1877–93	47.08	25.44	54.0	35.64	15.23	42.7	22.47	18.22	81.1	24.39	7.19	29.5	(11)
Perkiomen	152	1884–97	47.98	23.62	49.2	38.67	15.66	40.4	23.27	16.52	70.8	24.71	6.98	28.3	(18)
Tohickon	102.2	1884–97	50.17	28.43	56.7	38.34	18.75	49.0	24.28	20.42	83.3	26.12	8.00	30.6	"
Neshaminy	139.3	1884–97	47.88	23.24	48.5	36.30	16.19	44.3	23.04	17.05	74.0	24.44	6.10	25.0	"
Potomac	11,043	1886–91	45.47	24.03	52.7	37.03	14.50	39.2	22.13	16.27	73.7	23.34	7.77	33.0	(8)
Savannah	7,294	1884–91	45.41	22.19	48.9	43.10	16.26	37.7	21.51	13.03	60.5	23.90	9.16	38.4	(13*a*)
Des Plaines	630	1889, 1893–95	30.56	6.61	21.6	32.38	3.19	9.9	13.01	4.94	37.9	16.30	1.14	7.0	(15)
Upper Mississippi	3,265	1885–99	26.57	4.90	18.4	22.86	1.62	7.1	......	......	......	......	......	......	(16)

* See references on p. 87.

† In his report on New York's Water-supply, 1900, Mr. Freeman concludes that the estimates of run-off for the Croton have been about 10 per cent too high.

fallow or timbered. The Croton is a hilly watershed with about 30 per cent timbered. The upper Hudson is a rugged mountainous watershed with about 70 per cent in forest. The Genesee has moderate slopes and only 25 per cent of forest area. The Passaic has 58 per cent of forest area and is of varied topography, some parts being very hilly and others quite flat. The Perkiomen, Tohickon, and Neshaminy are small streams near Philadelphia. Their watersheds are hilly, with elevations from 250 to 1000 feet high, and contain areas of timber and waste land equal to 25 per cent in the case of the first two streams, and 7 per cent in the case of the Neshaminy. The Potomac watershed has steep mountainous slopes with a large proportion of forest and waste land. The Savannah lies mostly in a rolling country with a considerable percentage of forest area. The Des Plaines, a stream near Chicago, has a watershed of very flat slopes, a considerable amount of low swampy land, and very little forest area. The watershed of the upper Mississippi is heavily wooded and nearly level. The percentage of water-surfaces on the various areas is, for the Sudbury about 3 per cent, the Cochituate 7.6 per cent, the Mystic 3 per cent, the Croton 1.8 per cent, the upper Mississippi 18 per cent, and for the others less than 1 per cent.

The very considerable variation in average percentage flowing from the different watersheds, due to differences in climate and physical features, is quite marked.

74. **Minimum Yearly Flow.**—From the data given in the table it appears that the least yearly run-off for some of the streams of the upper Atlantic coast region is only 9 or 10 inches, or about one-half the average run-off. For the Genesee it is only 6.67 inches. The data for the Massachusetts streams cover a very great drought and are considered by those of experience as being safe for future estimates in that region. That such low values of run-off have not occurred on many of the other watersheds appears to be mainly due to the fact that the rainfall has never been so low at those places during the period covered by the records. There seems to be no reason, however, why it may not at some future time be equally low.

It is important to note that in dry years the proportion of the precipitation flowing off is much smaller than the average, and, in general, the smaller the rainfall the smaller is likely to be the proportion running off. In California, for example, it is estimated by Le Conte* that in the vicinity of San Francisco, when the yearly rainfall is 10, 20, 30,

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* Trans. Am. Soc. C. E., 1892, XXVII. p. 292.

40, or 50 inches, the flow is approximately 0.5, 2, 9, 18, and 30 inches, respectively. The percentages thus vary from 5 to 60.

Like yearly rainfalls will not necessarily give like flows, as the amount flowing depends much upon the distribution of the rain throughout the year, and upon whether it is concentrated in a few large storms or is more evenly distributed. The least flow for a given yearly rain is caused by a combination of unfavorable conditions, and in making estimates such least flows are the ones to be considered. Fig. 14 represents by the shaded portion approximately the least flows

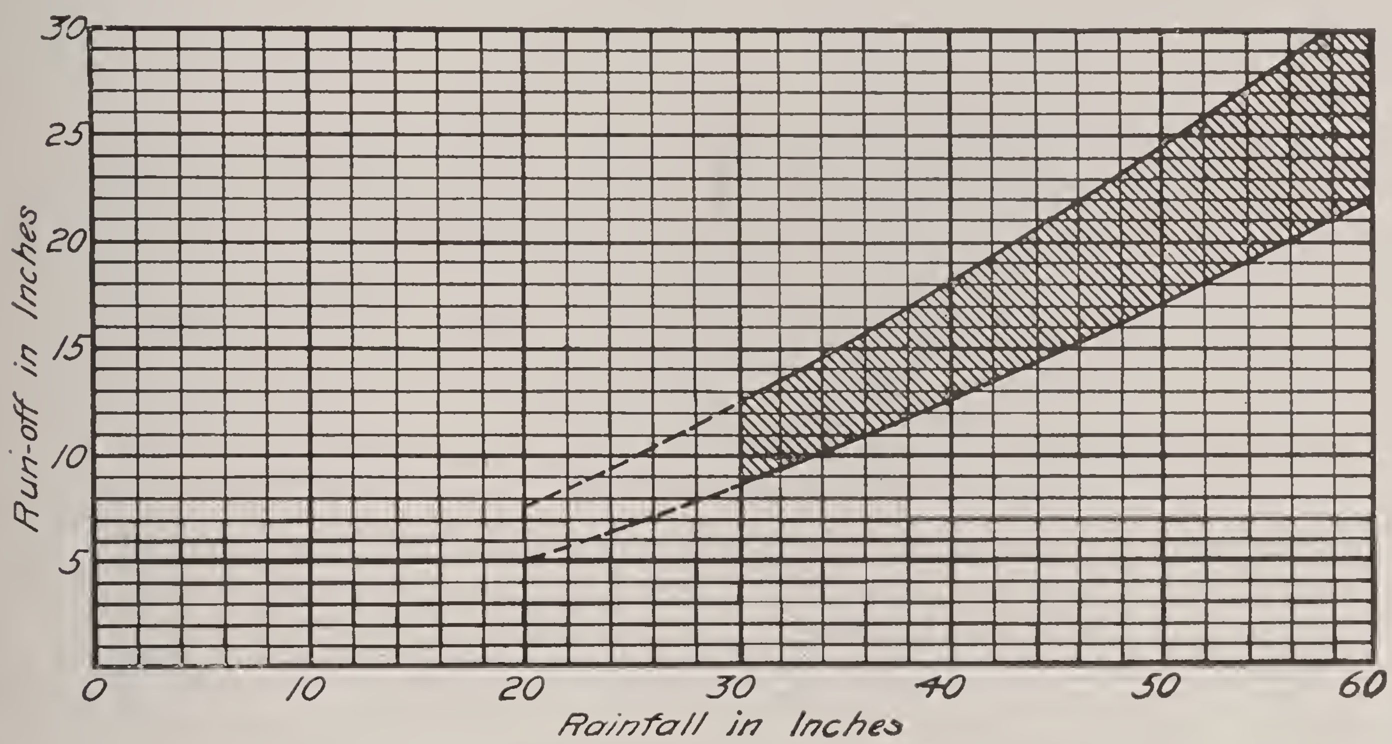

FIG. 14.—PROBABLE MINIMUM YEARLY STREAM-FLOW.

for given rainfalls for most of the streams represented in Table No. 16, as determined from the detailed statistics. The dotted portions are extensions of the curves beyond the field covered by the data. The upper limit represents such streams as the Connecticut, Passaic, and Tohickon, and the lower curve the Cochituate, while the curves for most of the other streams fall somewhere between these limits. The upper Hudson falls somewhat above the diagram, and the Genesee below. The Des Plaines and upper Mississippi fall far below. In some cases the curve for a stream will be low at one end and high at the other, as, for example, the curve for the Mystic. For very low rainfalls this watershed gives as low a yield as the Cochituate, but for rainfalls of 40 or 50 inches the run-off is much greater.

According to this diagram, the least run-off to be expected from ordinary watersheds in the Eastern States for a rainfall of, say, 40 inches would probably be somewhere between 12½ and 18 inches, depending upon the character of the watershed. For a rainfall of 30 inches the least flow to be expected would be between 7½ and 12½ inches.

For other parts of the United States having about the same general distribution of rainfall, these data and curves will be of assistance in making approximate estimates. The effect of varying yearly rainfalls will at least be similar to that shown by the curves in the figure, and with this kept in mind even one or two years of gaugings will be of much value. For localities with a much different distribution of rainfall than in the Eastern States it will be necessary to consider carefully this distribution as shown below.

75. **Monthly and Seasonal Flow.** — The average monthly flow, together with the average monthly rainfall, is shown for several streams in the diagram Fig. 15. There are also given in Table No. 16 the

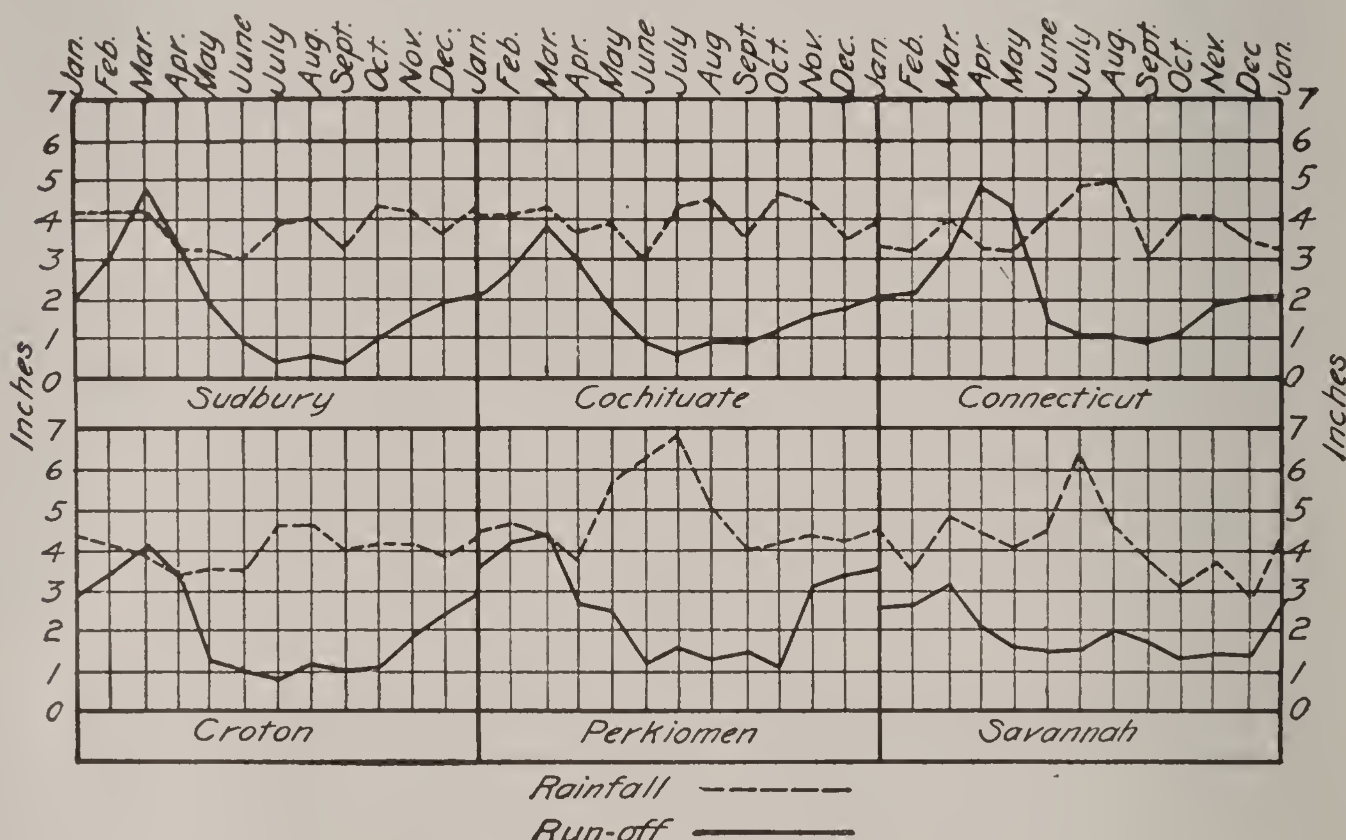

FIG. 15 —AVERAGE MONTHLY STREAM-FLOW.

average rainfall, run-off, and percentage running off for the six months from June to November, and for the six months from December to May, the former period being in general the six months of least proportionate flow, and the latter that of greatest flow. From these figures and diagrams the small value of summer rains in furnishing water to the streams is evident.

A detailed analysis for the several years shows that what was true for yearly rainfall is also true for seasonal, namely, that the less the rainfall the less the percentage flowing off. The relation between rainfall and run-off is represented approximately by the curves in Fig. 16, which is constructed similarly to Fig. 14, using here seasonal rainfall and seasonal flow. The diagrams represent by the shaded portions minimum values of run-off which may be expected for various seasonal rainfalls for the same streams as are represented in Fig. 14. They will be of some service in estimating stream-flow where the rainfall has a somewhat different distribution than upon the watersheds in question.

Detailed statistics pertaining to the Sudbury River for the years from 1879 to 1884 are here appended in Table No. 17, p. 84.* They furnish a good illustration of the variation in stream-flow from month to month, and cover the most critical period so far observed in that region.

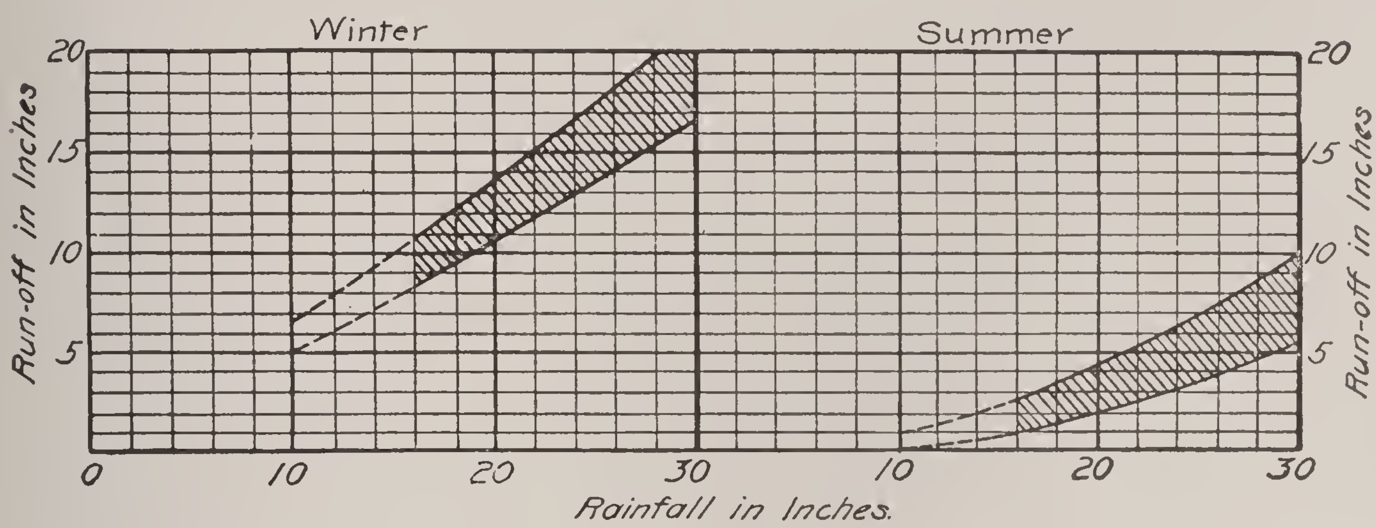

FIG. 16.—PROBABLE MINIMUM SEASONAL STREAM-FLOW.

They form the basis of storage computations mentioned in Chapter XV. The effect of the difference in the distribution of the rainfall in the years 1880 and 1882 should be noted. Compare also the data for 1880 and 1883.

76. **Estimates of Flow.**—The data in the preceding articles will enable fairly close estimates to be made of the amount of run-off for streams in the Eastern States or for regions of like characteristics. For other regions rough estimates may still be made by a judicious use of the data in connection with rainfall statistics, and by a careful consideration of the influences affecting evaporation and percolation.

In any given case the years for which estimates are required are those of least flow; and since it is not usually desirable to have impounding-reservoirs for water-supply purposes drawn below the high-water line for more than two or three years at a time, the period covering the two or three driest consecutive years is all that need be investigated. Rainfall data for such periods can be obtained from Chapter IV, and also the average distribution during the two parts of the year (Table No. 6, page 47). This being known, approximate estimates of the seasonal flows can then be made from the diagrams of Figs. 14 and 16, making allowance as far as possible for differing conditions. In making use of these diagrams it should be borne in mind

* Mass. Board of Health Reports.

that the minimum curves represent extreme conditions, such as would obtain for a single season or a single year only. In considering a number of consecutive years or seasons the values given by these

TABLE NO. 17.

RAINFALL RECEIVED AND COLLECTED ON THE SUDBURY RIVER WATERSHED, 1879–1884.

Months.	1879.			1880.			1881.		
	Rainfall.	Rainfall Collected.	Per cent Collected.	Rainfall.	Rainfall Collected.	Per cent Collected.	Rainfall.	Rainfall Collected.	Per cent Collected.
January	2.48	1.25	50.4	3.57	2.00	56.02	5.56	0.74	13.31
February	3.56	2.76	77.4	3.98	2.98	74.92	4.65	2.49	53.62
March	5.14	4.16	80.9	3.32	2.45	73.93	5.73	7.14	124.64
April	4.72	5.38	114.1	3.11	2.02	64.97	2.00	2.67	133.44
May	1.58	1.99	125.8	1.84	0.92	49.95	3.51	1.72	49.03
June	3.79	0.71	18.8	2.14	0.30	14.16	5.40	2.30	42.80
July	3.93	0.28	7.1	6.27	0.32	5.02	2.35	0.49	20.98
August	6.51	0.71	10.8	4.01	0.21	5.29	1.36	0.26	19.45
September	1.88	0.24	12.9	1.60	0.14	8.64	2.62	0.34	13.01
October	0.81	0.13	15.6	3.74	0.18	4.85	2.96	0.33	11.20
November	2.68	0.36	13.2	1.79	0.35	19.85	4.09	0.68	16.66
December	4.34	0.83	19.0	2.83	0.31	11.05	3.96	1.38	34.93
Total and averages.	41.42	18.76	45.3	38.18	12.18	31.91	44.17	20.57	46.56

Months.	1882.			1883.			1884.		
	Rainfall.	Rainfall Collected.	Per cent Collected.	Rainfall.	Rainfall Collected.	Per cent Collected.	Rainfall.	Rainfall Collected.	Per cent Collected.
January	5.95	2.21	37.19	2.81	0.60	21.25	5.09	1.78	34.91
February	4.55	3.87	85.18	3.87	1.66	43.05	6.55	4.74	72.45
March	2.65	5.06	191.16	1.78	2.87	161.42	4.72	6.75	143.06
April	1.82	1.50	82.09	1.85	2.33	126.27	4.41	4.93	111.82
May	5.07	2.30	45.48	4.19	1.67	39.96	3.47	1.84	52.97
June	1.66	0.91	54.87	2.40	0.52	21.58	3.45	0.72	20.86
July	1.77	0.15	8.70	2.68	0.21	7.68	3.67	0.40	10.89
August	1.67	0.10	5.91	0.74	0.14	19.06	4.65	0.46	9.85
September	8.74	0.53	6.05	1.52	0.16	10.36	0.86	0.08	8.87
October	2.07	0.53	25.74	5.60	0.33	5.92	2.48	0.15	5.98
November	1.15	0.36	31.51	1.81	0.35	19.52	2.65	0.30	11.44
December	2.30	0.56	24.45	3.55	0.35	9.72	5.17	1.65	31.91
Total and averages.	39.39	18.10	45.95	32.78	11.19	34.13	47.14	23.78	50.46

curves should therefore be used for but one or two such periods, much more liberal values being assumed for the remainder, or for the average.

If the watershed contains large areas of water-surface, it is important that proper allowance be made for the evaporation from such surfaces, data for which are given in Chapter V.

77. Effect of Lakes and Ponds on Stream-flow.—The result found by the preceding method takes no account of the effect of lakes and ponds acting as storage-reservoirs; the calculations would indeed often indicate a negative flow due to evaporation from excessive areas of water-surface, when in reality the flow is rendered more steady and continuous by such ponds, although the total yield may be much diminished thereby. This equalizing effect of natural ponds depends upon the amount their flow-line can be lowered, that is, upon the available storage contained therein; and can be more easily and logically taken account of in connection with the question of artificial storage. (Chapter XV.)

78. Example of Estimate of Flow.—As an example, let it be required to estimate the yield per square mile of a watershed containing 10 per cent of water-surface, having moderate slopes, and about two-thirds in meadows and under cultivation, the remainder being forest area. Suppose the rainfall distribution and evaporation to be as given for Davenport, Iowa, in Tables Nos. 6 and 11, pp. 45 and 56; but for safety we will take 0.55, 0.65, and 0.70 as the ratios to the average, of the rainfalls for the one, two, and three driest years respectively, the average rainfall being 33.3 inches. Assuming the distribution of the rainfall in dry years to be the same as the average, we will have the following rainfalls in each six months of the three consecutive years, putting for convenience the driest year second and the wettest year first:

	First Year.	Second Year.	Third Year.
Total................	26.5 inches	18.3 inches	25.1 inches
December–May......	11.1 "	7.7 "	10.5 "
June–November.....	15.4 "	10.6 "	14.6 "

By the aid of Fig. 16 we may then estimate the flow for each six-month period. On account of the flat slopes and agricultural character of the country, the area in question would not be classed higher than the poorest of those represented in the diagram. For the driest year we may therefore assume the flow according to the lowest curves, giving about 4 inches and 0 inch as the least flows to be expected for the two six-month periods of the second year. For the third year, with rainfalls of 10.5 and 14.6 inches for the two seasons, we would have run-offs of perhaps 5½ inches and 1 inch respectively; and for the first year, allowing somewhat more liberal figures, we may estimate the flow at 6½ and 2 inches.

The run-off for the driest year is so small that it is largely dependent upon ground-storage and upon the occurrence of a part of the rainfall in heavy storms. That such small flows may be expected as are here given can be seen by a reference to the data for the Des Plaines in Table No. 16. The flow of this stream, it may be noted, ceased altogether for a time in nearly every summer during the observations; and furthermore, of the 3.19 inches flowing in the year 1895, 1.80 inches flowed in the month of December, leaving but 1.39 inches for the previous eleven months.

So far the estimates are for land-surface only, or for a watershed with insignificant water-areas. To these values must now be added or subtracted the excess or deficiency of rainfall over evaporation from the 10 per cent of water-surfaces, the evaporation data being taken from Table No. 11, Chapter V. This gives a negative flow in some cases, which means that evaporation from the lakes and ponds exceeds the natural flow from the area. The various items are recapitulated in Table No. 18.

To estimate the distribution of the stream-flow throughout the various months it is sufficiently close, and as accurate as the above method of estimation warrants, to assume the excess of winter's flow over summer's flow to be all concentrated in the four months, January to April, and as uniformly distributed over these months. The remainder may be assumed as uniformly distributed over the entire year. As regards the necessary storage-volume, the exact distribution of the flow whenever it exceeds or falls short of the average consumption is of no consequence, the only matters of importance being the total amount of excess or deficiency and the time when such excess or deficiency begins. Where a negative flow occurs it is subtracted from the flow for the succeeding period, and the remainder assumed as flowing in the four months of January to April. The results are given in Table No. 18, in gallons per day for the two periods, January to April, and May to December.

Instead of using general percentages for the rainfall for dry periods, and an average distribution of the rainfall, the actual rainfalls might have been used. In either case, however, the results must be looked upon as but a rough indication of what the flow is likely to be.

TABLE NO. 18.

ESTIMATE OF FLOW FOR THREE DRY YEARS FROM ONE SQUARE MILE OF WATERSHED CONTAINING TEN PER CENT OF WATER-SURFACES.

Period.	Rainfall.	Flow from Land-surfaces.		Flow from Water-surfaces.			Net Flow.		
		Total.	90%	Evaporation.	Rainfall Minus Evaporation.		Inches each Six Months.	Gallons per Day.	
					Total.	10%		Jan.-Apr.	May-Dec.
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.			
First Year:									
Dec. to May...	11.1	6.5	5.85	11.6	− 0.5	−0.05	5.80	800,000	
June to Nov....	15.4	2.0	1.80	27.4	−12.0	−1.20	0.60	....	60,000
Second Year:									
Dec. to May...	7.7	4.0	3.60	11.6	− 3.9	−0.39	3.21	460,000	
June to Nov....	10.6	0	0	27.4	−16.8	−1.68	−1.68	....	0
Third Year:									
Dec. to May....	10.5	5.5	4.95	11.6	− 1.1	−0.11	4.84	450,000	
June to Nov....	14.6	1.0	0.90	27.4	−12.8	−1.28	−0.38	....	0

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CHAPTER VII.

GROUND-WATER.

GENERAL CONSIDERATIONS.

79. **Occurrence of Ground-water.**—In Chapter V it was shown that water precipitated upon the earth's surface in any form is disposed of in three ways: by evaporation, by surface-flow, and by percolation. In Chapter VI the stream-flow was shown to include both the surface-water and the water of percolation. In the present chapter it is proposed to deal with the last-mentioned portion more in detail, as to its quantity, movement, and availability as a direct source of supply.

Percolating water that escapes beyond the reach of vegetation must, in obedience to the law of gravitation, pass on downward until it reaches an impervious layer of some sort. The immediate impervious stratum is the surface of the water which has preceded it and which has in past ages filled every pore and crevice of the earth's crust up to a certain level at which the escape of the water laterally becomes equal to the addition from percolation. The accumulation of water which thus exists in the ground is called *ground-water*, and its surface the *ground-water level* or the *water-table*.

In limestone regions it is sometimes the case that quite large streams are found flowing underground, and large cavernous spaces may be converted into underground lakes of considerable size, as in the great caverns of Indiana and Kentucky. Such bodies of water are, however, rarely available for a water-supply, and it may be taken as a safe rule that for ground-water supplies dependence must be placed upon the water which percolates into and flows through the pore-spaces in soils and rocks, the amount of which is strictly dependent upon the rainfall and the laws of hydraulics that govern the flow.

80. General Form of the Water-table.—Under the action of gravity the surface of the ground-water always tends to become a level surface, and as long as a supply is maintained through percolation there will be a continual lateral flow which will on the average be equal to the percolation. In surface streams a very slight inclination is sufficient to cause a rapid movement of water, but in the ground the resistance to movement is so great that a relatively steep gradient is necessary to maintain even a very low velocity.

If we imagine the ground to be throughout of uniform porosity, the ground-water surface will conform in general outline to the ground-surface, but with less variations. Such an ideal condition is represented in Fig. 17. At the margin of streams the level of ground- and

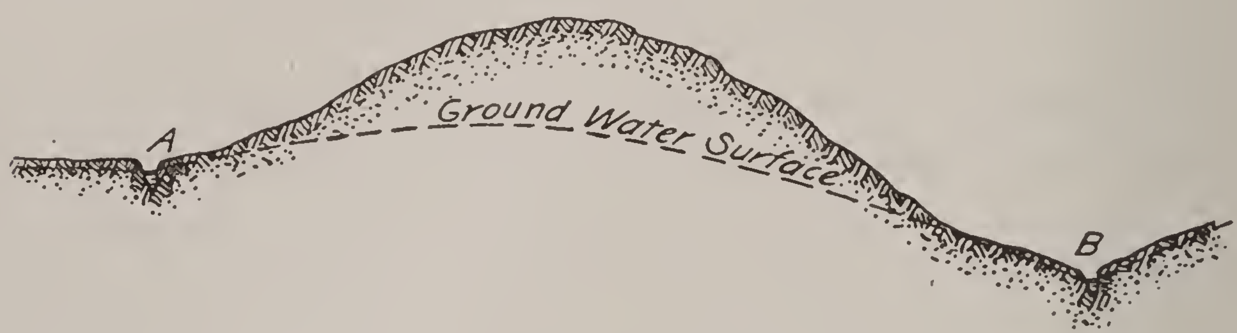

FIG. 17.—GENERAL RELATION OF SURFACE OF GROUND-WATER TO THE SURFACE OF THE EARTH.

surface-waters will coincide. Passing back from the stream the ground-water level will gradually rise, but at a less rate than the ground-surface, then descend again into another depression, etc. In the valley there is also a fall parallel to the stream, corresponding to that of the surface-water, and the direction of flow will be towards and slightly down the stream in the line of greatest declivity.

Increased percolation will raise the level of the ground-water, but less rapidly at the outlet than elsewhere, thus increasing the gradient and consequently the flow. During a period of drought the flow will continue at a slower and slower rate, due to decreased gradient, until the water ceases to flow laterally into the stream; the stream then becomes dry, and the flow continues at a slow rate parallel to the valley and entirely underground. Thus in a region where the formations are very porous and where the slopes are very steep, large streams will disappear and flow for long distances underground.

In any actual instance the ground is usually far from having the uniform porosity as assumed in the ideal example above; and the areas to be studied will ordinarily consist of alternating strata of coarse porous materials, and of fine and more or less impervious deposits.

In general the change from coarse to fine material in the direction of flow will be accompanied by an increased gradient in the ground-water level owing to the increased resistance; and conversely. If the gradient necessary to carry the quantity of water reaching the section in question is greater than the surface gradient, an overflow to the surface takes place, thus giving rise to a marsh or to a surface stream. Overflows thus occur frequently at the foot of hills where the ground-water surface is apt to be at a very small depth.

Variations in ground-water level take place comparatively slowly, following gradually the variations in yearly, seasonal, and briefer periods of rainfall. Near streams and in lowlands the level varies little, being fixed largely by the level of the adjacent surface-water. At higher points in the water-table the level is subject to correspondingly great fluctuations, often many feet in extent. In porous material where slopes are small the variations are small.

81. Porosity of Soils.—All soils and rocks near the surface of the earth are capable of absorbing more or less water.

If the particles of a body of sand or soil were of uniform size and perfect spheres, and arranged in the most compact manner, the volume of pore-space would be about 26 per cent of the total volume. Owing to irregularities in form and arrangement the porous space is usually greater than this. In sand of a fairly uniform size it is commonly from 35 to 40 per cent. Mixed sand and gravel will have a smaller percentage of voids, the decrease depending on the variation in size of particles; but it will seldom be less than 25 per cent. Rocks will vary in porosity from a very small fraction of 1 per cent in the case of some granites to 25 or even 30 per cent for some loose-textured sandstones.

The amount of moisture which a soil or rock will absorb is, however, not of so much importance to the water-works engineer as is the carrying capacity and the amount which can readily be drawn from such material when previously saturated. In fine soils the movement of the water is so slow and such a large part of the water is retained by capillary action that such soils are of little value as carriers of water; and to obtain economically the large quantities required for public supplies it is necessary that the water-bearing material be of a very open, porous character. Adequate supplies are rarely obtained from anything but sand and gravel deposits, or from very porous rock.

The absorptive capacities of various rock formations and of soils are given in Table No. 19.

82. Formations Favorable for the Transmission of Ground-water.—Rock formations are divided into two general classes, the igneous and

TABLE NO. 19.

POROSITY OF VARIOUS FORMATIONS, IN PER CENT, BY VOLUME.

Formation.	Locality.	Porosity Percentage.*	Authority.
Granite		0.02 to 1.5	
Dolomite	Joliet, Ill.	2.7	Merrill†
Limestone (oölitic)	Bedford, Ind.	9.8	"
Dolomite	Winona, Minn.	11.7	"
Sandstone	Marquette, Mich.	10.8	"
"	Fort Snelling, Minn.	13.9	"
"	Jordan, Minn.	23.7	"
"	Medina, N. Y.	4.4	"
"	Berea, O.	13.2	"
Dakota sandstone	Colorado	12 to 26	Gilbert‡
Granite	Wisconsin	0.019 to 0.62	Buckley§
Lower magnesian limestone	Bridgeport, Wis.	13.17	"
Limestone	Fountain City, Wis.	11.66	"
Trenton limestone	Duck Creek, Wis.	1.04	"
Niagara limestone	Sturgeon Bay, Wis.	0.58	"
Potsdam sandstone	Dunnville, Wis.	28.26	"
" "	Lake Superior, Wis.	20.19	"
" "	Ablemans, Wis.	7.12	"
St. Peter sandstone	Argyle, Wis.	19.06	"
Limestone (oölitic)	England	14 to 17	Humber‖
Chalk	"	14 to 44	"
Sand		35 to 45	
Sand and gravel		25 to 30	
Sandy soil	South Carolina	41.8	Whitney¶
Truck land	Maryland	37 to 45	"

* The figures from Merrill were obtained by multiplying his "ratio of absorption" by the specific gravity of the stone.

† Stones for Building and Decoration. New York, 1897.

‡ Report U. S. Geolog. Survey, 1895–96, p. 584.

§ Bulletin No. 4, Wisconsin Survey, 1898.

‖ Water-supply of Cities and Towns, p. 47.

¶ Bulletin No. 4, Weather Bureau, 1892, p. 25.

the sedimentary rocks. To the former class belong the granites, syenites, and gneisses; these rocks are usually very dense and impervious and therefore poor water-carriers, but occasionally they may furnish considerable water by virtue of their decomposed and fissured condition near the surface. Of the sedimentary rocks, those composed of very fine-grained material, such as the clays, shales, and other argillaceous deposits, are relatively impervious. Limestones and dolomites contain little water as a rule, but if fissured they may be the source of considerable supplies. The most favorable formations, by far, for furnishing large quantities of water, are the various sandstones, conglomerates, and gravel deposits. Sandstones are found which vary in texture from a very compact rock having a very small degree of porosity to a material almost as porous as sand. Uncemented sands and gravels are of course the most favorable as regards porosity, but they are apt to be rather limited in extent.

83. Occurrence of Water-bearing Formations. — In studying the various water-bearing formations from the engineer's standpoint, it will be convenient to divide them roughly into three classes, depending upon their extent and outline.

(1) *Broad, extensive formations of porous material, usually of considerable thickness and of a fairly uniform character over large areas.* In the case of formations of this class comparatively few widely scattered borings will often serve to give a reliable knowledge of the strata, and wells may be sunk many miles apart with confidence as to the result. Most of the deep and artesian water of the United States is obtained from such formations, some of which underlie great areas of country. In England an example of such a formation is the immense deposit of chalk which underlies a large part of the country and furnishes much water for public supplies. The Tertiary deposit of sand and gravel underlying the marl throughout a large portion of the Western plains is supposed to have once been the bed of an inland sea. The so-called underflow, or ground-water of the plains, lies chiefly in this formation, and wells sunk to this stratum at any point are unfailing. These deposits are estimated at from 17 to 120 feet thick. Other examples of extensive water-bearing formations are given in Arts. 101–103.

(2) *Deposits of porous material in old lake- and river-beds* often furnish very good collecting-areas for ground-water, and many of the

FIG. 18. — IDEAL SECTION OF SAN JOAQUIN VALLEY, CALIFORNIA.

shallower ground-water supplies are from such sources. These deposits are usually covered by other and less pervious strata, and indeed often consist of a series of strata alternately of a pervious and nonpervious nature. This is particularly true of the lacustrine deposits in the basins of the Western mountain region. Fig. 18 is an ideal section through such a basin in California and shows many alternate layers of clay and gravel.*

* Report on Irrigation, Part I. p. 321. (U. S. Pub. Document.)

Old river-channels filled with débris of a porous character give rise to veritable ground-water streams. These may be located some distance from the modern streams, or may at places coincide with or underlie them, forming porous, gravelly beds.

Examples of such ground-water streams are very numerous. Leipsic, Germany, is supplied from such a stream about 2 miles in width, 40 feet thick, and having a fall of about 6 feet per mile. The covering stratum is 6 feet thick, and the velocity of flow is estimated at about 8 feet per day.* Pueblo, Colorado, is supplied with water from a gravel-bed 66,000 square feet in cross-section with an average depth of 14 feet and a length of 25 miles. This deposit fills the former bed of a stream which now flows partly through and partly over the surface of the gravel.† Many of the Western streams where they emerge from the mountains are of a similar character.

(3) *Deposits of sand and gravel in the drift* are often of considerable extent, and furnish many ground-water supplies, but such deposits are apt to be very irregular in character and uncertain in extent. They occur as accumulations in former stream-beds and also in the form of thin, irregular strata, sometimes of considerable extent, lying for the most part in valleys and covered with more or less clay.

Still another formation of much value in certain localities is the dune-sand, such as occurs so extensively in Holland and from which many of the water-supplies of that country are drawn.

In seeking ground-water supplies a study of the geology of the region is essential to intelligent action. Such a study will generally enable a decision to be made as to whether or not a supply is likely to be obtained from the deeper strata, and will give much information as to the nature of the glacial or other surface deposits. The location of extensive deposits in valleys is often shown by wells in the vicinity, but at other times they can be located only by a careful study of surface indications and by borings.

FLOW OF GROUND-WATER.

84. Methods of Determining the Flow of Ground-water.—When a particular ground-water source is to be investigated for a water-supply, the same question must be answered as in the case of a surface supply, namely, what is the quantity of water available from day to day from the given source? In the case of a surface stream the rate of discharge

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* *Jour. f. Gasbel. u. Wasservers.*, 1881, p. 686.

† *Eng. News*, 1891, XXV. p. 53.

is determined by multiplying the observed velocity by the cross-section of the stream, and such observations carried on for a considerable length of time will give the necessary information. In the case of a ground-water supply similar determinations would be desirable, but they are much more difficult to make.

The best method of estimating capacity is by means of actual pumping tests carried on for a sufficient length of time to bring about an approximate state of equilibrium between the supply and the demand as determined by the level of the ground-water. It will rarely be practicable to continue such tests until perfect equilibrium is reached, for in many cases several years of operation would be required to determine the ultimate capacity of a source. Pumping tests of short duration are apt to be very deceptive, as the ground-water may exist in the form of a large basin or reservoir with very little movement, corresponding to a surface pond with small watershed, and brief tests would give but little more information than similar tests on a pond.

Where it can be done it is very desirable to get an approximate idea of the amount of water actually flowing per unit of time through the area in question. This may be done by estimating the velocity of flow, the cross-section of the porous stratum, and the percentage of porous space; or an approximate estimate can sometimes be made by estimating the probable percolation on the tributary area.

85. Formula for Estimating Velocity of Flow.—The velocity of flow of a ground-water stream is a function of the hydraulic gradient, or slope, on the one hand, and the resistance to flow offered by the particles of soil on the other.

The slope can readily be determined by borings sunk to ground-water level, care being taken to measure it in the direction of greatest declivity. In case the porous stratum is overlaid by a more or less impervious one the water in the lower stratum may flow under a pressure greater than atmospheric. The slope or hydraulic gradient is then found by determining the height to which the water will rise in tubes sunk to the porous stratum, care being taken to prevent the escape of water between the tube and the upper strata. Samples of the material can also be obtained at the same time and examined as to size and porosity. The latter element is influenced not only by the variation in size of grain, but also by the degree of compactness of the material in its natural bed; it can therefore be only approximately determined from loose samples. The size can readily be determined by means of sieves.*

* For further details relating to sand analysis, see Art. 511.

These elements having been determined, it remains to express the relation between them and the velocity of flow.

Experiments by Darcy, Hagen, Hazen, and others show that the rate at which water at a given temperature will flow through any particular sand or fine gravel follows closely the law of flow through capillary tubes, that is, the velocity is approximately proportional to $\frac{h}{l}$, where h is the head, and l is the distance through which the water flows. (In the case of a stream flowing through such material down a uniform slope, $\frac{h}{l}$ would be the sine of the slope angle.)

For different grades of material the velocity depends primarily upon the size of the pore spaces contained therein. This is a function of the size of the grains of the material and the degree of compactness with which they are arranged. For a sand of uniform size and of a given degree of compactness it is found that the velocity of flow is closely proportional to the square of the diameter of the sand grain; and, expressing the degree of compactness in terms of the percentage of pore space, it is found that for materials of the same size the velocity of flow through the pores is roughly proportional to the square of the porosity ratio. The *volume* of flow, which is equal to the velocity multiplied by the area of net section, is thus proportional to the cube of the porosity.*

Natural sands and gravels vary greatly in character, consisting of material of many different sizes and of different degrees of porosity as a result of differences in compactness and of differences in the proportions of large and small grains. These conditions render it difficult to apply mathematical formulas or to reduce experimental results to a working basis. The results obtained by Hazen from experiments on filter sands † are probably the most widely known in this field. The formula derived by him as applicable to sands of from 0.1 to 3.0 mm. effective size is

$$v = cd^2 \frac{h}{l}\left(\frac{t\ (\text{Fah.}) + 10^\circ}{60^\circ}\right), \quad \ldots \ldots \quad (1)$$

where v = velocity in meters daily of a solid column of the same cross-section as that of the sand;
c = a constant = 400 to 1000;
d = effective size of sand grains in millimeters;
h = head of water causing motion;

* Slichter, W. S., Paper No. 67, U. S. G. S., 1902; also 19th annual report U. S. G. S., Pt. II. 1899, p. 295.

† Report Mass. Board of Health, 1892, p. 553.

l = thickness of sand layer $\left(\frac{h}{l}\right.$ = slope of ground-water surface$\left.\right)$;

t = temperature in degrees Fahrenheit.

The "effective size" is a very important element in the formula. In the natural material, consisting of coarse and fine particles, it is obvious that the size of the pore space is chiefly determined by the size of the finer particles, and that a small per cent of fine particles will cause an otherwise coarse sand to become essentially a fine sand so far as the transmission of water is concerned. As the result of experiments it was concluded by Hazen that the maximum size of the finest 10 per cent of the material represented fairly well the "effective size" of the sand as a whole; that is, the "effective size" is the size of grain, such that 10 per cent of the particles are smaller and 90 per cent are larger than this size. To express variations in proportions of large and small particles a "uniformity coefficient" was devised. This is the ratio of the size of grain such that 60 per cent of the sand is finer than this size, to the "effective size" above described. Ordinary sands will have uniformity coefficients of from 1.5 to 2.5. The analysis of a sand may be made by sieves as more fully described in Art. 511.

The value of the constant c in Eq. (1) varies with the compactness and uniformity of the sand. For new clean sand of a fairly uniform character, it varies from 700 to 1000; for old compacted sand it may be as low as 400.

Assuming a porosity of 40 per cent, the actual average velocity of flow through the pore spaces will be 2.5 times that given in Eq. (1). Neglecting temperature corrections, as being, in any case, small for ground-waters, we derive the following value of velocity in foot units,

$$v = 8.2\, cd^2 s = k s \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

where v = actual average velocity through the pores of the sand in feet per day;

d = effective size of sand;

s = slope of free ground-water surface, or the hydraulic gradient;

$k = 8.2\, cd^2$ = velocity for a slope of unity.

A sand of an effective size of 0.10 mm. would be called a very fine sand, one of 0.3 a medium sand, and one of 0.5 a very coarse sand, although much depends on the uniformity.

In estimating the value of k it is to be noted that the coefficient c varies considerably with the porosity. Professor Slichter, in the papers already referred to, calculates the following values of relative velocities of flow in material of the same size but of varying porosities due to different degrees of compactness:

Porosity, per cent	25	30	35	40
Relative velocity	34	52	74	100

Taking a value of c equal to 1000 for a porosity of 40 per cent and reducing it in accordance with the above values for lower porosities, we derive the approximate values of k given in Table No. 20.

TABLE NO. 20.

VALUES OF k IN EQ. (2) FOR VARIOUS VALUES OF d AND FOR VARIOUS POROSITIES.

Porosity, Per cent.	(d) Effective Size of Sand in Millimeters.									Porosity, Per cent.
	.10	.20	.30	.40	.50	.80	1.00	2.00	3.00	
25	28	112	251	446	697	1,780	2,790	11,150	25,100	25
30	43	171	384	681	1,066	2,730	4,260	17,050	38,400	30
35	61	243	546	970	1,517	3,880	6,070	24,270	54,600	35
40	82	328	738	1,312	2,050	5,248	8,200	32,800	73,800	40

For sands of low uniformity, and mixtures of sand and gravel, the velocity will still depend on the size of the finer particles; and if the larger stones are neglected in the estimation of the effective size, the above formula may still be used as an approximation.

Lembke suggests values of k as follows, based on experiments of Darcy, Kröber, and others:*

Material.	k in Feet per Day.
Sand and gravel	9,400
Coarse sand	2,800
Medium sand	760
Fine sand	150

In Professor Slichter's investigations the effective size was determined by measuring the flow of air through a sample of the material by means of King's aspirator.† This method of determining the effective size probably gives more accurate results than any other method yet devised, but it is not in general use. Comparing the work of Hazen and Slichter it would seem that the size reported as the effective size of a given sand is somewhat smaller in the former case than in the latter.

* *Revue Univ. des Mines*, 1888, I. p. 155.

† Fifteenth Ann. Rept., Agr. Exp. Sta., University of Wisconsin, 1898, p. 123.

86. *Coefficients for Coarse Gravels.* — For gravels larger than 3 mm. and containing little or no fine material, experiments indicate that the velocity increases at a less rate than the square of the diameter, and also less rapidly than the slope. Results of such experiments on screened gravel are given in Table No. 21 as indicating in a general way the variation in velocity in coarse gravel deposits.

TABLE NO. 21.

VELOCITIES OF FLOW OF WATER IN FEET PER DAY IN SCREENED GRAVEL, ASSUMING 40 PER CENT POROSITY. BASED ON EXPERIMENTS OF THE MASSACHUSETTS STATE BOARD OF HEALTH.*

Slope, s.	Effective Size of Millimeters.									
	3	5	8	10	15	20	25	30	35	40
.0005	28	82	164	246	410	656	902	1,230	1,640	2,050
.001	57	172	335	475	820	1,210	1,680	2,250	3,030	3,690
.002	115	328	639	902	1,550	2,250	3,030	3,930	4,830	5,820
.004	221	631	1,230	1,700	2,870	3,930	5,000	6,060	7,130	8,200
.006	336	918	1,690	2,250	3,690	5,080	6,390	7,620	8,930	10,100
.008	443	1,160	2,060	2,780	4,340	5,900	7,380	8,930	10,400	11,800
.010	549	1,410	2,460	3,150	5,000	6,800	8,440	10,000	11,500	...

87. **Direct Method of Determining Velocity of Flow.**— The rate of flow of ground-water may be directly determined by tracing the movement of a soluble salt introduced into the ground-water stream. The first to employ this method for this purpose was probably Thiem of Germany, who has studied the flow of ground-water at several places with good results.† His method is as follows:

Three or four borings are sunk to ground-water in a line in the direction of flow. A large dose of salt is then put into the upper hole, and at frequent intervals analyses are made of water drawn from each hole below until the salt content has reached its maximum in each case, and the rate of movement is inferred from these results.

At Stralsund, velocities of 12.9, 12.6, and 12.0 feet per day were found in this way, with a slope of 2 per cent; and a velocity of 13.1 feet at another place. The former values would correspond to a value of k equal to about 625, or to that for a medium sand.

A much more expeditious method is that developed by Professor Slichter.‡ In this method the movement of salt is determined by electrical means in a very convenient way. The arrangement of appar-

* Report, 1892, p. 555.

† *Jour. f. Gasbel. u. Wasservers.*, 1888, p. 18.

‡ W. S. Paper No. 67, U. S. G. S., 1902, p. 47. *Eng. News*, 1902, XLVII. p. 151.

atus is shown in Fig. 18a. Two small drive wells are sunk three or four feet apart and in the line of flow, if this is known. Both wells are provided with brass strainers, through which the ground-water may enter readily. An electric battery with ammeter is connected to the wells as shown. One terminal is connected to the casing of the upper well and also to an electrode of brass inserted in the lower well and insulated from the casing of this well. The other terminal is connected to the casing of the lower well. An electrolyte is introduced in a single dose into the upper well. As this passes towards the lower well with the ground-water the amount of current passing from casing to casing will gradually increase. When the electrolyte reaches the lower well and enters it, a short circuit will be

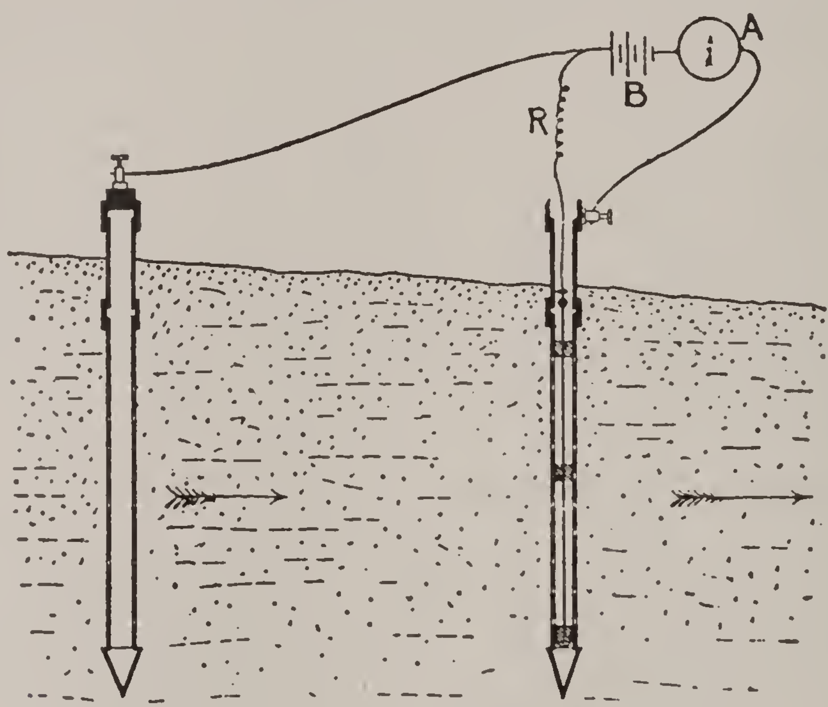

FIG. 18a.— ARRANGEMENT OF WELLS FOR DETERMINING VELOCITY OF FLOW.

created between the interior electrode and the well casing and there will be a sudden increase in current.* Fig. 18b illustrates a typical curve thus obtained. The point "A" represents the instant when the electrolyte was introduced into the upper well; the point of inflection of the steep part of the curve at "B" represents the time when the electrolyte reached the lower well. Except for the effect of diffusion the steep portion would be a vertical line. The portion of the curve to the left of the steep part shows a slow increase in current passing from casing to casing. This information assists in estimating the regularity of flow and is especially valuable when the electrolyte entirely misses the lower well through an erroneous estimate of the direction of flow. Where this occurs additional wells may be sunk until one is obtained in the line of flow.

* From W. S. Paper No. 67, p. 49.

The most convenient electrolyte seems to be ammonium chloride. Where the velocity is very high caustic soda has been added in order to cause greater diffusion, and thus to make it more certain that the direct effect will be felt in the lower well if somewhat out of line.

Measurements of velocities of the underflow of several of the western streams have been made by Professor Slichter. Velocities of from 5 to 10 feet per day are common, while velocities as high as 50 feet per day have been observed in a coarse deposit with a slope of 20 feet per mile.* Average velocities of about 4 feet per day have been measured on Long Island.

88. Quantity Flowing.—The velocity of flow having been determined, also the porosity of the material and the cross-section of the

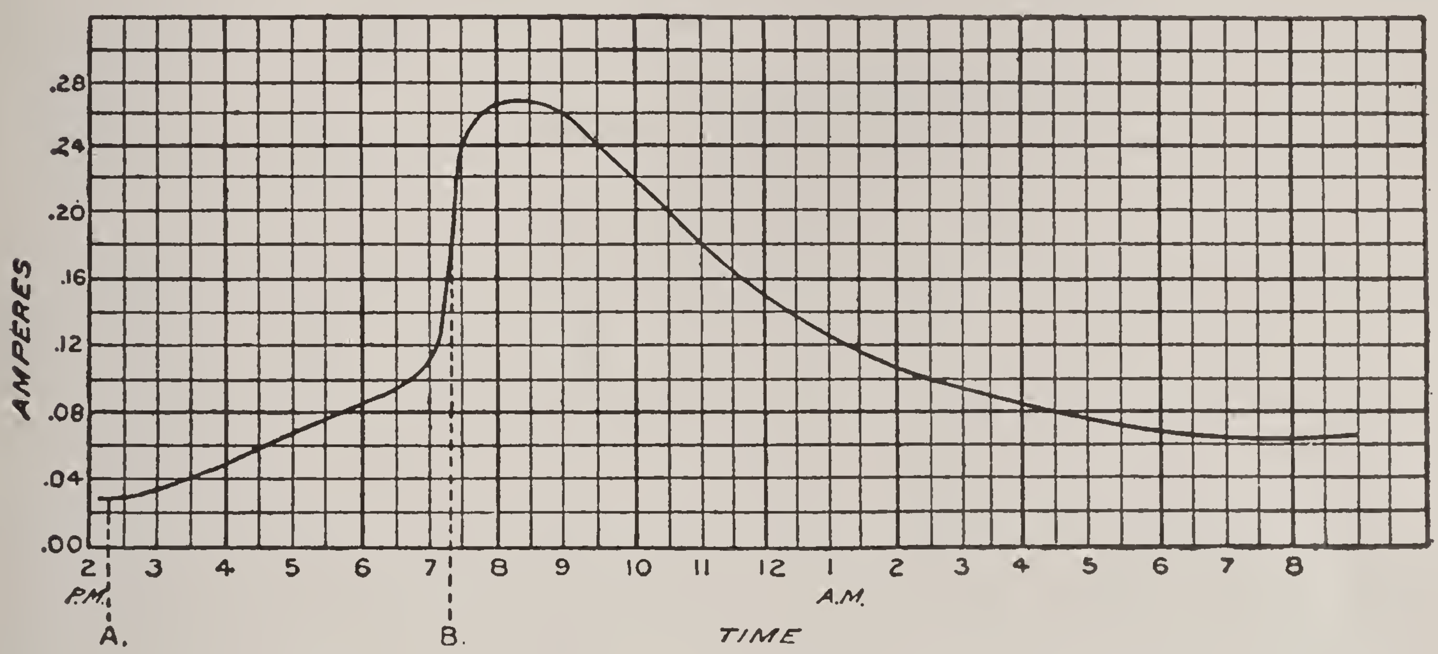

FIG. 18b. — CURVE SHOWING TRANSMISSION OF ELECTROLYTE.
(From W. S. Paper No. 67.)

porous stratum at right angles to the direction of flow, the total rate of flow will be the product of these three factors, or

$$Q = \text{velocity} \times \text{area of cross-section} \times \text{porosity} = vAp = ksAp, \quad (3)$$

in which the units are the foot and day. Irregularities in cross-section, slope, and material will of course render the result more or less uncertain, but estimates made in this way will nevertheless be of very considerable value in examinations of ground-water sources, and will tend to modify the very exaggerated notions which frequently prevail concerning their capacity.

* The river Mohave, Jour. West. Soc. Eng'rs., 1904, IX. p. 635.

In Table No. 22 are given the rates of flow in sands of different degrees of fineness and porosity, and for a slope of 1 per cent. For other slopes multiply by the slope expressed in per cent. In this table the rates of flow for porosities below 40 per cent have been reduced in accordance with the coefficients of Art. 85.

An inspection of this table will show clearly that it requires very extensive areas and collecting-works to obtain much ground-water from fine material; and even with coarse material, if the slopes are flat (they are frequently only one-tenth of 1 per cent), a relatively large cross-section must be available.

TABLE NO. 22.

RATES OF FLOW OF GROUND-WATER FOR A ONE PER CENT SLOPE ($s = .01$) IN GALLONS PER DAY PER SQUARE FOOT OF CROSS-SECTION.

Porosity, Per cent.	(d) Effective Size of Sand in Millimeters.									Porosity, Per cent.
	.10	.20	.30	.40	.50	.80	1.00	2.00	3.00	
25	0.51	2.6	4.7	8	13	33	52	208	470	25
30	0.90	3.8	8.6	15	24	61	96	383	862	30
35	1.6	6.4	14.4	25	39	102	160	635	1,430	35
40	2.4	9.8	22.1	39	61	157	246	980	2,210	40

89. **Quantity Available.**—The proportion of the ground-water that can be intercepted depends upon the character of the collecting-works, a question which will be discussed in Chapter XIV. The useful capacity of such a supply — the quantity which it can deliver daily throughout the year — depends upon the minimum rather than the average flow, and in determining the flow a dry period should be selected if possible. The natural storage furnished by the ground not only renders the flow ordinarily quite uniform, but enables the draught to more or less exceed the minimum flow; and if the character of the works is such that the ground-water level can be considerably lowered, this natural storage can be made to increase very materially the daily capacity of the source. In estimating the capacity of a source by estimating the percolation, this element of ground-storage must be taken into consideration.

SPRINGS.

90. **Formation of Springs.**—Springs are formed where, for any reason, the ground-water is caused to overflow upon the surface. The

conditions causing their formation are varied and should be carefully studied in connection with the design of collecting-works, as upon them depend largely such questions as the constancy of flow, the possibility of increasing the yield by suitable works, and the probable success of a search for additional springs. According to differences in these conditions springs may be divided into three general classes, each of which will be discussed separately.

91. *First Class.*—The most important class of springs is that in which the water, in its lateral movement, is brought to the surface at the outcrop of a porous stratum where it is underlain by a relatively impervious one (Fig. 19). The porous stratum may be sand or gravel,

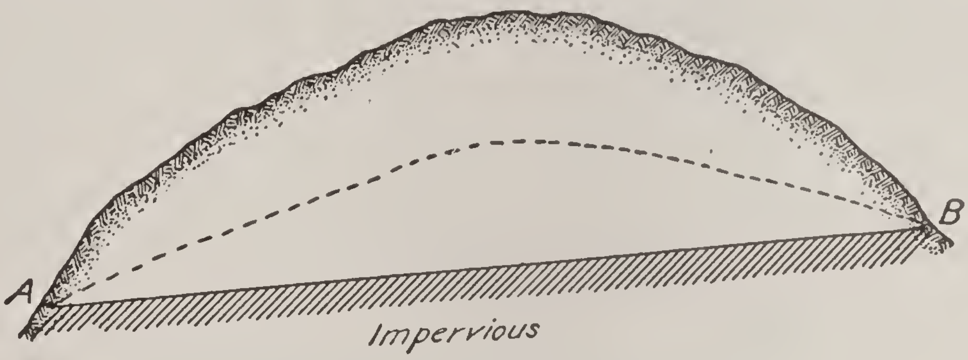

FIG. 19.

or a porous rock; while the impervious layer is usually clay, or rock of an argillaceous character.

If the porous material is fairly uniform, the springs will be scattered all along the outcrop and will be small in size, the larger amounts of water appearing in the valleys or re-entrant angles of the outcrop. If the porous deposit be much fissured, especially if the rock itself be fine-textured, the location of a spring is largely a matter of chance, although topography controls in a general way.

There are many cases of large springs of this class, the supplies for some of the largest cities of Europe being obtained from such sources. The city of Vienna is supplied from springs 60 miles distant that occur at the outcrop of a fractured dolomitic limestone underlain by slate. The largest spring, the Kaiserbrunnen, has an average flow of about 150 gallons per second, varying from 60 to about 250.* Munich receives its supply from galleries constructed in fissured slate, which collect the ground-water that previously appeared in part as springs at the surface of the slate, and in part flowed through fissures into the river below. Baden-Baden intercepts, by means of a gallery about 2 miles long, several springs occurring at the junction of granite and overlying sandstone. The flow varies from 4 to 18 gallons per second, averaging about 8.†

* Lueger, p. 410.

† *Ibid.*, pp. 275, 400.

In the United States many supplies of considerable amount are obtained from similar springs, the most noteworthy instance being perhaps that of Roanoke, Va. The supply there is from a spring issuing from the limestone and having a flow of about 5,000,000 gallons per day.

92. *Second Class.*—Under this class are considered those springs where the water-bearing stratum is covered to a greater or less extent by an impervious one, and which are therefore more or less artesian in character. In this case the water finds its way to the surface where the overlying impervious material is wanting, or through a fault, or it breaks through at places where it is not sufficiently strong or compact to resist the upward pressure. Most of the springs which occur in the drift are of this character, the alternating layers of sand and clay so often found there being favorable to their formation.

In Fig. 20 is given a section showing the formations immediately

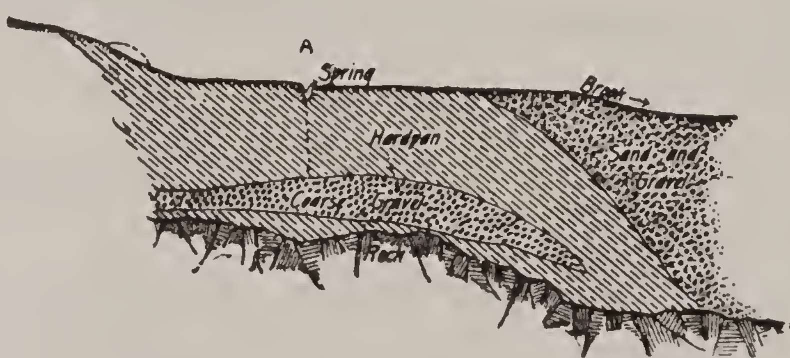

FIG. 20.—SPRING AT AVON, MASS.

surrounding a spring of this kind, located at Avon, Mass.* Here the water is carried by coarse gravel which is overlain by hardpan. The location of the spring at this point was doubtless due to some local weakness. The outcrop of the porous stratum lies considerably higher than the spring.

In some cases springs of this character are fed by water coming long distances through extensive formations which at other points offer conditions favorable for artesian wells. Of such character are the artesian springs at the eastern outcrop of the Dakota sandstone (Art. 102). Conditions of this sort also give rise to the peculiar phenomenon of large fresh-water springs which boil up in the ocean several miles out from the Florida coast, and it is supposed that the great springs in northern Florida are from a similar cause.

93. *The Third Class of Springs* includes those in which the porous

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* *Jour. New Eng. W. W. Assn.*, 1896, XI. p. 160.

stratum in the vicinity of the spring is neither overlain nor immediately underlain by an impervious one. They are mere overflows of the ground-water, and occur whenever the carrying capacity of the porous material is insufficient to convey the entire tributary flow.

In a region where the soil is very porous to a considerable depth, the surface-flow of streams will commence only at a considerable distance from the head of the valley, the point of beginning being a spring of larger or smaller size of the class under discussion. If the formation is quite uniform, the springs will be small and numerous, and the source of the stream will move up and down the valley according to the weather, the point of beginning being determined by the carrying capacity of the ground. If the formation is irregular, the springs tend to be larger in size. In irregular formations it also often happens that after having flowed on the surface for some distance the water will again disappear, only to reappear farther down the valley. Such action is noticeable in almost any small brook, but in certain parts of the country it occurs on a very large scale. Where springs are thus formed by water that has recently flowed on the surface the character of the water is likely to differ greatly from ground-water proper.

94. **Yield of Springs.**—The yield of any particular spring can readily be determined by weir measurements, and if these are carried out through a period of drought they will give all needed information regarding the supplying capacity of the existing spring. If, however, but a short series of gaugings is available, it will be necessary to make allowances for variations in rainfall; and a knowledge regarding the area of percolation, quality of soil, and possibilities in the way of ground-storage will be of assistance in drawing conclusions. The possibility of increasing the flow should also receive attention.

Springs of the first class will vary in yield with the variations in ground-water level, but will not wholly cease to flow if the water is intercepted by suitable constructions. The yield of a series of springs of this class would, if the lower stratum be impervious, be equal to the entire percolation on the tributary area. This area is determined by the direction of the slope of the ground-water surface, and does not always correspond with the watershed for surface-water. Thus in Fig. 19 more water will appear at *A* than at *B*.

Springs of the second class are apt to be much less affected by variations in rainfall than either the first or the third class. Their yield varies with the variation of the ground-water level in the area of percolation, and if this is many miles distant, as is the case with many artesian wells or springs, the flow may be practically invariable. In most cases, however, the deposits are local in extent and the variation is considerable.

Where a spring of this class exists, investigation may show that the ground-water stream from which it is fed is of considerable size and that the water of the spring is but a small portion of the entire flow. In such a case the yield may be increased by simply enlarging the opening, or by sinking wells and pumping therefrom, as in the case of an ordinary ground-water supply.

Springs of the third class are liable to very great fluctuations, the flow often ceasing entirely. Occasionally, owing to the concentration of large volumes of ground-water into a small area, conditions are favorable for large and steady yields. Springs of this class form the source of the Vanne, from which Paris draws a portion of its supply. The largest of these, "Le Bîme de Cerilly," has an average yield of about 50 gallons per second, with a minimum of 18 gallons.

ARTESIAN WATER.

95. General Conditions.—Whenever a water-bearing stratum dips below a relatively impervious one the former becomes in a sense a closed conduit, and if the flow out of this conduit at the lower end be impeded from any cause, the water will accumulate and exert more or less pressure against the impervious cover. The amount of this pressure will depend on the extent to which the flow is obstructed and on the elevation of the upper end of the conduit, that is, of the outcrop of the porous stratum. If a well be sunk through this impervious

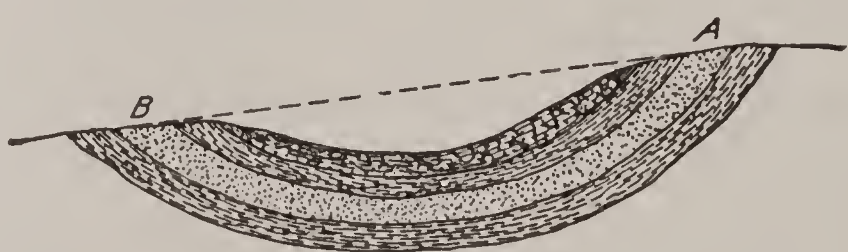

FIG. 21.

stratum at any point, the water will rise in it in accordance with the pressure; and if the surface topography and pressure are favorable, the water may rise to the surface, or considerably above, in which case the well becomes a true artesian, or flowing, well.

The obstruction to flow at the lower end of the porous stratum may be due to various causes, chief among which are the three following:

1. The stratum may be turned up at the lower end, thus forming a synclinal, or a curved conduit, as in Fig. 21. In this case water entering at *A* could escape at the lower lip *B*, but at intermediate

points would exert a pressure on the covering. If the resistance to flow were uniform, and no water could escape except at B, the decrease of head from A to B would be uniform, or in other words the hydraulic grade-line would be a straight line AB. Water would rise to this line in a tube sunk to the porous stratum, and a flowing well would be possible wherever the surface lies below this line.

2. The inclined stratum may be subjected at its lower outcrop to hydrostatic pressure from the waters of the ocean, as actually occurs along a large part of the Atlantic and Gulf coasts of the United States. The conditions obtaining in that region are roughly shown in Fig. 22.

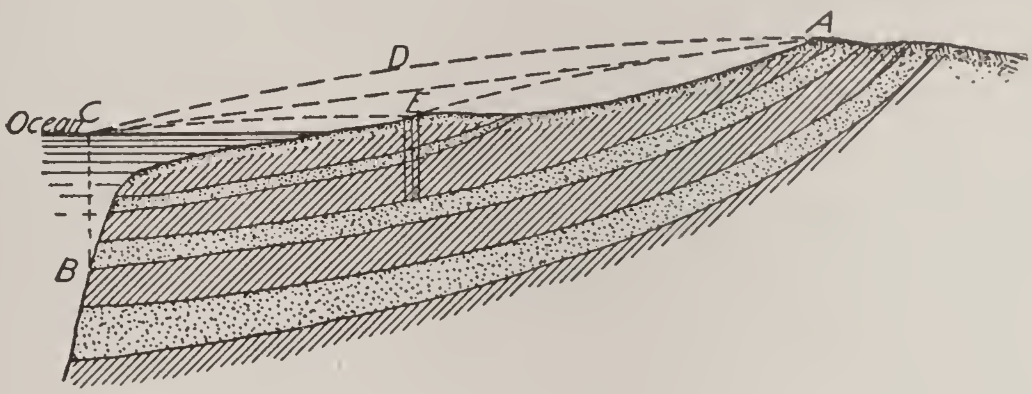

FIG. 22.—ARTESIAN CONDITIONS NEAR OCEAN.

In this case a porous stratum outcropping at A and passing into the ocean at B would be subjected to a pressure throughout its length, varying according to some hydraulic grade-line AC. Flowing wells are here possible at all points where the surface falls below this line.

3. An increased resistance to flow is frequently caused near B, Fig. 22, by increased density of the stratum or by a decrease in thickness. Such increased resistance will have the effect to increase the slope of the hydraulic grade-line at the point of greater resistance, and give it a form something like the line ADC, thus making conditions still more favorable for flowing wells. Complete stoppage at B and no leakage would give a horizontal grade-line through A. Leakage through the overlying strata, or flow through many wells as at E, will reduce the pressure and consequently lower the grade-line to AEC. The local effect of wells upon each other is more fully discussed in Chapter XIV.

96. Use of the Word "Artesian."—The term "artesian" was formerly applied exclusively to flowing wells and is derived from the word "Artois," the name of a province in France where such wells were first extensively bored. More recently, however, the term has come to be applied in a broader sense, according to which an artesian well may be defined as one in which the water is drawn from a porous stratum underlying a relatively impervious one and so located that the contained water, drawn from a distant elevated outcrop, naturally exerts more or less pressure upon the overlying cover. Water will rise in such wells, but whether it will overflow depends much on local conditions, such as elevation of surface, and nearness of other wells. Many wells once flowing have ceased to flow owing to increased draught by others, and wells but a few hundred feet from others sunk to the same stratum will exhibit variations in this respect; but it is still convenient to call all such wells artesian, and the water artesian water.

97. **The Character and Inclination of the Strata,** both of the porous stratum and the impervious cover, largely determine the capacity and usefulness of the artesian area. The important water-bearing formations of an artesian character belong to the sedimentary rocks, but small areas of considerable local importance are met with in the drift formation. The water-bearing stratum is most often a porous sandstone, although artesian water is also obtained from limestone and in many places from extensive strata of loose uncemented material.

The overlying impervious strata usually consist of clays and shales, these being practically impervious except where fissured. Probably some leakage always takes place through such strata; but a condition favorable to small leakage is the existence of an elevated country between the outcrop and the area where wells are practicable. In such a case the ordinary ground-water level is likely to be above the hydraulic grade-line of the artesian basin, and the leakage would then tend to be into rather than out of the confined stratum.

To be of most value the inclination of the beds of an artesian formation should not be great, a steeper inclination than is necessary to furnish a good covering being disadvantageous. A small inclination furnishes a wide percolation area at the outcrop, and at the same time the area is large over which the stratum can be reached by wells of practicable depth. Thick strata give proportionately large percolation area and great carrying capacity.

It often occurs that water is obtainable from two or more parallel strata. In such cases the lower usually furnishes the higher head, the outcrop being more remote and at a higher elevation.

98. **Capacity.**—Except in the case of very limited areas, the capacity of an artesian source as a whole is a question of little importance where it is to be used only for water-supply purposes in towns widely separated; for the total amount of water capable of being drawn from porous rock strata, often hundreds of feet thick and having an outcrop of hundreds or thousands of square miles, is ordinarily very great as compared to any possible demands for such purposes. The

problem is rather one concerning the number and arrangement of wells to furnish a given quantity, a question which is discussed in Chapter XIV.

In localities where wells are extensively used for irrigation purposes the total capacity of the source becomes a serious question, as is already the case in some portions of the West.

The total possible yield of an artesian source may be limited either by the rainfall and percolation on the outcrop or by the carrying capacity of the strata. With the slight slopes and broad outcrops commonly occurring, the carrying capacity will probably determine the maximum yield, while with steep slopes and small outcrops the water may be drawn out faster than it flows in.

The velocity of flow through the pores of rock formations is necessarily extremely slow on account of the great resistance offered. In many cases, no doubt, fissures and other openings of a large size relative to that of the pores of the stone add very greatly to the carrying capacity of a rock stratum. In a sandstone formation such are not so likely to be of great influence as in the case of limestone, where indeed they may be the controlling factors in determining the capacity.

99. Some rough calculations relating to the flow through thick porous strata may be of value in suggesting the possible limitations in the carrying capacity of sandstones and of strata of loose material.

The Potsdam sandstone of northern Illinois and southern Wisconsin has a thickness estimated at from 700 to 1200 feet, and a width of outcrop in Wisconsin of 40 to 60 miles, or about 250 times its thickness. The percolation with a rainfall of about 35 inches may at the very lowest be taken at 5 inches per year. Assuming a porosity of 25 per cent, this would fill the rock to a depth of 20 inches vertically, or a horizontal length of the stratum of $20 \times 250 = 5000$ inches, or 417 feet. Now the rate of flow depends upon the available head or hydraulic slope, which in the region here considered would not, even with the use of deep-well pumps, exceed 3 or 4 feet per mile. Assuming the resistance to flow to be equal to that in a fine sand of 0.1 mm. size (it is probably much greater),* the velocity would be, according to the tables on p. 98, equal to $82 \times .34 \times \frac{4}{5280}$ ft. per day = 8 feet per year, a very much less rate than that of the percolation. If the material were a coarse sand of a size of 0.35 mm., the carrying capacity would, under the assumed conditions, be about equal to the rate of percolation.

In some basins the possible slopes are somewhat greater than in the example given, but it will seldom be found that artesian areas are likely to be limited in supplying capacity by the lack of percolation.

The actual quantity flowing through a given cross-section may be roughly estimated in the same manner as for any other ground-water stream. In the

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* Some experiments by King on the flow of water through Madison sandstone indicate a resistance to flow equal to that in a sand of an effective size of about .03 to .05 mm. See Nineteenth Annual Report U. S. Geolog. Survey, p. 140.

case above discussed the flow would be from Table No. 22, p. 102, equal to $.51 \times \frac{4}{52.8} = 0.039$ gallons per day per square foot of cross-section, or for a section 1 foot in width and 800 feet deep it would be about 32 gallons per day. The quantity flowing per mile in width would therefore be about 170,000 gallons per day, and this would represent the maximum delivering capacity of a line of collecting-works of indefinite length. For isolated groups of wells the flow is lateral as well as in the direction of the general slope, and the capacity is relatively very much larger. In view, however, of the great distances over which large draughts affect the pressure in this area it is doubtful if the flow is much greater than the above figure.

100. Predictions Concerning Artesian Wells.—The question of the existence of water-bearing strata at any point, their character and depth, and the location of outcrops, is a geological one; and where full information on this point has not been gained by the sinking of wells or by borings, a geologist familiar with the region in question should be consulted. Much money has often been wasted in fruitless attempts to obtain water in areas and at depths where none could be expected, and frequently such work has been carried on contrary to the advice of experts.

The pressure which will exist in a well is a question of hydraulics. With a knowledge of the pressure in neighboring wells, and of the surface topography and elevation of outcrops, a fairly close estimate of pressure may be made.

The questions of percolation, freedom of flow, and capacity cannot of course be very closely determined, but a careful consideration of the principles of hydraulics, which govern here as elsewhere, will at least enable one to avoid the absurd estimates which are sometimes made and which lead to disastrous results.

In the construction of wells it is important to preserve samples of the borings, as it is largely through these that a knowledge of the geology of the region is acquired. Chemical analyses of the water are also a valuable aid in identifying strata.

101. Important Artesian Areas in the United States.—*The Atlantic and Gulf Coast Region.*—One of the most extensive and important artesian-well areas in the United States is that which borders the Atlantic Ocean and Gulf of Mexico, extending from Long Island on the north to Texas on the south. Along the Atlantic coast it will average perhaps 100 miles in width, but along the Gulf it broadens out, extending up the Mississippi valley as far as the Ohio River, and in Texas it has a width of 200 to 300 miles.

The several strata in which water is found belong to the Cretaceous formation. They outcrop along the foothills of the higher country to the west and north, dip towards the ocean at a considerable slope (20 to 40 feet per mile along the Atlantic coast), and presumably have their lower outcrop in the deeper ocean many miles from shore. Their connection with the ocean is indicated by the occurrence of ocean springs as already noted (Art. 92), and by the effect of the tides upon the pressure in certain wells, it being stated that at Pensacola the water-level in wells located several feet above sea-level varies from 6 to 10 feet as a result of tidal influence.*

Fig. 22, p. 107, is a section showing arrangement of strata typical of this region. The conditions here are evidently very favorable for artesian wells, and the various water-bearing strata furnish an exceedingly valuable source of water-supply for the cities of this region that otherwise could procure pure water only at much cost. Among the cities which get their supply from this source are Savannah, Charleston, Jacksonville, St. Augustine, Key West, Memphis, Galveston, and Fort Worth, besides a large proportion of the other towns in Florida, Mississippi, and Texas. Many wells have also been sunk in Long Island, New Jersey, Delaware, Maryland, and in the city of Philadelphia.

102. *Artesian Areas in the West.*—Another very important artesian area, noted for its high pressures and the great number of wells, is that of the James River valley in eastern North and South Dakota.† This area is supplied chiefly from the Dakota sandstone, a formation belonging to the Cretaceous. It outcrops along the slopes of the Black Hills and the Rocky Mountains at altitudes of 3000 or more feet above sea-level, furnishing a percolation area estimated at about 14,000 square miles. From there it descends beneath the more recent formations of the plains, again ascends slightly, and outcrops at its southeastern edge along the Missouri and Sioux rivers at an elevation of about 1000 feet. Farther to the north the edge of the stratum is covered, and the waters are there largely prevented from escaping. The result is that wells sunk to this stratum at points where the surface is low, as in the James and Missouri river valleys, will have high pressures (these are in some cases as high as 150 pounds per square inch static pressure), but towards the south and east the pressures rapidly fall off. Fig. 23 is a section through South Dakota showing the arrangement of the strata.‡ The static head at the various wells is indicated by the heavy line.

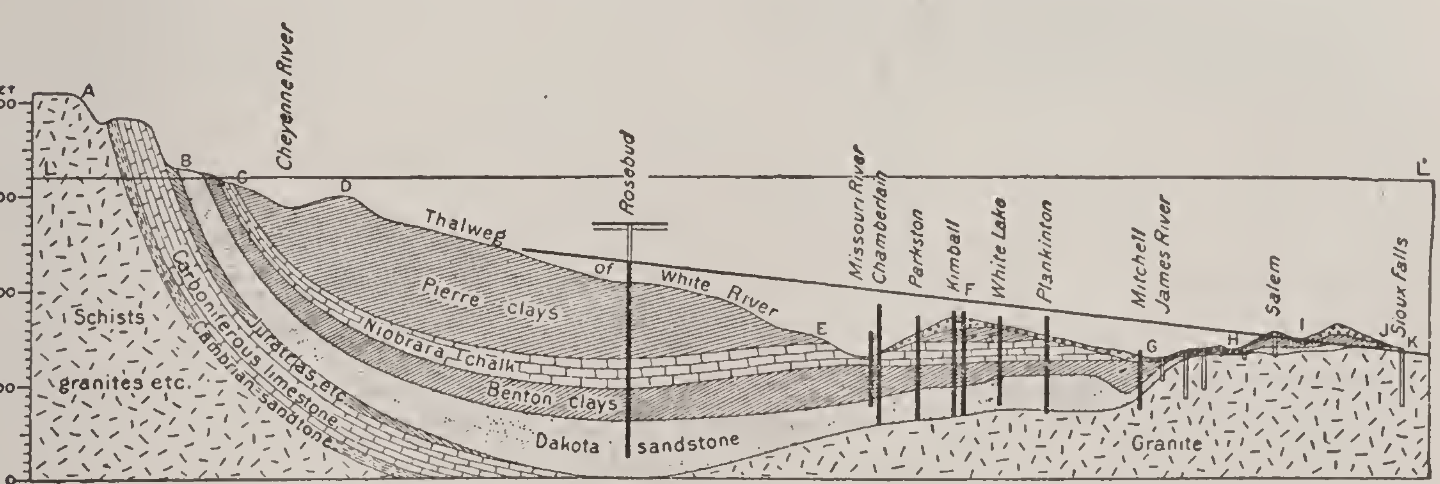

FIG. 23.—SECTION THROUGH THE DAKOTA ARTESIAN AREA.
(From W. S. Paper No. 67.)

The report of Mr. Darton gives a very instructive map showing the reduction of pressure towards the eastern outcrop. The hydraulic slope amount-

* Trans. Am. Soc. C. E., 1893, xxx. p. 695.

† See report by N. H. Darton in U. S. Geolog. Survey, 1895–96, p. 603.

‡ From W. S. Paper No. 67, U. S. G. S., 1902.

in general to from 4 to 6 feet per mile, which agrees fairly well with the slope from outcrop to outcrop. The estimated number of wells in this basin in 1896 was 400, of which 350 were flowing wells. The total estimated yield was 232 cubic feet per second. The water is used for irrigation, water-supply, and power purposes.

This same Dakota sandstone is the source of supplies over small areas in Nebraska, Kansas, and Colorado, while still other strata in more recent formations furnish artesian water in many places of the Western plains. Among the Rocky Mountains and in California are also many basins where artesian conditions are found. One such is shown in Fig. 18, p. 93.

In 1890 there were altogether about 9000 artesian wells in the western part of the United States, located for the most part on farms for water-supply and irrigation purposes. Of this number over 3000 were located in California.*

103. *The Artesian Area in the Upper Mississippi Valley.*—A large area, mentioned on p. 109, in which artesian water is extensively used for town supplies is that in northern Illinois, eastern Iowa, and southern Wisconsin. Here the water is furnished by several strata, chief of which are the St. Peter and the Potsdam sandstones. Fig. 24 is an approximate north and south section through central Wisconsin showing the general arrangement of the water-bearing strata.† The collecting area of the Potsdam strata is estimated at 14,000 square miles, while that of the St. Peter is only 2000 to 3000. These strata dip deeply below the surface in southern Illinois, and are there beyond reach. They also dip both eastward and westward from the section

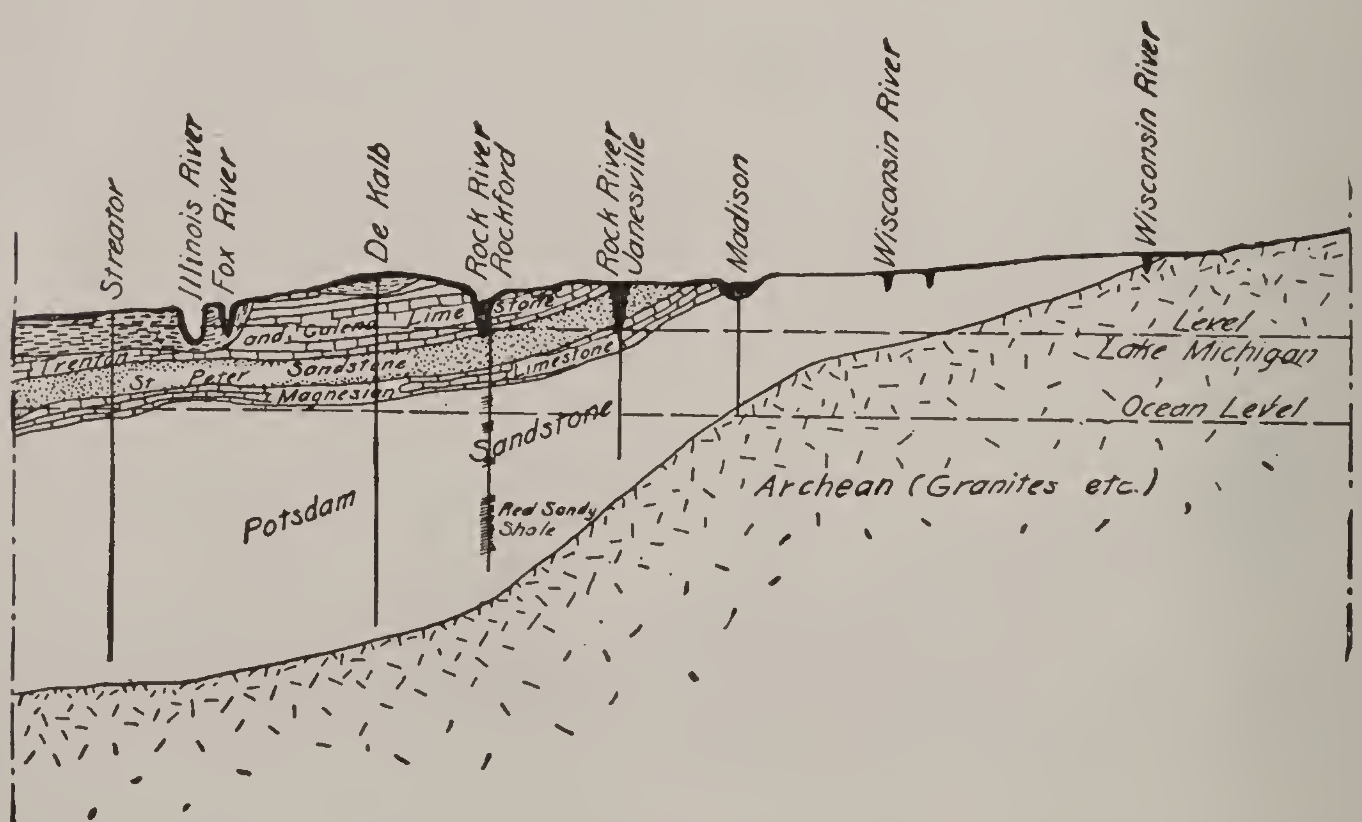

FIG. 24.—SECTION THROUGH NORTHERN ILLINOIS AND SOUTHERN WISCONSIN.

shown. The slope of the surface of the ground is quite small, and the available head and the quantity obtainable are therefore rather limited. At

* Eleventh Census. Report on Irrigation by F. H. Newell.

† From a paper by D. W. Mead, *Jour. Assn. Eng. Soc.*, 1894, XIII. p. 396.

Chicago the available head originally was about 100 feet, but the draught has been so great that now the wells seldom flow, and the exhaustion is felt for several miles distant.

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B. QUALITY OF WATER-SUPPLIES.

CHAPTER VIII.

EXAMINATION OF WATER-SUPPLIES.

104. **Scope and Extent of Examination.**—The most important examination of water is that which is made to determine its potableness and wholesomeness. Attention should of course be given to a water-supply from other points of view, but the relation to disease-dissemination is of paramount interest.

In determining by means of a sanitary analysis whether any water is suitable as a public supply, various methods have been instituted as knowledge regarding these problems has been broadened. None of these methods, however, that have yet been introduced are wholly satisfactory, and the subject of sanitary water-analysis is far from being reduced to terms of mathematical accuracy. One prominent reason for the unsatisfactory results that are frequently found in analytical work is that many of the determinations apply only indirectly to the presence of specific disease organisms. This indirect relation therefore raises a question that must be interpreted anew in each individual instance. It calls for a discriminating judgment on the part of the analyst. In fact the data which he collects do not answer definitely and decisively the question as to the purity or pollution of a supply, but, as Mason has well expressed it, it assists his judgment in interpreting these data.

105. **Necessity of Full Data in Interpreting Conditions.**—Concerning this phase of the water-analyst's work there is a great deal of misconception. Many people think that the withholding of all data as to the origin and local conditions surrounding the supply in question will enable the analyst to arrive at an unbiased opinion. Some in fact think that such a procedure is necessary to test his expert skill. They look upon a water-analysis as something similar to an assayer's test for gold, something which can be positively determined. But when there is

taken into consideration the wide range that may exist in waters from various sources, and how the analytical results obtained in such examinations will of necessity be interpreted differently in samples coming from different regions, it is manifestly impossible for the analyst to arrive at any satisfactory judgment unless he is more or less familiar with all of the data that are intimately related to the case. The character of the stratum from which the supply is derived, the possibility of pollution, and the nature of the same, the kind of water, the conditions under which the sample has been secured and kept are all questions concerning which he should have full and explicit knowledge. Of course it may be possible in extreme cases, as with badly polluted or exceptionally pure waters, to determine their nature with certainty from a mere examination, but in the great majority of cases where the conditions are less pronounced, his judgment may be at fault because he is kept in ignorance of local conditions. A sample which would be regarded as satisfactory for a surface-water might be condemned from a biological standpoint if from a well, while the chemist would frequently reverse the conditions in testing certain artesian and surface waters. The case has been well compared to the physician who is able to diagnose any malady by an examination of the urine alone, or in some cases by an inspection of even a lock of hair. Such a diagnosis would have but little worth against the judgment of one who had studied the case at first hand. This should also be the position of the analyst. He should be able to inspect the local surroundings, as these frequently give evidence that may be of more value than the analytical data secured. If not able to secure the data at first hand, as full information as possible as to these facts should be furnished him.

106. Collection of Samples.—The results obtained in the analysis of waters are frequently misinterpreted because of errors in sampling. It is necessary in this, as in all analytical work where reliance is to be placed on the results obtained from an examination of a comparatively small amount of material, to have a representative sample; not only as to its composition, but as to subsequent changes that may take place in same. In sanitary water-analysis the conditions are not as they would be in a mineral analysis of a rock or an ore. The substances for which search is to be made are for the most part living things, or the products of vital activity. Generally a water is not in stable equilibrium, but is constantly undergoing changes which, if they occur before the analysis is made, may markedly affect the interpretation that might be given to the results.

In taking samples for chemical or bacteriological purposes, certain requirements must be observed that are somewhat different for the two purposes mentioned.

107. Samples for Chemical Analysis.—For a chemical sample take a glass-stoppered bottle of about one-half gallon capacity.* Rinse out the same thoroughly so that it is free from dust or dirt. If it is impossible to secure a bottle having a glass stopper, one fitted with a new cork may be used, but it should be very thoroughly rinsed before using. Fill the bottle nearly full with water, leaving a small bubble of air for expansion. To protect the mouth of bottle in transit, tie over same a piece of cloth. The cork should not be sealed in by using sealing-wax or paraffin.

Great care should be observed to secure a representative sample. If from a well, the water should be pumped out so as to remove that which has been standing in the pipe. In a dug or open well it may even be preferable to pump out enough so as to remove at least a part of the quantity originally present in the well and thus allow a fresh supply of ground-water to flow in; although in cases of surface pollution the withdrawal of the water in the well diminishes the amount of polluted matter. If sample is taken from a surface supply, the bottle should be plunged beneath the surface before removing the stopper, so as to prevent the entrance of dust-particles and other floating impurities. Care should also be taken not to draw the sample from too near the bottom.

108. Samples for Bacteriological Analysis.—For a bacteriological analysis still greater care must be taken in order to secure a representative sample; and also to lessen the changes that quickly occur in the germ-life in a water. As ordinary glass receptacles always contain more or less bacteria adhering to the inner wall, it is necessary first to destroy these even after the bottle has been thoroughly cleaned. This is done by baking in a dry sterilizer (hot oven) at a temperature of about 280–300° F. for one hour, or steaming in a steamer at 212° F. When facilities are not at hand for sterilization by heat, the adherent germ-life can be destroyed by rinsing out the flask with chemical disinfectants (corrosive sublimate, 0.1 per cent solution, carbolic acid, 5 per cent, or dilute mineral acids). It is then necessary of course to remove all trace of the disinfectant, which can be easily done by rinsing out the bottle at least four times with the water to be sampled.

Every endeavor should be taken to exclude the influence of extraneous factors, and in a bacterial test these are much more numerous and exert a more profound effect than in chemical work. In a surface-

* For a full mineral analysis five gallons is generally taken.

water the influence of land contamination should always be considered. Samples taken from a stream after a rain where any turbidity has been produced will not be representative of normal conditions. In taking samples from wells, the pump and pipe should be thoroughly washed out, as bacterial growth generally occurs in water standing in the same. Preferably bacterial cultures should be made immediately after the sample of water is secured, as a marked change occurs in the germ content of water stored for a period, especially if temperature is rather high (135). This multiplication is less in tightly stoppered glass bottles than in those closed with cotton, and is less in full bottles than those partially filled. To lessen these changes as much as possible where it becomes necessary to transport samples any considerable distance, they should be packed in ice, in which condition growth will be greatly retarded. Not infrequently in samples so treated there is a diminution to be noted.

The bottles used to collect bacteriological samples do not materially differ from those employed in securing chemical samples, except that generally a smaller quantity of water will suffice, unless it is desired to filter a large amount through a germ-proof filter (Pasteur) and in this way concentrate the bacteria. In taking samples from deep waters, special kinds of apparatus have been devised that permit of the securing of a sample uncontaminated from that of any other depth.

109. Sanitary Analysis of Water.—Definite knowledge of a certain character is often desired as to the quality of water for different purposes (manufacturing, etc.), but these data do not fall primarily within the province of the sanitary engineer. A sanitary analysis is the study of a water with the view of determining whether it now contains, has contained, or is likely to contain, anything which is detrimental to public health. Not only must potable water be free from any taint of suspicion that would indicate dangerous pollution, but, at the same time, water should not be objectionable in taste and appearance. Moreover, water-supplies must furnish water that is suitable for laundry and general domestic use, although this is not strictly a sanitary consideration; but inasmuch as a supply must cover all purposes for which water is commonly used, this must also be considered.

It is a notorious fact that the majority of people mainly judge of the quality of a water by its taste and appearance. If it is clear and sparkling and is fresh in taste, they will use it without question, caring little as to the possibility of pollution with disease bacteria. Once let these physical conditions be altered and suspicion at once attaches itself to the supply. This deep-grounded opinion arises for the most part from a rational conviction that wholesome water, especially that derived from the ground, is clear and sparkling and ought to remain so. If for any reason a change occurs, it signifies a variation in conditions—a state to which ground-waters of first quality ought not to be subject. While this rule applies universally to ground-waters in wells, it is not so pertinent to surface-supplies or waters in large distributing systems as in large cities.

In determining the sanitary condition of a supply, a single analysis is of but little value, especially if this is made by one unfamiliar with local conditions. To be able intelligently to interpret conditions with any marked degree of accuracy, analyses should be conducted at frequent intervals, in order to determine the stability of the chemical and biological composition. A water-supply subject to sudden and considerable fluctuations in these respects is generally one that should be regarded as suspicious, at least until the cause of such variation is satisfactorily explained.

110. Detection of Pollution by Addition of Chemicals.—Not infrequently a simple qualitative test that can be readily applied by the non-expert is of considerable service in detecting a possible polluted condition in a water-supply. This is generally done by the addition of some chemical substance to the source from which pollution is possible and then determining whether the same reappears in the water-supply. For this purpose a number of different chemicals are used. Those most readily recognized are substances having a marked taste or appearance.

Nördlinger recommends for this purpose saprol, which tastes like naphtha and is so penetrating that its odor can be readily recognized in proportions of 1 : 1,000,000, and by taste in solutions of 1 : 2,000,000. Some of the anilin dyes, as fluorescein, often color the water in such dilute solutions that a change in color will be recognized even after filtering through a deep stratum of soil.

Trillat* has recently experimented with a large number of these dyes and finds that fluorescein dissolved in alcohol and diluted with 5 per cent ammonia solution can be detected by means of a fluoroscope in proportions of 1 : 2,000,000,000. The fluoroscope used is a tube of white glass three or four feet long and one-half inch in diameter, closed at one end with a rubber cork. In such an apparatus natural waters have a somber blue color which changes to a clear green if fluorescein is present. This dye possesses the evident advantage of

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* *Ann. Past.*, 1899, XIII. p. 444.

not being precipitated by the soil ingredients, a reaction that readily occurs with most anilin dyes brought in contact with calcareous soils.

Where chemical methods can be employed the use of readily soluble salts as, NaCl (common salt), permits of ready recognition. Salts of lithium are sometimes employed. These admit of detection in inappreciable quantities if the water is examined by the aid of a spectroscope. It does not necessarily follow because these soluble salts reappear in a water that organisms and dangerous pollution would likewise find its way through the soil for an equal distance, for the filtering power of the soil if free from actual channels would be such as to remove suspended particles, even no larger than bacteria, while salts in solution would pass through soil by diffusion; but nevertheless these methods are of service in showing whether the possibility of danger exists.

111. **Various Analytical Methods.**—In examining a water as to its suitability for public use, four different kinds of tests can and should be applied. These are as follows:

Physical examination.
Chemical examination.
Bacterial examination.
Microscopical examination.

The respective value of these independent analytical methods differs much in various instances, yet in the examination of most waters all of them have a distinct value. The judgment arrived at as a result of these tests should be interpreted in the light of an actual inspection of surroundings, if possible. More and more, the experienced sanitarian is coming to regard an ocular inspection as the final court of appeal to which all analytical conclusions should be referred.

112. **Value of Different Methods.**—Naturally the physical tests as to the character of a water have been noted for the longest period. By the aid of the senses any one can detect in water an abnormal appearance, odor, or taste, if it is at all pronounced. If such obtains, this is generally sufficient to discredit the reputation of the supply.

With the determination of the relation that exists between various water-borne diseases and human fecal matter, the chemical methods of examination were gradually devised. These have been slowly perfected, so that at the present time they permit of the recognition of a larger number of factors that affect the value of a water than is to be determined in any other single way. But even the chemical method of examination is largely an indirect method of analysis. The presence of nitrites or chlorine in considerable quantities in a water is not a source of disease in and of itself, but under natural conditions a water revealing the presence of these substances in large quantities is generally one that has been polluted with organic matter, possibly of fecal origin. So generally has this indirect relationship been determined that the detection of considerable quantities of such chemical compounds as these is regarded as sufficient evidence of sewage pollution. It must of course be kept in mind that sewage from healthy sources may be diluted to such an extent as to be comparatively harmless; but the fact remains that such sewage may suddenly become detrimental by reason of disease bacteria gaining access to the same, a condition which is of course readily possible if even dilute sewage was to be tolerated in any supply used for potable purposes.

Again, a distinct value of the chemical method of analysis is that it tells something of the previous history of the water. If nitrates are present under certain conditions, it shows that organic matter has had access to the water and has undergone the decomposition changes incident to such material. This may therefore represent a condition of past pollution.

Unfortunately, on the other hand it is not always possible to decide by the chemical method as to the origin of such organic decomposition products; whether they are associated with human or animal sources or perhaps attributable to vegetable decay. Often the chemical analysis is extended to include incrusting constituents, a determination of the alkalinity, the carbon dioxid and the iron dissolved in the water. Ordinarily, though, these have no special sanitary significance, and are made to determine the character of the water from other points of view. Under certain conditions, as in filtration work where coagulants are used, the determination of alkalinity is of sanitary importance as a basis for the addition of the coagulating agent.

Inasmuch as the direct causal agent concerned in the production of disease by the use of impure water usually belongs to the bacteria, it would be reasonable to suppose that a bacteriological examination would be a direct method of determining the quality of water, and it would therefore possess a value that does not obtain in the use of any of the indirect methods. This hope, however, has been only imperfectly realized as yet; for, in the main, these methods do not often consist in a direct search for the specific disease organism, but in apprehending the conditions that might permit of the recognition of sewage pollution or the possibility of infection. Therefore these methods of examination as now used are also to be considered as indirect. This course is rendered necessary by reason of the fact that disease germs rarely exist in any water in sufficient numbers for any considerable length of time so that they can be readily recognized; whereas if they find their way into water through the introduction of fecal discharges, this evidence will be apparent for a longer period of time. The bacteriological tests are also a more sensitive measure of the changes in the condition of the water. By carefully controlled quantitative estimates it is thus possible to detect variations in composition that would remain unobserved if sole reliance were placed on other analytical methods. Again, the bacteriological method offers by far the most accurate way to determine the efficiency of filter practice.

The microscopic examination of water does not so much concern itself with a determination of whether sewage or the possibility of such pollution is actually present or not, as it does with the character of the minor organisms of a vegetable and animal nature. Some of these not infrequently cause bad odors and tastes in waters, but for the most part they do not have any other sanitary significance (183). Few are more or less distinctive of polluted waters, and hence their recognition is of value in this connection.

PHYSICAL EXAMINATION OF WATER.

To the sanitary engineer as well as the non-technical individual, the physical tests applied to any water are of considerable importance, as frequently an acute sense will be able to determine by these means a water that is unsuitable for use.

113. **Color.** — A water to be perfectly satisfactory as to its physical requirements should be colorless, free from any turbidity, undesirable odor or taste, and of sufficiently low temperature to be refreshing. Ground-waters are for the most part free from color, but some surface-waters, particularly those of swampy origin, are often highly colored by the soluble organic matter that is dissolved in them. The peaty waters of north England and a large number of the streams draining the forest areas of the United States and Canada are typical of this class. Water colored from this cause generally exerts no noticeable effect on the health of persons using it.

In determining color, comparison with some arbitrary standard is usually made.* For this purpose several standards have been proposed. One of the earliest was to use the colors produced by the Nessler standards employed in the estimation of ammonia. With yellow

* The following papers give a full discussion concerning the subject of color determination: *Jour. Frank. Inst.*, 1894, p. 402; *Jour. Am. Chem. Soc.*, 1896, XVIII. pp. 68, 264, and 484; *Jour. N. E. Water-works Assn.*, 1898, XIII. p. 94.

waters, this standard was fairly satisfactory. A more recent and more satisfactory standard is made by comparing waters with dilute solutions of salts of platinum and cobalt. By varying the ratio of cobalt to platinum it is possible to simulate closely the hue of the natural water. The color is recorded in terms of the platinum, one part of the metal in 1,000,000 parts of water equalling one unit.

114. **Turbidity.** — Waters drawn from surface sources, particularly from running streams, are often more or less turbid from the presence of suspended matter that finds its way into the drainage-streams by reason of the run-off. Depending upon the geological nature of the watershed, this turbidity may be sandy or clayey. If sandy, the actual amount of suspended matter may be quite large without making the water unsightly. On the other hand, if clay particles abound, a much smaller amount may render the water densely turbid. Sometimes these particles are so minute and of such a gelatinous nature that even after a long period of quiescence the water remains more or less cloudy.

While turbidity in a water is generally due to the presence of inorganic matter, yet vegetable growths at times may render a water turbid. Such troubles are generally seasonal, due to the increase of these vegetable forms during the warmer months. Algæ, and particularly the diatoms, are most frequently concerned in such changes. In iron-containing waters, a turbid condition may be induced by the presence of the iron-bacterium, *Crenothrix* (196), or by simple chemical oxidation of the ferrous salts to ferric oxid. Lake waters, such as those of our Great Lakes, are relatively free from turbidity, except as disturbed by storms. Rivers draining forest areas are generally quite clear, although they may be colored from dissolved organic matter.

Several tests for turbidity are in use. The silica standard is prepared from ground diatomaceous earth that will pass a 200-mesh sieve. Where 100 parts of silica per million of water are used, a platinum wire one mm. in diameter that is just visible in open air, 100 mm. below surface, gives a turbidity of 100.

Another method is the candle turbidimeter * which consists of a graduated glass tube with a flat bottom enclosed in a metal case. This is held over an English standard candle and so arranged that one may look vertically down through the tube, and see the image of the candle. The water is poured into the tube until the image of the candle just disappears from view. The tube is either graduated into parts per million of silica, or into numbers which correspond to silica standards.

* Made by Baker & Fox, 83 Schermerhorn St., Brooklyn, N. Y.

Another method used is to employ a white disk. Whipple employs one 8 inches in diameter that is painted black and white alternately.

115. Odor and Taste.—Normal waters should be relatively free from any pronounced odors or tastes. The naturally pleasant taste noted in good water is due in the main to the oxygen and CO_2 dissolved therein. Some waters, particularly spring-waters, may at times give forth an earthy odor due to the volatile substances absorbed from the upper soil layers. In other cases they may be so thoroughly impregnated with various mineral ingredients as to possess a distinct taste, as is the case with salt, iron, or sulfur springs.

A considerable number of the lower plant and animal forms are able to affect the taste and odor of waters, especially open surface-waters. "Fishy," "grassy," and oily conditions are those most frequently noted. These odors are not attributable so much to the decay of organic matter as they are to the growth of certain odor-producing algæ (183). To recognize more thoroughly the odor of water, it should be warmed to about 65° F., the bottle remaining tightly corked until the test is applied.

116. Temperature.—From the standpoint of the consumer, the temperature of a water is considered of first importance. Naturally this condition is determined by the source of the supply. Surface-waters follow in general the atmospheric variations, but, owing to the high specific heat of water, they never show the range to be noted in the atmosphere. In winter, the temperature of water-supplies may almost reach the freezing-point, while in summer it frequently exceeds 80° F., being as much too warm at this season as it is too cold for use in winter. In quite large and relatively deep bodies of water the temperature changes are not so marked. In deep lakes protected from strong wind action, the temperature of the lower stratum changes very slowly owing to the low conductivity of water; but in shallow waters the temperature coincides more closely with that of the mean atmospheric temperature.

Ground-waters have a much more uniform and lower mean temperature than waters exposed to the air. At a depth of 40–60 feet, varying in soils of different composition, the zone of constant temperature is reached and waters from this level remain quite uniform throughout the year, ranging from 48–52° F. If the temperature of the supply is subject to much fluctuation, and especially if it is above these limits, it indicates a supply of shallow origin. Very deep wells, as artesian supplies, frequently have a considerably higher temperature, due to the effect of the internal heat of the earth.

Often a city supply that has a suitable initial temperature has its temperature raised to a point where it tastes insipid because of the shallow depth at which the mains are laid, or the long distance from source to place of consumption; but usually the temperature as delivered to consumers depends mainly upon that of the source.*

Generally the temperature of ordinary supplies derived from lakes can be measured quite closely by lowering a thermometer in a vessel of considerable capacity. This can be withdrawn before it materially changes. For accurate determination Warren and Whipple † have devised an instrument known as the thermophone, which is practically an electrical thermometer of the resistance type. This instrument permits of the registration of the temperature at any depth.

117. **Chemical Reaction.**—The chemical reaction of a water is usually slightly alkaline, due to the presence of calcium and magnesium carbonates; in peaty waters the reaction is acid, caused by the vegetable acids here found (humic, geic, and ulmic).

CHEMICAL EXAMINATION OF WATER.

118. **Purpose of Chemical Tests.**—The chemical methods of water analysis do not seek to ferret out the presence of any specific disease-producing organism. A water may possibly be regarded as bad from a chemical point of view, and yet be wholly free from disease organisms, but under ordinary conditions the disease-germs that are disseminated by polluted water-supplies generally find their way into the same through the medium of sewage. Under these conditions, then, the chemist does not test directly for any specific microbe, but for sewage pollution, present or past. This he does on the basis that a water-supply intended for human use should under no condition contain any evidence of fecal pollution. His aim, as Drown states, is to discover the origin and history of the nitrogen compounds in the water.

The tests that the chemical analyst employs in passing judgment on the sanitary quality of a water are for the most part, however, methods that indirectly permit him to recognize the presence of living organisms in the water. The determination of organic matter by the loss in weight of total solids before and after ignition, the presence of nitrites and nitrates, the amount of oxygen consumed, the free and albuminoid ammonia present, are all of them directly related to organic matter of vegetable or animal origin.

119. **Expression of Chemical Data.**—Much confusion exists in the interpretation of chemical data because no single standard is recognized

* Exam. of Water, 1890, p. 675, Mass. Bd. Health.

† Microscopy of Drinking-water, p. 55.

the world over in presenting the results of analytical work. The earlier method of giving number of grains per gallon * has been for the most part supplanted by the method of expressing the data either as parts per 100,000 or parts per million in weight, the evident advantage of the latter method being that no computation is necessary where weight is expressed in milligrams, as this gives parts per 1,000,000 when referred to a liter.†

120. Interpretation of Chemical Data.—It is beyond the purpose of this book to take up methods of analysis, but the sanitary engineer should be able at least to interpret in a general way the results of such analyses.

Desirable as it would be to have definite standards of water analysis that would apply to all waters, such are, nevertheless, impossible. The changing conditions under which various potable supplies occur make it altogether out of the question to have a standard that would be of general application. In the present state of the science there is even a lack of uniformity in interpreting results, some analysts placing more emphasis on one factor than on another.

While a general standard of purity is not possible, many have advocated the adoption of local standards that embrace a definite geological formation in a restricted region. This standard of course could not apply to all classes of waters from even a single region, but would have to be limited to waters of the same origin, as wells, springs, or streams.

121. Total Solids and Character of Same.—Ordinarily a water is examined in an unfiltered condition, but in certain cases it is necessary to differentiate between substances in solution and those held in suspension. The total solids of a good water, including both suspended and soluble matter, vary considerably, depending upon the geological formation. Well- and spring-waters are naturally much higher in soluble solids than surface supplies.

The solids in a water are made up of mineral matter such as carbonates, chlorides, sulfates, etc., together with the organic matter of vegetable and animal origin. The inorganic ingredients determine the hardness of the water, a characteristic that is generally determined, but which is of more economic than sanitary importance. The hardness

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* *Conversion Table.*—To convert grains per Imperial gallon (parts per 70,000) into parts per million, divide by 7 and multiply by 100.

To convert parts per million into grains per gallon, multiply by 7 and divide by 100.

† This standard is recommended by the Committee on Methods of Water Examination appointed by the American Association for the Advancement of Science.

in a water is temporary when it is caused by carbonates which are precipitated upon heating, while the sulfates and chlorides produce a permanently hard water. A water of moderate hardness is generally preferred by most people to soft water for drinking purposes. Not infrequently, waters are so hard as to be unsuitable for industrial as well as domestic purposes. In such cases they can be softened by the aid of chemical treatment (Chapter XXIII).

122. Loss on Ignition.—If the evaporated residue obtained in determining the total amount of solid matter is gradually heated to redness, the organic matter is driven off by degrees. If the ash is white, it denotes the presence of mineral solids, although the presence of iron will tend to discolor the ash. If much organic matter is present, it blackens and the peculiar smell inherent to vegetable or animal substances may often be detected.

Peaty waters will naturally contain a considerable amount of organic matter, the presence of which may not be incompatible with good water.

The relation between the weight of the total solids obtained by drying at 212° F. and the ash after ignition marks the amount of organic matter, but some mineral salts break up on being heated and so diminish the value of this determination.

123. Chlorine.—All surface- and ground-waters contain chlorine in variable proportions, the majority of the chlorides existing in the form of sodium chloride (common salt). In certain regions which are underlain with salt-bearing strata, as central New York and Michigan, the chlorine content of the ground-waters is of course high. Proximity to the ocean also increases appreciably the chlorine of unpolluted waters, both those of deep and surface character. This has been strikingly shown in Massachusetts and Connecticut, where a survey of these States has been made with reference to this point. The lines representing approximately equal amounts of chlorine, called isochlors, run, in general, parallel with the coast. They range from 24 parts per million on ocean-engirdled Cape Cod to .6 part per million in the northwest portion of Massachusetts. These data are very valuable in determining a local standard as to the normal condition of unpolluted waters of different regions.

Chlorine is also a constant accompaniment of sewage and house-wastes, urine containing from 0.75 to 1 per cent of the same. The readiness with which this element percolates into the soil, and its stability, are such that it serves as a ready means of determining whether the ground-water is polluted with household or animal wastes.

The presence of no more than normal amounts in a water is therefore good evidence that it is unpolluted, but the converse of this does not necessarily mean pollution. Here is where the necessity of additional data is evident as to the normal chlorine content of waters in the region under investigation. Excluding chlorine due to salt deposits and that derived from the sea, a high content generally means pollution with sewage or household wastes. Chlorine in itself, however, may be misleading, as it tells nothing of the time of pollution. Being soluble it percolates slowly into the ground, and it by no means follows that disease bacteria or any other harmful substance is capable of following it. Pollution may have occurred at some previous date and the organic matter undergone complete oxidation, and yet the chlorine remains to tell of past pollution.

In this way the soil of inhabited areas becomes gradually impregnated, so that the ground-water of such regions is generally much higher in chlorine than that from less thickly populated localities.

The observations of the Massachusetts Board of Health indicate that 100 persons to the square mile will increase, on the average, the normal chlorine of a region about 0.5 part per million. Thresh's* estimate for England is about 0.43 part per million for the same increase in population per square mile.

124. Organic Matter.—Inasmuch as the really dangerous substances in a water from a sanitary point of view are organic in nature, the determination of this factor is of prime importance. The organic material in water may be of either animal or vegetable origin. Purely vegetable matter, as in peaty waters, may frequently be present in excess and still such waters be perfectly wholesome. That which is associated with human wastes is of course the most dangerous, but it is not easy to determine by chemical analysis the exact origin of the organic matter as to whether it is derived from animal or human sources.

125. Free and Albuminoid Ammonia.—Inasmuch as the nitrogen content of organic matter throws much light on the character of the same as to whether it is of animal or vegetable origin, a determination of this element in the form of free and albuminoid ammonia is of great service in sanitary chemical analysis. In the decomposition of organic matter, more or less complex nitrogenous by-products are produced that are classed as albuminoid in character. In the more ultimate stages of this disintegration, the nitrogen appears in the form of ammonia which may unite with acids to form salts. These products are finally converted by other bacteria in nitrites and nitrates.

* Pearmain and Moor, Chem. and Biol. Analysis of Water, p. 51.

Albuminoid and free ammonia therefore represent nitrogen in the earlier transition stages, and inasmuch as these products invariably accompany fecal matter, their presence in water is of sanitary significance.

Waters may, however, contain considerable quantities of free ammonia under normal conditions and still be entirely wholesome, as in peaty moorland waters, in rain-water, and even in artesian wells.

In the case of many ground-waters the ammonia is probably due to the reduction of nitrites and nitrates by reducing substances present in the soil. Albuminoid ammonia should not be present in such waters. If it is, it is indicative of surface pollution or imperfect filtration. The ratio between the free and albuminoid ammonia is of importance in judging of the character of the organic matter. Generally a high ratio between the albuminoid and free ammonia in connection with low chlorides and nitrates characterizes vegetable pollution; increased amounts of free ammonia with an excess of the chlorides, animal matter.

Something as to the character of the organic matter present can be told, according to Smart, by the rate at which the ammonia is evolved, gradual evolution signifying fresh pollution, while rapid production shows the organic matter in a more advanced state of decomposition.

The necessity for early analysis is to be observed in the change which the ammonias undergo in waters that are allowed to stand for some days, the free ammonia gradually being converted into nitrites and nitrates. The organic ammonia is more stable, but it, too, in time breaks down and passes into the "free" stage as a result of biological changes.

126. Oxygen Consumption.—Another method of determining organic matter is to find out how much oxygen is required to oxidize the matter present in a water. Generally in a water deficient in unoxidized substances, as ferrous salts, nitrites, etc., the carbon of organic matter readily takes up oxygen, so that a determination of this capacity for a standard length of time enables the amount of organic matter to be approximately determined. This is accomplished by using an acidified solution of potassium permanganate. Often two determinations are made; one for 10 or 15 minutes, in which the readily oxidized matter, as nitrites, ferrous salts, and sulfides, are acted on; the other for a number of hours, during which the less readily oxidizable organic matter will be acted on. Surface-waters carrying suspended matter, or peaty waters, also show a high oxygen-consuming capacity.

127. **Nitrites.** — Nitrites represent nitrogenous matter in an intermediate stage of decomposition, and therefore their presence signifies present pollution with organic matter in which germ-life is active, and is, therefore, an unfavorable symptom in water if present in any considerable degree. This salt may occur as a result of the incomplete oxidation of ammonia products by the nitrifying organisms, or it may sometimes be formed by the reduction of the more stable nitrates by the denitrifying bacteria which abound in decomposing organic matter. Usually such substances are absent in good well-waters, but if present they may be due to reduction processes, the change often being accomplished by such mineral substances as ferrous oxid. In such instances the existence of nitrites may have no sanitary significance, as they are not likely to be associated with disease-producing bacteria. The presence of high nitrites and high free ammonia is usually indicative of sewage pollution either in surface or subterranean waters.

128. **Nitrates.**—These salts represent the ultimate, the final stage into which nitrogen is changed by the biological processes in soil and water. In this form nitrogen is more stable, and these salts therefore collect in the soil, subject only to leaching, and the use they play in the development of the green plant. Their presence therefore may indicate merely past pollution, without any present danger. Not infrequently deep wells may contain high nitrates without suspicion being cast on the quality of the water; but if associated with free ammonia or nitrites, it is evidence of incomplete oxidation. The higher nitrogen content of animal in comparison with vegetable matter is generally betrayed in the amount of nitrates present in a water.

129. **Summary.**—From the foregoing considerations it is manifestly impossible to determine the character of a water by the use of a single test. The substances that accompany sewage, which is primarily dangerous on account of the disease-producing micro-organisms that it may contain, are so frequently found in connection with waters that have had no opportunity for dangerous pollution, that the analyst must use the greatest care in interpreting the results of an analysis. Chlorine and nitrites as such, for example, are not dangerous to human health, but it is because these substances prevail in waters that are polluted with dangerous matter. If their presence was characteristic of sewage only, then the matter of sanitary water-analysis would be reduced to simple terms, but unfortunately such is not the case.

The most that a chemical analysis can do is to prove the presence of organic matter that may be a source of pollution. It throws no light on the question as to whether the same is actually disease-producing or not. Even though sewage is shown to have polluted a water, this does not prove it to be absolutely dangerous; but of course if the possibility of pollution is present, all it requires is the accident of disease to start an epidemic. The history of polluted waters is so uniformly in harmony with the view that typhoid is so distributed that, generally speaking, no further proof is required. (See Chapter X.)

BACTERIAL EXAMINATION OF WATER.

130. **Development of Methods.** — Inasmuch as the specific organisms of disease which are the really dangerous and polluting elements in a water are for the most part included in that group of lower plant-forms known as the bacteria, it might naturally be thought that the bacterial examination of a water would quickly and satisfactorily solve the question as to the wholesomeness of any supply, but, for reasons cited before (112), such is not the case. In comparison with the chemical methods of investigation, the technique of bacteriological methods is of recent introduction, being based on the epoch-making discoveries of Koch, made in the early eighties. Much improvement has taken place in the development and unification of methods, but even yet analytical practice is not as uniform as in chemical manipulation. Bacteriological methods have, however, aided greatly in sanitary analysis, and it is quite necessary that they should be utilized to gain the most accurate idea of the sanitary quality of the water-supply.

131. **Scope of Bacterial Tests.** — The information to be obtained by the various bacterial tests of waters is principally as follows:

1. Detection of presence of sewage or foreign pollution which may or may not be associated with infective matter. In this respect the bacterial method embracing both quantitative and qualitative work is practically coordinate with the usual sanitary chemical analysis.

2. Quantitative bacterial analysis affords a very sensitive measure for making comparative tests as to distance to which pollution can be traced in a stream or lake, to establish presence of leaks in submerged pipes and to study effect of external conditions; in fact, the determination of many variations in quality. In this respect it is often a more accurate measure than a chemical determination.

3. In the control of the operation of filters bacterial analysis is very much superior to any other method, for the reason that it determines directly the number of organisms before and after filtration. In a chemical analysis so many of the determinations are of substances in solution which readily pass a filter that will hold back the dangerous suspended matter (bacteria).

4. The isolation and study of pathogenic organisms from waters. This is generally done by combining cultures on artificial media with animal experiments.

132. Methods of Determining Bacteria. — The bacteria are altogether too small to permit of individual recognition by simple microscopic examination of water. Their number * and general character is determined by adding the water to be examined to various kinds of culture media, i.e., food substances in which bacteria can readily grow. Then as each organism develops, a tiny aggregation of cells is produced which is made up of organisms that belong to a single species. Such a mass of germs is known as a "colony." A "pure culture" is then made by transferring a bit of this colony growth to tubes filled with sterile culture media, on which there appears in due time the characteristic growth of the germ in question. For culture purposes gelatin or agar is used. Making a satisfactory culture medium requires considerable care, especially as to the proper chemical reaction of the same. Slight variations in this regard are the cause of wide differences in results, a condition which readily explains the discrepancies frequently noted between different observers.

In studying the bacteria various liquid and other solid media are constantly made use of, for the purpose of differentiating species, but the technique of their preparation and use is a question that concerns the bacteriologist rather than the sanitary engineer.

133. Multiplication of Bacteria in Collected Sample. — If water samples are allowed to stand at ordinary temperatures before cultures are made, the accuracy of quantitative determinations is much reduced. This is due to the very rapid growth of the bacteria in the water after sampling. Often the development in such cases is enormous for a few days, and then a marked decrease may occur.

This fact has considerable bearing on the question of analyzing water samples from a distance. Unquestionably, for quantitative results it is preferable to make culture-plates at the time samples are collected, but frequently this cannot be done; in which case they should be maintained at low temperatures in full bottles during transportation. Frankland † has noted that in bottles closed with cotton stoppers growth was very marked, while in tightly sealed bottles filled *completely* with water practically there was no development.

* The number of bacteria in any given sample is invariably expressed in number of organisms per cubic centimeter (cc.) which is approximately one-third of a teaspoonful.

† Micro-organisms in water, p. 234.

134. Quantitative Bacterial Analysis.—Although too much stress in the past has been laid on the simple quantitative enumeration of bacteria in a water as an index of its quality, yet, notwithstanding this, the determination of mere numbers when properly controlled gives considerable information concerning a water. It is wholly an erroneous conception that the quality of a supply can be measured by a mere numerical estimate, for there are so many disturbing factors that modify this determination that as a standard it has no value. Improper selection of samples, slight possible contamination with unsterile surfaces at time of sampling, development of bacteria in sample before cultures are prepared, slight variations in composition of media, different kinds of media, variation in incubation temperature, in moisture of culture-dish, the possible error due to small quantity of water tested, and numerous other conditions, all contribute to make a numerical estimate too delicate a measure. It is therefore impossible to propose a quantitative norm or standard, and pass or reject waters on such an arbitrary basis.

Still the previous history of a water is to a large extent revealed in a bacterial enumeration of a properly handled sample. Waters that have come in contact with the bacteria-rich upper soil-layers normally contain a higher number than waters of subterranean origin. If then the normal condition of a water is known, a marked quantitative increase indicates pollution from some outside source. The germ content of various waters noted in Chapter IX will indicate in a general way the normal condition, and will thus serve as a basis for comparison. Generally speaking, good waters have relatively few bacteria, but it does not necessarily follow that a water rich in bacteria is necessarily poor in quality.

For comparative estimates the quantitative determination of bacteria is often more sensitive than any other method of testing. In studying the efficiency of filter operations, or the natural purification of a stream or lake polluted with sewage or surface drainage, this method is of great value, as in the case of the Toronto water-supply, where the intake-pipe was broken near shore and so permitted the entrance of water from the polluted shore region.*

Where the natural variation in germ content between the two waters compared is marked, this method is of no avail, but its usefulness decreases as the normal bacterial contents of the compared samples approximate each other.

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* Jour. N. E. Water-Works Assn., June, 1896, p. 211.

135. **Qualitative Bacterial Analysis.** — While the quantitative enumeration of bacteria in any given sample is under proper conditions an index of some value of the relation which such sample bears to the bacterial content of the soil, a determination of the nature and kind of germ life present is of much more significance in studying the quality of waters. The typhoid bacillus and other disease-producing organisms that are invested with special interest by reason of the fact that they are disseminated through the medium of water-supplies, find their way into such water-supplies generally through introduction of human excreta. The intestinal tract of animal life offers an abundant opportunity for the development of bacteria, and it is therefore a question of prime importance whether there is a more or less distinctively bacterial flora of the intestine. Numerous culture methods have been devised for the detection of organisms of a sewage type, some of which are of material value as approximate methods of determining the general character of any supply.

136. **Presumptive Tests.** — Whipple has applied this term to certain tests which may be used with waters for the purpose of determining approximately the origin and condition of samples tested. These presumptive tests rest upon certain biological peculiarities of bacteria commonly found in the intestinal canal. Bacteria accustomed to a habitat like the intestinal canal of warm-blooded animals naturally have a higher optimum growing temperature than normal water bacteria. As a class intestinal bacteria are fermentative forms and generally possess the property of fermenting certain sugars forming acid and gaseous by-products.

137. **Litmus-Lactose Agar Test.** — When polluted water is added to litmus-lactose (milk sugar) agar (Wurtz' method), and incubated at body heat (98°–100° F.), abundant bacterial growth takes place and numerous strongly acid (red) colonies develop. An unpolluted supply usually shows but slight development, and few, if any, strongly acid colonies, as these types are not as a class able to thrive luxuriantly at blood heat, and produce the fermenting changes commonly obtained with fecal types.

138. **Fermentation Tests.** — Sewage bacteria are usually able to ferment dextrose sugar solutions with the formation of acid and gaseous by-products. The addition of varying quantities of water to dextrose in fermentation tubes enables the analyst to determine readily whether gas-generating bacteria are present. While these so-called presumptive tests may be very readily applied, they should not be regarded as final in determining the quality of water, especially in the case of surface

waters.* More value is to be attached to negative results than positive findings as total freedom of acid-forming gas-generating organisms in a water sample of one to ten cc. is only associated with unpolluted waters.

In case of positive findings by these presumptive tests, the suspected species should be isolated and carefully studied by differential methods in order to determine with exactness the characteristics of the organisms.

139. **Number of Species.** — The bacterial flora of a water is of course subject to more or less change, due to variation in environmental factors, but at any single time the number of species in an unpolluted supply, even though of surface origin, is not generally very large. Where pollution has arisen from the introduction of decomposing material rich in organisms, not only in number but often in kind, the number of species present will be increased. Some have placed an arbitrary limit on the number that ought not to be exceeded (Migula's standard is 10),† but such conclusions cannot be drawn with safety. The gelatin-plate cultures afford the best medium for this differentiation of species.

140. **Significance of Liquefying Bacteria.** — In growing on gelatin plates, bacteria are either able or unable to render gelatin fluid. Putrefactive organisms are often liquefying species, and hence an abnormally high percentage of liquefying colonies is considered undesirable in a water. Such a condition is certainly abnormal, but it is hardly possible to attach much specific importance to this finding, for all natural waters normally contain liquefying species, although they are usually much less numerous than the non-liquefying forms.

The separation of individual species is generally made from the culture-plates prepared for quantitative work. Where the colonies on gelatin or agar plates are separate from each other, pure cultures of the different forms should be made. It is not customary in a sanitary examination to make a detailed study of all the different forms found in a water because of the time required, but if it is desirable for future study to separate any species that appear on the gelatin plates, it can best be done at this time.

141. **Significance of Colon Bacillus.** — The significance of the colon organism has been a subject of much discussion. Originally this species was found in the contents of the human intestine and was thought to be characteristic of fecal pollution, but more thorough examination shows

* Gage, XXXIII. Mass. Report 397, 1901.

† Prac. Bact., English trans., p. 167.

that it is a common inhabitant of the intestinal tract of domestic animals* and lower forms of life. It has been found in abundance in the intestinal contents of mammalia and birds. Amyot † and also Johnson ‡ have found it frequently in fishes, and Clark § has noted its presence in shell fish, especially where such water forms of life were associated with polluted waters.

Some investigators have held that the colon organism is so ubiquitously distributed that it possesses no value as a sewage type.‖ Prescott ¶ reports finding a type on cereals and mill feeds that cannot be distinguished from colon. Recently, its presence as an index of fecal pollution has, therefore, been somewhat discredited, especially where surface waters were under consideration.

In spite of these differences of opinion among bacteriologists, there is no question but that the colon test properly performed is of great service in determining the quality of any supply. In deciding the case as to whether pollution exists or not, much more emphasis, however, should be laid on the number of colon bacilli found than its mere presence. Moreover, in large samples of water (100 cc.–500 cc.) positive findings are not as significant as in smaller samples, as the occasional presence of this widely spread type is not regarded as of vital importance. If, however, a large percentage of one cc. tests reveal this germ, as shown by characteristic cultures and reactions, it is generally regarded as indicating an unsafe condition in a water-supply.

142. **Importance of Other Sewerage Types.** — Two other forms have been isolated from polluted waters that are thought to bear a more or less direct relation to sewage pollution. The spore-bearing sewage anærobe of Klein, Bacillus sporogenes is generally found in sewage, but it is much less abundant than the colon type. More recently sewage streptococci ** have been readily and abundantly demonstrated in recently polluted waters †† and in presumably unpolluted waters they are apparently absent. ‡‡

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* Moore, V. A., and Wright, F. R., *B. Coli* from different species of animals, *Jo. Bost. Soc. Med. Sci.*, IV.: 175, 1900. Dyar and Keith, *Tech. Quarterly*, VI.: 256, 1893. Theobald Smith, *Cent. für Bakt.*, XVIII.: 494, 1895.

† Amyot, Trans. A.P.H.A., XXVII.: 400, 1901.

‡ Johnson, Trans. A.P.H.A., XXIX.: 385, 1903.

§ Clark and Gage, Proceedings A.P.H.A., XXIX.: 386, 1903.

‖ Weissenfeld, *Zeit. für Hyg.*, XXXV.: 78, 1900.

¶ Prescott, *Medicine*, XI.: 20, 1903.

** Streptococci are round celled types that develop in long chains.

†† Houston, A. C., 28 Rep. Loc. Govt. Bd., Med. Supp. 469, 1898. Winslow and Hunnewell, *Jo. Med. Res.*, VIII.: 502, 1902.

‡‡ Winslow and Nibecker, *Tech. Quart.*, XVI.. 227, 1903.

143. Isolation of Sewage Types. — The separation and identification of the sewage forms previously referred to requires considerable time and previous experience, so that detailed examinations of this sort are preferably to be left to the laboratory expert rather than attempted by the engineer.

A large number of methods have been devised and perfected, in most of which the principle of encouraging the rapid growth of *B. coli* is followed by placing the water sample under extremely favorable conditions for the growth of such species. By addition of small quantities of phenol, growth of the water bacteria is largely inhibited. The addition of readily fermentable sugars, as dextrose, permits of the formation of characteristic gases (H and CO_2) which are produced in quite definite proportions. Other detailed characters are to be noted that can be found on referring to any standard bacteriological text-book.

The sewage streptococci are also readily separated from the presumptive cultures. Prescott has shown that in sewage mixtures the colon organism develops quickly in dextrose broth and is later supplanted by the streptococci. By isolating the organisms at different stages of development it is possible to secure data on presence of both types from the same plate.*

144. Quantitative Estimation of Colon Type. — The quantitative estimation of the colon group is essential in interpreting the character of a supply. This was first done by Theobald Smith, who suggested the inoculation of dextrose fermentation tubes with small quantities of water, varying from tenths to hundredths of a cubic centimeter. Development of gas in a series of 0.3 cc. samples, but not in those inoculated with 0.1 cc. would indicate at least 3 but not 10 colon organisms per cc. The mere presence occasionally of organisms of colon type is not considered as sufficient evidence to warrant condemnation of water-supply, but if this type is found continuously and abundantly, it speaks strongly for evidence of pollution.

145. Animal Tests. — Some investigators † follow the practice of inoculating directly into animals a beef-broth culture made by adding water direct. Varying quantities of water are incubated in beef bouillon or a peptone solution, and such animals as white mice, white rats, guinea-pigs, doves, or rabbits are inoculated with varying quantities of the culture. The animal may be killed by the toxic products formed in the culture, or it may die from direct infection. This can be readily

* Prescott and Winslow, Elem. Water Bact., p. 104.
† Vaughan, *Arch. f. Hyg.*, XXXVI., p. 190.

determined by making subcultures from such organs as the liver, spleen, or kidney. It does not necessarily follow that organisms capable of killing lower animals are able to cause disease in the human, but the presence of such forms is certainly undesirable in water, and supplies containing such are generally regarded as polluted.

146. **Concentration of Organisms in Water.** — Where the degree of pollution is very slight, it oftentimes becomes very difficult to determine the presence of dangerous bacteria. It must be kept in mind that water suitable for human use is not generally adapted to the growth of specific pathogenic bacteria (222); consequently such organisms may be present in such sparse numbers as to elude detection. Then, too, the amount of water that is ordinarily subjected to a bacteriological test is so small as to render it difficult to determine the presence of occasional forms.

Filtration. — When necessary the germ content of a water can be concentrated by filtering a relatively large quantity through a germ-proof filter (Pasteur or Berkefeld system. Cultures can then be made of the sediment adhering to the filter.

Enrichment Cultures. — Another method is to incubate the water sample under such conditions as to composition of culture medium, temperature, etc., as to cause certain types of organisms to grow luxuriantly while possibly holding back other forms not desired. With some bacteria that are of importance in water analysis (*B. coli*, *Sp. choleræ Asiatica*), these enrichment methods are successfully used; but unfortunately with the typhoid organism no method has yet been devised that can be employed in a thoroughly satisfactory manner.

It is evident that the presence of distinctively pathogenic bacteria is sufficient to condemn any supply for potable purposes, but the brief existence of these forms in drinking-waters makes it difficult to use such a standard for the practical determination of the quality of water-supplies. While of course it would be desirable to be able to isolate such from suspected waters, yet direct proof of their presence is not necessary to justify a condemnation of a supply. If a water shows unmistakable evidences of sewage pollution, this in itself is sufficient proof to warrant the same being considered dangerous. If this fact is associated with an increase in typhoid cases especially, the proof is practically as strong as if the typhoid germ itself were found therein.

147. **Detection of Specific Disease Bacteria.** — Not infrequently are *B. coli* and the *Proteus* species found in pathological processes in the

human body, but nevertheless these species are not usually regarded as pathogenic. Typhoid fever, cholera, and dysentery are the distinctively water-transmitted diseases. It might with propriety be thought that the bacterial method would permit of their ready detection, but as a matter of fact it does not. There are several reasons why this is so.

1. These pathogenic microbes do not find in drinking-water a favorable environment. They may live in such a medium for some time (222), but it is questionable whether under ordinary conditions actual multiplication of cells takes place unless there is a degree of pollution due to influx of organic matter that practically makes a culture medium of the water.

2. Owing to the considerable period of incubation (9–14 days in the case of typhoid) that must elapse between time of infection and appearance of outbreak before waters would ordinarily be subjected to examination, it is quite probable that the disease germ may frequently have disappeared.

3. Difficulty of detection is increased because ordinarily the amount of water submitted to examination is only a few cc. at most, unless the concentration of bacterial life by filtration is resorted to.

4. Inability, especially in the case of typhoid, to find an elective medium that will permit of the rapid growth of this germ, while at the same time retarding the development of *B. coli* or other luxuriant congeners.

These reasons suffice to show some of the difficulties that the analyst has to contend with in this phase of his work, yet, in spite of these unfavorable conditions, the presence of such disease organisms as cholera and typhoid has been determined in a considerable number of cases. It should be said, however, that in these cases the conditions were rendered especially favorable through the timely search and facilities for such examinations.

The methods that are the most successful in the isolation of specific organisms are those which permit of a preliminary development of the water sample under conditions extremely favorable for the growth of the species for which search is made. The use of elective media therefore necessitates the introduction of different methods in each case, for, as a matter of fact, the biological requirements of the different pathogenic bacteria are rarely similar.

148. Isolation of Typhoid Organism. — Much endeavor has been made by bacteriologists to find a suitable culture medium that would permit of the ready separation of the typhoid bacillus from its closely related associate, the colon bacillus. A number of the technical methods proposed have been discarded after a varying amount of use when it was found that strains of diverse origin gave unsatisfactory results, but several are now in quite general use as furnishing suitable means of differentiation. For the most part, substances are added which have a tendency to repress the development of the saprophytic water forms. Thus, the addition of crystal violet inhibits in large measure the ordinary types of organisms found in water. The addition of small quantities of phenol or carbolic acid causes the same effect, although the action on both the typhoid and colon organism is not nearly as marked. The typhoid organism can be differentiated from the colon type by virtue of its difference in acid and gas production.

These tests all require so much experience that they can only be applied by the expert. They are mentioned here as indicating that proper tests for satisfactory differentiation do exist and should be used where necessary.

In making the final culture tests certain physiological reactions serve to distinguish quite sharply the typhoid from the colon germ.

In contradistinction to *B. coli*, *B. typhosus* does not ferment sugar solutions of any kind in the fermentation-tube, neither does it produce indol. It does curdle milk in time, although the acid production in comparison with *B. coli* is much less. Since the introduction of the Widal test in diagnosing typhoid fever, it has become possible to take advantage of a reaction that is so specific as to be of greatest service. If a fresh culture of a genuine typhoid organism is brought in contact with the blood of a person suffering from this disease, the bacilli lose their motility and become aggregated in clumps, a phenomenon known as the Widal reaction, now so extensively used in the diagnosis of this disease in the human. By taking advantage of this fact, it is possible to test a doubtful germ against a positively known typhoid blood. If the isolated culture gives the Widal reaction with known typhoid blood and does not with perfectly healthy blood, the evidence as to nature of the organism in question is practically decided, for when properly examined the per cent of accurate returns from this test is very high, approximating the possible limit.

While the typhoid organism has been reported as having been found more or less frequently in waters of varying character, yet those cases that are reported prior to the introduction of the "agglutination test" are now looked upon with suspicion.*

* For fuller discussion of this subject, see bibliography appended to Chapter X "on the detection of pathogenic bacteria in water."

149. Isolation of Cholera.—This organism grows with great rapidity in alkaline solutions of peptone and salt. By taking advantage of this characteristic and incubating suspected samples of water at blood-heat, the cholera spirillum can be greatly increased in number so that a subsequent examination of the surface pellicle will generally indicate the presence of cholera-like organisms. If positive microscopic findings are made by this enrichment method, the preparation of subcultures in various media will soon tell positively whether the organism is the genuine "comma bacillus" of cholera or a spirillum of similar form, a number of which occur in flowing or surface waters.

The culture characters of the cholera germ are fairly distinctive, but there are two tests that are considered so specific as greatly to aid in diagnosis. These are the cholera-red reaction (indol test) and Pfeiffer's phenomenon. Tests of this character can be made only by the bacteriological expert.

150. Disinfection of Polluted Wells and Pipes.—It may happen that wells and water systems may sometimes become temporarily polluted with disease-producing matter, without such material continuing to find its way into the same. Under such circumstances it is necessary to disinfect the water system in such a way as thoroughly to destroy all disease organisms. These methods should not be interpreted as applying to wells that are so poorly constructed that surface-drainage cannot be kept out. Such wells should be condemned and closed. Open or dug wells are much harder to disinfect thoroughly than tubular wells, owing to the larger cubical content, but more particularly to the loose and open character of the walls. Driven or drilled wells enclosed in iron pipes can be disinfected with little or no difficulty should they happen to become infected.

For this purpose several methods have been used. Neisser found that steam could be very successfully employed. A pressure of 50–60 pounds per square inch succeeded in raising the temperature of a well containing about 500 gallons from 50° to 210° F. in $2\frac{1}{2}$ hours. This destroyed all trace of the organisms added, although it did not render the well wholly sterile.

A solution of crude carbolic and sulfuric acid can also be added to wells with good results.* In old wells, particularly those that are open, dirt collects in the bottom, in which case the bacteria retain their vitality for some time. The disappearance of the carbolic acid in water can be detected by applying ferric chloride.

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* Fränkel, *Zeit. f. Hyg.*, VI. p. 23.

Sometimes it becomes necessary to disinfect the whole hydrant system. According to Stutzer * 0.05 per cent solution of sulfuric acid suffices to destroy the cholera organism in 15 minutes in distilled water. As this acid unites readily with the alkaline earths and iron present in the water, it is necessary to increase the amount added. For actual disinfection work he used 0.2 per cent. The acid solution is allowed to fill the entire system, remain in contact with the same a number of hours, and is then flushed out. In disinfecting the water-mains after the cholera epidemic of Hohenlohehütte and the typhoid outbreak in Freiburg, he found that it took about three days to remove all trace of the acid, but the bacterial tests of the water were then found to be wholly satisfactory.

151. **Bacterial Control of Filter Operations.**—To determine the efficiency of a filter system as a means of purifying water-supplies, the bacterial method of examination has evident advantages. This is done by making a quantitative bacterial examination of the water before and after being applied to the filter. A chemical analysis generally shows but little improvement because most of the substances determined are of a soluble nature, and therefore readily pass the pores of the filter. The real elements of danger in water, however, are the living organisms—the disease bacteria, and these are prevented, by reason of their insoluble nature, from passing through a properly constructed filter.

Of course there is no differentiation in the filter between those species capable of producing disease and the harmless water inhabitants, but a determination of the percentage removed from water during filtration gives an approximate estimate of the degree of efficiency of the filtering process. At first it was throught that an enumeration of the number of organisms in the applied water and the effluent would give the exact extent of purification, but later it was found that some bacteria possess the ability of growing in the body of the filter and underdrains, and so the number in the effluent may not represent the actual number passing the filter.

Later the custom was introduced of applying cultures of some specific kinds of bacteria not normally found in the filter sand, and determining the number of such organisms in the effluent. *Bacillus prodigiosus*, one of the most characteristic pigment-producing bacteria, has been used for this purpose to a considerable extent, but of late years, *B. coli communis* † has been more extensively employed because

* *Zeit. f. Hyg.*, XIV. p. 116.

† Clark and Gage, *Jour. Bost. Soc. Med. Sc.*, 1900, IV. p. 172.

of its closer relation to disease bacteria and the fact that it is in a sense an index of fecal pollution.

The importance of a careful examination of filter-works by this method is especially recognized in Germany, where every municipality using sand-filtered water is obliged to make frequent reports, especially on the bacterial results, to the Imperial Board of Health, as to the working of the filters.

MICROSCOPICAL EXAMINATION OF WATER.

152. **Scope of Microscopic Examinations.** — In the microscopical examination of water a determination of the suspended matter other than bacteria is generally included. This may embrace particles of inorganic as well as of organic origin. An opalescent water may sometimes be caused by extremely fine fragments of clay that may even be so small as to pass a filter. Quartz splinters or particles of iron oxide also not infrequently occur. These inorganic materials have, however, no sanitary significance, but their recognition becomes a matter of import only as explaining the physical condition of water.

Of far more importance is the material of organic origin. Much may be learned of the nature of a water and its possible sources of pollution by a microscopic examination, which generally permits of a differentiation between matters of animal and vegetable character. A recognition of any fibers, such as cotton, wool, or flax, starch grains, and undigested muscular tissue indicates a source of pollution generally due to household wastes.

In matter of distinctively fecal origin it is possible that eggs of some of the intestinal parasites of man and animals may be present. Many of these retain their reproductive powers for a long time, but fortunately are unable to develop in man directly, requiring an intermediate host (201).

In addition to such microscopic findings as reveal the presence of suspended particles that are often closely related to house-refuse, there are a large number of living organisms whose natural habitat is that of water. These may be either animal or vegetal (183). Generally speaking, their presence in water-supplies is not such as to render the water dangerous to human health;* but not infrequently the physical qualities of the water (taste, odor, color) may be profoundly modified by their presence. As Whipple well says, bacteria may render a water unsafe, but other microscopic organisms are likely to make it unsavory.

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* Neisser, *Zeit. f. Hyg.*, XXII. p. 475.

A direct microscopical examination will not generally reveal many forms unless precautions are taken to concentrate the same in a small volume. For this purpose plain sedimentation will not suffice, but a method of filtering large quantities of a water through sand has been generally adopted (Sedgwick-Rafter method).*

In many waters organisms of this class occur only sparingly, or they possess no disagreeable properties that impair the quality of the water; hence their presence is of no particular import. In other cases certain species are so abundant that the quality of the water is distinctly injured by their presence.

Difficulties of this sort are quite apt to occur in stored waters, as in ponds or reservoirs, for the access of light is necessary for the development of these plant-forms. Filtered or ground-waters are very prone to develop these troubles unless reservoirs are covered.

153. Direct Microscopic Examination in Filtration-work. — The microscopical method of examination is sometimes of service in comparing waters, as in the case of sand-filters or in filtration-galleries. One of the writers once had an opportunity successfully to use this method in determining the presence of a leak in a submerged pipe, the outer water being a surface-water and therefore containing algæ. In determining the efficiency of filtration in filter-galleries, it is necessary to use freshly filtered waters, as microscopic organisms are likely to develop rapidly in such waters open to the sunlight. In Taunton, Mass., trouble was experienced in the water from a filter-gallery from the growth of both *Asterionella* and *Dinobryon* (183).

SANITARY SURVEYS.

154. Object and Value.—The normal condition of the water-supply of different regions is subject to considerable variation. Even that of the ground-water, which is generally supposed to be more stable, fluctuates in different parts of the country with reference to many of its constituents. In some cases local causes are operative in changing the nature of the supply, as in the case of hard waters in limestone regions. The same holds true with reference to the proximity of the sea, the chlorine content gradually diminishing as the distance from the sea increases (123).

In different States these sanitary water-surveys are being taken up by the respective Health Boards, and the normal condition of the water-supplies determined. These afford a basis for comparison that

* For details of apparatus and use, see Whipple's Microscopy of Drinking-water, p. 15.

enables the analyst to judge more accurately as to whether any water he is testing is abnormal or not for the region from which the sample comes.

Not only are these sanitary surveys being made of the ground-water supplies, but the surface-waters are now receiving considerable attention. The importance of this is readily recognized when one considers that the supplies for our larger municipalities must of necessity be drawn from open waters, as these are often the only adequate sources that can be used. With the steady growth in our urban populations and the consequent increased danger of pollution, it becomes more and more necessary to secure these supplies from distant sources that are free from pollution, or to purify those that are more available and more likely to be polluted.

To keep close check on the effect that the constant increase in population has, it is necessary to know the normal conditions of a water-supply, both chemically and bacteriologically. If these surveys are made before the sources are polluted, then a standard of comparison can be had from which the effect of a growing population can be determined. For instance, the Massachusetts and the English sanitary authorities estimate that the increase in chlorine content is between .4 and .5 part per million for an increase of every 100 persons per square mile.

The absolute necessity of thus determining the quality of waters that are to serve as sources of supply for large cities is evident, but these sanitary surveys are now being extended so as to include entire river systems. Several of the States, as Ohio, Illinois and Minnesota, are engaged in making a study of the surface waters within their limits. Cities situated on large streams very often use these as natural drainage-channels for sewage disposal. The consequence is that the pollution in such streams is constantly increasing, so that the municipalities situated farther down-stream are in danger of having their most available source of water-supply polluted from the wastes of other towns. It is true that there is a natural purification process (168) going on in such rivers, but the question is always pertinent as to whether such natural processes are wholly able to purify the water. Mere loss in turbidity is no criterion to depend upon in settling this question. To obtain a basis from which to determine whether conditions are materially changed as density of population increases, these sanitary surveys are of great value, but they should always embrace a chemical and bacteriological examination and preferably engineering data should also be accumulated.

LITERATURE.

For a more detailed consideration of different phases of sanitary analysis, reference may be made to the following list. This list is not intended to be exhaustive, but merely comprehensive enough to direct the sanitary engineer to the more important publications relating to the sanitary analysis of water, and the interpretation of such work. The technical water analyst will need to consult much of the chemical and bacteriological periodical literature in order to learn of methods available for his work.

Questions of technique are considered more or less in detail in all of the following books:

P. & G. C. Frankland. Micro-organisms in Water. 1894.
A résumé of the bacteriological phase of the subject, including a description of over 200 species of bacteria found in waters.

Tiemann-Gaertner. Handbuch d. Untersuchung u. Beurtheilung d. Wasser, Vierte Auflage. 1895.
A complete handbook on matters relating to both chemical and bacteriological examination of water-supplies.

Loeffler, Oesten and Sendtner. Wasserversorgung, Wasseruntersuchung u. Wasserbeurtheilung. 1896. (In Weyl's Handbuch der Hygiene.)
Very useful to the engineer as well as the water analyst.

Leffmann and Beam. Examination of Water for Sanitary and Technical Purposes. 1895.
Confined to the chemistry of the subject.

Pearmain and Moor. Chemical and Biological Analysis of Water. 1899.

Mason. Examination of Water. 1899.
A revised reprint of two chapters on chemical and bacteriological examination of water included in his larger, more general work on Water-supply which appeared in 1896.

Fuller, G. W. Water Purification at Louisville. 1898; also,—Report on Water Filtration at Cincinnati. 1899.
While primarily concerned with filtration experiments, yet valuable for full exposition of analytical methods.

Hill, John W. Public Water-supplies, 1898. Although written from the general engineering point of view, this work contains valuable data that will be of use not only to the general student but the technical analyst as well.

Williston, Smith, Lee, and Foote. Rept. on Exam. of Conn. Water-supplies. 14 Rept. Conn. Bd. Health, 1891, p. 231.

Whipple. The Microscopy of Drinking-water. 1899.
A most complete presentation of the relation of microscopic organisms other than the bacteria to water-supplies, including a classification of such organisms as far as genera. An indispensable book to the student of this phase of water investigation.

Savage, W. G. Water Bacteriology, 1907.

Sedgwick, W. T. Principles of Sanitary Science and Public Health.
A general exposition on hygiene, but includes several excellent chapters on the relation of water as a vehicle of infectious diseases.

Horrock, W. H. An Introduction to the Bacteriological Examination of Water, 1901.

Whipple, George C. The Value of Pure Water and Study of the Different Characteristics of Water and What They Cost the Consumer, 1907.

Prescott and Winslow. The Elements of Water Bacteriology.
An excellent up-to-date presentation of the subject from the bacteriological point of view. Second Edition, 1908.

The Bibliography of analytical methods of water analysis is quite voluminous, and widely scattered in numerous scientific journals, as well as more technical publications. Besides the references already given as foot-notes to the text, the reader is referred to the following list of papers that includes those of general interest, as well as some that relate more specifically to the technique of water examination:

Annual Reports of the Massachusetts State Board of Health.
The State Board of Health of Massachusetts has for a number of years carried on extensive experimental researches on water and sewage as well as methods of control of both. The publications of this Board form one of the most valuable contributions to the literature of water analysis and should be carefully studied by every student of this subject.

Report of Committee on Standard Methods of Water Analysis of the American Public Health Association.
This Association appointed a committee in 1897 to formulate methods of procedure relating to water examinations. This report published in the transactions of the American Public Health Association, Vol. XXVII, 1902, forms the basis of laboratory procedures relating to physical, microscopical, chemical and bacteriological methods of water examinations.

Frankland, Percy. The Hygienic Value of the Bacteriological Examination of Water. Trans. 7th Internat. Cong. of Hygiene and Demog., London, 1891.

Kruse, W. Kritische u. experimentelle Beitraege z. hygien. Beurtheilung d. Wassers. Zeit. f. Hyg., 1894, XVII. p. 1.

Korn and Kammann. The Hamburger test for Pollution. Gesundheits Ingenieur, March 16, 1907.

Winslow, C. E. A. Bacteriological Analysis of Water and Its Interpretation. Jour. N. E. W. W. Assn., December, 1901.

Clark and Gage. Value of Tests for Bacteria of Special Types as an Index of Pollution. Report Mass. Board of Health, 1902.

Whipple, George C. Practical Value of Presumptive Tests for B. coli in Water. Tech. Quart. March, 1903.

Hesse, W. and Niedner. Methods of Bacteriological Water Examination. Zeit. f. Hyg. XXIX. p. 454, 1898.

Hill and Ellms. Apparatus for Collection of Water Samples for Chemical, Microscopical, and Bacteriological Analysis. Trans. American Public Health Association, XXIII. p. 193, 1898.

Houston, A. C. Value of Examination of Water for Streptococci and Staphylococci. Supp. to 29th Report L. G. B. of England containing Report of Med. Off. for 1899–1900, p. 458.

MacConkey. Experiments on differentiation of B. coli and B. typhosus by use of sugars and bile salts. Thompson–Yates Laboratory, Report III. p. 41, 1900, also ibid. IV. p. 151, 1901.

Winslow. Bacteriological Examination of Water and Its Interpretation. Jour. N. E. W. W. Assn., XV. p. 459, 1901.

Winslow and Nibecker. Significance of Bacteriological Methods in Sanitary Water Analysis. Tech. Qu., XVI. p. 227, 1903.

Sanitary Surveys. Sanitary surveys of individual streams, watersheds furnishing municipal supplies, and in some cases general state surveys have been or are being made, generally under public auspices. In some cases these surveys have been undertaken from the chemical point of view; in other instances both chemical and bacteriological examinations have been made.

The most extensive work yet performed is that done under the auspices of the Massachusetts State Board of Health. (See Report on Exam. of Water-supplies, 1890 *et seq.*)

Similar examinations have also been made in Connecticut (18 Rept. Conn. Bd. Health, 1895, p. 230).

In New York a careful sanitary survey has been made of the Croton water-shed, the base of supply for New York City (9 Rept. N. Y. State Bd. Health, 1889, p. 189); also a chemical and bacteriological study of the Mohawk–Hudson valleys (12 and 13 Rept. N. Y. State Bd. Health).

Similar surveys were begun by the Ohio Bd. of Health in 1897. Two reports have already been issued (1897 and 1900), embracing the results obtained in the study of five of the larger river systems of the State.

In the State of Illinois the State Board of Health is making a chemical survey of the water-supplies. (Report published 1897.)

Report of Streams. Examinations of waters between Lake Michigan at Chicago and the Mississippi River at St. Louis issued by the Sanitary District of Chicago, 1902.

A complete study of the biological and chemical relations of these waters made prior to the opening of the Chicago Drainage Canal.

CHAPTER IX.

QUALITY OF WATER.

155. Importance of Quality.—In securing a water-supply for public or for private use, the question of quality is of supreme importance. An adequate or copious supply is not so much to be desired if it means that quantity must be purchased at the expense of quality. In this respect European cities are much ahead of American municipalities. The per capita consumption in this country is greatly in excess of that of Europe, but in the matter of quality they frequently excel our standards.

Pure water from the standpoint of the chemist is not to be found in nature; neither is it desirable that such should be furnished for general purposes, for the presence of certain salts in water makes it more palatable and better for use than distilled water. The origin of all water-supplies is primarily to be traced to the rainfall, although it by no means follows that the supply utilized in any particular region is derived from the precipitation in that immediate locality.

As has previously been shown, the rainfall is either evaporated from the surface of the earth, runs off, or percolates into the ground. Only that which remains on the surface or in the soil is of any avail as a source for water-supplies. Between the surface "run-off" and that which flows beneath the surface there is a constant interchange which exerts its effect on the quality of the water.

156. Changes in Quality Determined by Course of Water.—As it condenses in the atmosphere and falls to the earth's surface, it begins to absorb impurities; and its whole history from the time it is precipitated until it finally finds its way back into the air through evaporation is marked by the absorption of substances which pass into solution or are held in suspension, as well as the precipitation or elimination of the same or other ingredients. Some of these changes are harmless so far as affecting the ordinary use to which water is put; others are of much consequence, depending upon the requirements to which the water-supply is subjected.

157. Requirements as to Quality.—The ordinary purposes to which water-supplies for human use are put may be included under the following heads: potable, domestic, and manufacturing uses. So far as quality is concerned, the conditions desired for each purpose do not necessarily coincide.

158. Potableness.—A suitable supply for drinking purposes should not only be pleasant and palatable, but if possible free from any marked color or turbidity. While these latter requirements are desirable, they are not obligatory, for experience has fully demonstrated that many peaty supplies and often turbid waters may be used with perfect safety. A potable water should not be excessively charged with mineral matter in solution. Mineral waters have a value for medicinal purposes, but not as general supplies. It has long been a disputed question as to the effect on human health of waters heavily loaded with dissolved mineral matter. A common prejudice exists against the use of very hard waters, as they are supposed to result in the production of various diseases, as urinary calculi, goitre, cretinism, etc., but there are no established scientific data that would positively confirm such an opinion. It is more likely that intestinal and gastric disturbances may occur where permanently hard waters are used.

It sometimes happens that water may dissolve poisonous metals either in the soil, or in pipes used for distributing purposes, and so become unwholesome. Lead, zinc, and iron are the metals most likely to occur under such circumstances. Where water is acid, as in peaty waters, or where CO_2 is present in large quantities, the solvent action on the lead is much increased. In such districts many cases of lead poisoning not infrequently occur, although all waters of this class are not necessarily affected. Zinc is much less liable to cause trouble, although where water is in contact with galvanized pipes an appreciable amount of zinc may often be determined. When iron is present in waters it generally comes from the source of supply, and is not derived from the pipes except under certain circumstances. Where present in the proportion of 0.5–1.0 part per million, the water generally has an objectionable taste, and while the presence of this metal in small quantities is not attended with serious results on health, its undesirable taste and appearance is against its use.

The danger of direct absorption of poisons from water is, however, small compared with that attributable to the influence of disease organisms. Typhoid fever and other diseases of an intestinal character not infrequently find their way into water-supplies, often causing widespread epidemics of these infectious maladies; but this subject is of such importance as to require more detailed treatment later (Chapter X).

159. Domestic Use.—For ordinary domestic use the quality of water must be such that it can be used in cooking and for laundry use. For these purposes water should not contain too large a proportion of mineral ingredients. Naturally, the same sanitary requirements that are necessary in drinking-water also obtain in water used for culinary purposes. Excessively hard waters are not desirable, as the flavor of many foods is considerably impaired when cooked in the same. Iron-bearing waters are also unsuitable for this purpose, as the tannin in tea and many vegetables produces a black precipitate. These waters are likewise detrimental for laundry use, as the oxidation of the ferrous salts upon exposure to the air produces rust-spots upon clothes. The greatest difficulty to contend with in the laundry in the case of ground-waters is the presence of soluble salts of alkaline earths, such as lime and magnesia. When soaps are added to such "hard" waters, insoluble precipitates are produced, and it is therefore necessary to use a much larger quantity of soap before a lather can be produced. One part of lime carbonate requires about eight of soap, so the problem from an economic standpoint is one of importance. It is estimated that the city of Glasgow saves $180,000 annually in the amount of soap used since the introduction of the soft Loch Katrine water.*

The hardness of water is either temporary or permanent, depending upon the chemical nature of the dissolved salts. If bicarbonates are present, the CO_2 contained in the same is set free by boiling, in which case a white precipitate consisting of carbonate of lime is formed. As this reduces the amount of lime in solution, the hardness is diminished, and such is therefore called temporary, in contradistinction to the sulfates and chlorides of lime and salts of magnesia that are not so affected; hence waters containing these salts in abundance are permanently hard.

160. Manufacturing Purposes.—For different manufacturing purposes, such as brewing, sugar-making, dyeing, etc., the quality of water is subject to considerable variation. In the production of steam, trouble is experienced with all waters containing an excess of salts of the alkaline earths by the formation of boiler-scale. With the temporarily hard waters a friable deposit is produced, while permanently hard waters cause a much more compact "scale," that is very difficult

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* Parkes, Hygiene and Public Health, p. 10.

to remove. The accumulation of even a thin layer of boiler incrustation involves a very marked loss of energy in the coal used.

161. Distribution of Bacteria in Soil.—To understand aright the quality of waters as affected by germ-life in the same, it is necessary to know the distribution of bacteria in the soil. Generally the rock-masses are covered with a layer of more or less finely ground material that makes up the soil. This layer is differentiated into two strata: the upper one, the soil proper, that is darker in color and of a more porous texture; the lower, known as the subsoil, that simply represents the unchanged rock débris. As the soil supports abundant vegetable and animal life which, as it dies, is resolved into decomposing organic matter, the upper layer becomes enriched through the accumulation of the same which serves as future plant-food. The organized tissues are first disintegrated by the action of the saprophytic bacteria, as noted in the changes of putrefaction and decay. In this process humus is formed and the looser texture and darker color of the upper soil-layer are attributable to this series of changes. As these processes are controlled by bacteria, it is not surprising to find that the uppermost soil-layers are teeming with myriads of these forms, often millions of them being present in every gram. This number, however, diminishes rapidly below the surface, and at the depth of two or three yards soils are practically sterile. Cultivated, but more particularly inhabited soils, have a higher bacterial content than virgin forests or prairie soils. The character and texture of the soil-layers also influence to some extent the distribution of these micro-organisms. The reasons for this peculiar distribution lie in the filtering power of the soil-particles as the moisture percolates downward; in the absence of organic food-supplies in the deeper layers; and to generally unfavorable growth conditions (lower temperature, diminished oxygen).

Not only does the soil harbor all kinds of bacteria associated with the breaking down of organic matter, but the formation of nitrates from the ammonia so produced, due to the nitrifying bacteria, also occurs in the upper layers of the soil.

It is at once evident that the germ content of any water that comes in contact with the soil must be profoundly affected by the soil-layers. So, too, with the air; for in a dried condition, the fine dust-like particles with their adherent organisms are readily raised by wind-currents from the surface. Of course by far the majority of these organisms are harmless saprophytes; but if disease matter is deposited upon the surface of the soil, there is often nothing to prevent the distribution of pathogenic bacteria in quite the same way.

While the germ content of water may be greatly influenced by that of the soil, it must be remembered that the flora of the two habitats are not necessarily similar. There are to be found in water certain species that are so universally present that they may be called water bacteria. In this medium they are able to grow with the greatest ease, even in waters that are relatively poor in inorganic as well as organic nourishment.

METEORIC WATERS.

162. Absorption of Impurities from Air.—Meteoric waters include the various forms of rainfall, as rain, snow, hail, dew, etc.; and while they are not normally to be considered as immediate or direct sources of supply, except occasionally for individual use, yet the fact that they serve as indirect sources from which supplies are subsequently drawn make it desirable to consider their quality. As watery vapor condenses in the air and is precipitated, whether in the form of rain or snow, it absorbs impurities from the atmosphere. Particles of dust and dirt are washed out of the same, and in the neighborhood of large towns, soot and other combustion products may pollute these waters to a very considerable degree. The gases that are naturally present in the air are also more or less readily absorbed by water. Not only is this true with the more important constituents, N, O, and CO_2, but such substances as ammonia, sulfuric acid, nitrous and nitric acids are generally found. Naturally meteoric waters are deficient in mineral matter and hence are soft. Where extremely soft their action on lead pipes is severe.

Water in falling to the ground in the form of either rain or snow also takes up germ-life from the air. Bacteria and spores of fungi find their way into it in considerable numbers from the subjacent soil-layers, and while they are incapable of multiplication in the air, yet in a dried condition many forms can retain their vitality for long periods of time. The result is that either rain or snow catches these floating forms of life and so they are carried down. Even in hailstones they are to be found without exception.

Hill * records some observations made on the germ content of the air at different periods during a rain-storm. At first it was very high, 5495–5759 bacteria per c.c., but after a rain of 12 hours there were only 15–57 germs. The number of organisms in the air decreases rapidly with an increase in altitude. On mountain-sides this is not so marked until the snow-line is reached.

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* Public Water-supplies, p. 51.

SURFACE-WATERS.

163. **Character Determined by Nature of Underlying Soil.** — As meteoric waters fall to the earth, a portion of same is evaporated into the air, while the remainder, following the contour of the surface, either runs off or percolates into the underlying soil. That which is apparent on the surface of the soil is included under the term surface-waters, although an appreciable part of such supplies has been subject to more or less percolation through soil layers (springs, ground-water drainage into rivers). Owing to the fact that surface-waters may be brought into contact with mineral and organic matter that renders them more or less impure, the quality of such waters is subject to much fluctuation. When first precipitated, the character of the water is to a large extent determined by the nature of the soil over which it flows and the vegetable covering of the soil, but in river systems that traverse a wide range of territory, the suspended and dissolved impurities are not necessarily related to the character of the land drained.

164. **Surface-Waters as Potable Supplies.** — As organic refuse, either of human, animal, or vegetable origin, is generally found on the surface of soil, it is evident that the quality of surface-waters is often impaired by reason of pollution with such material. Inasmuch as the most dangerous refuse of this character is that connected with human existence, it follows, where pollution is at all possible, that the density of population will exercise a potent influence on the character of such supplies. For this reason the use of surface-waters, particularly those of flowing streams in densely populated watersheds, is a menace to public health, unless they are first subjected to some adequate method of artificial purification. In this respect lake-waters, particularly such enormous reservoirs as our Great Lakes, are naturally of much better quality than running waters that carry off the surface-wash and drainage of large land-areas. This is confirmed by the typhoid death-rates of cities using this water in comparison with those furnished with river supplies. In 1890–96 the typhoid deaths for the five largest cities situated on the Great Lakes averaged 42 per 100,000, while for the five largest river cities of the United States it was 58 per 100,000.

A. *Flowing Waters.*

165. Naturally the availability of running streams as sources of municipal water-supply has led to their more frequent adoption than any other kind of surface-water, but it must be remembered that this is not because they are of better quality. Water in flowing over the sur-

face of the soil naturally acquires numerous impurities that it would not take up if it remained quiescent. These substances are of inorganic and organic origin, each class being represented by suspended as well as dissolved material. One character of flowing waters is the sudden change in composition that is liable to occur at almost any time, but more particularly during flood seasons. By reason of this, a supply that is generally satisfactory may be rendered undesirable in a short space of time. Again, not only are running streams the natural drainage-channels of a region, but they must to a large extent also serve as sewage-outlets for urban populations that are generally increasing in density.

166. Physical Appearance.—Owing to the fact that running waters frequently have a considerable fall per mile, thereby producing more or less rapid currents, it naturally follows that waters of this class in direct contact with the soil erode their drainage-basins with considerable rapidity and so become more or less turbid.

Babb * has compiled the observations made on various rivers as to the visible load of sediment. They are as follows:

	Amount of Sediment by Weight.
Potomac	1 : 3575
Mississippi	1 : 1500
Rio Grande	1 : 291
Danube	1 : 2880
Nile	1 : 2050

Not only the amount but the nature of this silt varies much in different streams, depending mainly on the character of the soil and the rate of flow. Some streams, like the Missouri, may possess a very turbid water, but the size of suspended particles is such that much of this sediment is deposited when the water is quiescent; other streams, like the Mississippi and Ohio, carry a smaller load of silt, but the fine colloidal clay that is so abundant in the same makes it exceedingly difficult to clarify.

Surface-waters flowing through swampy regions are usually colored, due mainly to the extraction of soluble coloring matter from vegetable material. Such peaty waters, while perhaps unsightly in appearance, may be, however, perfectly wholesome in spite of this physical defect.

While flowing surface-waters do not dissolve so much mineral matter as ground-waters, yet by virtue of their dissolved carbon dioxide they also take up an appreciable amount, depending considerably on the character of the soil stratum over which they pass. The average

* Babb, *Science*, 1893, XXI. p. 343.

of a large number of European and American rivers shows about 180 parts per million of soluble solids, of which nearly one-half is carbonate of lime.*

167. Bacterial Condition of Flowing Streams.—Naturally the close contact with the upper soil-layers, which are so rich in micro-organisms, accounts for the much higher germ content of rivers and streams than lakes. It might be thought that such streams would show a larger number of bacteria duing a low stage of water than otherwise, for if the sources of pollution were at all uniform, the lessened summer flow would tend to concentrate the impurities. As a fact, however, the bacterial pollution of a stream is always greater during high-water stages. The more rapid rate of flow increases the carrying power of the stream, and much more suspended matter, as silt and dirt, is borne along together with the bacteria that invariably accompany such disturbance of the soil-particles. Theobald Smith † found that the Potomac River water contained the following number of bacteria at different seasons:

Dec.	Jan.	Feb.	March.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.
967	3774	2536	1210	1521	1064	348	255	254	178	75	116

The same general result has been noted by Fuller ‡ in his studies on the Ohio River at Cincinnati and Louisville, and by Frankland § in the case of the Thames and Lea, two rivers from which London derives in part its water-supply.

According to Johnston ‖ the bacterial content of uninhabited streams like the Saguenay in Canada is not materially different from that of rivers flowing through farming regions, although where a stream flows through a city or town of any considerable size, especially if it receives the sewage of the same, the amount of pollution is naturally much increased. Prausnitz ¶ determined the following data for the Isar River at Munich.

TABLE NO. 23.

BACTERIAL CONTENT OF ISAR RIVER.

	No. of Bacteria per c.c.
Above the city of Munich	531
150 feet above sewer outfall	1,339
Directly opposite sewer outfall	121,861
450 feet below sewer outfall	33,459
Ismaning (8 miles below sewer outfall)	9,111
Freising (20 " " " "	2,378

* I. C. Russell, Rivers of North America, p. 79.

† *Med. News*, April 9, 1887.

‡ Investigations on Purification of Ohio River, p. 34.

§ Micro-organisms in Water, p. 91.

‖ *Hyg. Rund.*, v. p. 796.

¶ Der Einfluss d. Münch. Canalisation, 1889.

Not only is there a marked increase in the bacterial content of the river, but it is also evident from the above table that a large part of this pollution is lost in a comparatively short time, as it only takes 8 hours for the current to reach Freising, 20 miles below. These conditions have since been reinvestigated (1898),* and it has been found that over 50 per cent of the bacteria introduced in the sewage are eliminated in a flow of twelve miles.

168. Self-purification of Rivers.—This process of spontaneous purification is to be noted in all streams that are polluted in any way by the introduction of sewage or soil drainage. Not only are organic impurities but inorganic as well eliminated in this way. The rate at which this process goes on depends upon a number of conditions, such as rate of flow, character of bed and shores, amount of sediment carried in water, etc.

Comparative studies (chemical and biological) have been made on a number of important streams on which cities are situated. Naturally most of the data yet collected are on European waters.

Stutzer and Knublauch † found an evident purification of the Rhine below Cologne in 2 miles' flow. Six miles below the bacterial content on the left shore was reduced to one-third. On the right shore the diminution was less rapid, as a tributary brought into the stream a large amount of factory waste from other towns. This could be traced for a distance of 16 miles below before it disappeared.

Heider ‡ traced the pollution of the Danube below Vienna for 25 miles, a distance covered in a flow of 8–9 hours. In this stream the sewage of the city was diluted from 225–880 times. Schlatter § in 1889 observed the effect of the sewage of Zurich for 6 miles, and recently Thomann, ‖ in investigating the same problem ten years later to determine if the zone of pollution had been materially increased, found only one case at the distance of 9 miles where the germ content was approximately as low as it was above the city. At this distance the average germ content was about 50 per cent higher than before the introduction of the sewage. During this period the city had increased its population about 50 per cent, so the zone of pollution was proportionally increased. The same result was observed at Munich in studying the Isar, as is indicated in Table No. 24.¶

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* *Hyg. Rund.*, 1898, VIII. p. 161.

† *Cent. f. allg. Gesundheitspflege*, 1893, abs. in *Hyg. Rund.*, IV. p. 225.

‡ Oest. Sanitätswesen, 1893, No. 31.

§ *Zeit. f. Hyg.*, IX. p. 56.

‖ *Ibid.*, XXXIII. p. 1.

¶ *Hyg. Rund.*, 1898, VIII. p. 161.

TABLE NO. 24.

BACTERIAL CONTENT OF RIVER ISAR BELOW MUNICH.

Year.	Name of Observer.	Number of Bacteria per c.c.			
		Oberführing, 3 Miles.	Ismaning, 8 Miles.	Freising, 21 Miles.	Landshut, 45 Miles.
1889	Prausnitz	....	6,824	3,608	
1890	"	3,140	2,960	1,510	910
1893	G., L., P., N.*	24,100	15,065	7,134	1,976
1895-6	Deichstetter, Willemar	....	14,185	7,893	2,900

A few American streams have been more or less perfectly studied in this regard. In the sanitary survey made in Ohio of the more important streams, the same general relations were noted. Bleile † found in every case a marked difference between the bacterial content of water above and below the various cities, but this marked increase always dropped again to normal in the course of a flow of a number of miles, providing there was no new source of pollution. In a general way the bacteriological fluctuations correspond to the variation in the free and albuminoid ammonia, but in some cases increase in ammonias was due to influx of vegetable impurities, in which instance of course the bacterial increase was not as marked as after addition of organic matter of animal origin.

The sanitary survey made on the Mohawk and Hudson rivers, New York, show a similar relation. The numerical increase in bacteria in river-water occasioned by the introduction of the sewage of Albany remained evident for a distance of about 11 miles below the city.‡

By far the most extensive study that has yet been made on American streams is that carried on by Jordan § on the Illinois River in connection with the Chicago Drainage Canal. The waters of this stream were studied chemically and bacteriologically both before and after the opening of the Sanitary Canal, in order to determine whether the introduction of the sewage of the city of Chicago would exert any deleterious influence on the quality of the St. Louis water-supply drawn from the Mississippi. The appended data show the purification observed in Illinois River under normal conditions.

* Goldschmidt, Luxemburger, Frans, Hans u. Ludwig, Neumeyer, Prausnitz.

† Examination of Sources of Ohio Public Water-supplies, p. 137.

‡ Rept. N. Y. State Board of Health, 1892, p. 526.

§ Jordan, Bacterial Self-Purification of Streams, *Jo. Expt. Med.* v.: 271, 1900.

TABLE NO. 25.

CHLORINE AND BACTERIAL DETERMINATIONS MADE ON WATER IN ILLINOIS RIVER AND ITS TRIBUTARIES UNDER AUSPICES OF CHICAGO SANITARY DRAINAGE COMMISSION.

Collecting Stations	Distance from Bridgeport in Miles.	Chlorine, Parts per 1,000,000.	Bacteria per c.c.	Number of Analyses Made.
Bridgeport	0	119.2	1,245,000	19
Lockport	29	117.4	650,000	30
Desplaines River at Lockport	....	7.9	9,180	28
Joliet	33	104.8	486,000	28
Kankakee River at Wilmington	....	3.4	5,000	28
Morris	57	68.1	439,000	26
Ottawa	81	58.5	27,400	26
Fox River at Ottawa	....	5.0	6,510	29
Big Vermillion at La Salle	....	61.2	7,970	30
La Salle	95	46.1	16,300	31
Henry	123	44.2	11,200	29
Averyville	159	40.9	3,660	30
Wesley City	165	40.1	758,000	22
Pekin	175	38.4	492,600	29
Havana	199	36.2	16,800	26
Sangamon River at Chandlerville	....	4.5	5,080	21
Beardstown	231	29.3	14,000	26
Kempsville	288	22.9	4,800	19
Grafton	318	18.3	10,200	28
Mississippi River at Grafton	....	2.8	7,600	29

The extent of natural purification of the Illinois River can be observed from the above table. The steady diminution in the amount of chlorine is noteworthy all the way from Bridgeport, where a large proportion of the sewage of Chicago is present, to Grafton, where the Illinois joins the Mississippi. The bacterial reduction is also continuous for a distance of over 160 miles, until the river receives at Wesley City the large amount of refuse from Peoria. It is to be noted that this large additional load of pollution does not increase the chlorine so much as it does the bacteria, but this is probably due to the fact that the sewage contains a very large amount of manufacturing wastes (distillery and glucose refuse).

The table also includes the tests made on tributary streams, and it is strikingly noticeable that in no case but one is the chlorine content of such a nature as to add materially to the pollution of the main river.

169. Causes of Self-purification of Streams.—The explanation of the cause of this phenomenon is so complex that no single principle can be cited that will apply to all cases. The different factors that are operative under changing conditions may be grouped under two heads:

(1) Factors concerned in apparent purification, as dilution and sedimentation.

(2) Factors concerned in actual destruction of bacteria, as sunlight, vital concurrence, unsuitable food-supply.

Polluted waters may have their germ content reduced per unit of volume by the first class of factors without necessarily destroying the bacteria associated with the polluting material.

170. **Dilution.**—In a purely mechanical manner, polluted material is greatly diluted when discharged into a running stream. This dilution varies greatly with the varying amount of sewage discharged and the stage of water in the stream. In rapidly flowing streams this factor is more potent than in sluggish rivers. Although a stream may not receive any material additions by way of tributaries, yet the volume of water in a river is constantly being augmented by the influx of ground-water that drains into the drainage-channels from the surrounding land, and so the extent of dilution is being gradually increased.

171. **Sedimentation.**—Removal of bacteria by sedimentation may occur in two ways. There may be a gradual settling of the organisms themselves by virtue of their specific gravity, or they may be entangled and carried down by the subsidence of suspended particles of silt. The latter method is by far the most effective, and in streams is the only way in which sedimentation exerts any influence. Subsidence of suspended matter begins to occur whenever the current is lessened, due either to expansion of stream or diminished fall per mile. The Spree below Berlin illustrates the influence of diminished flow.* From 190,000 bacteria per cc. found in the river as it emptied into the Havel, an expansion of the stream 7 miles broad, the number fell to 9000 as it issued from this natural sedimentation basin.

A peculiar case of sedimentation has been noted by Van't Hoff, † and is utilized in securing the water-supply of Rotterdam from the Maas (Rhine). This city is on tide-water, and at flood-tide the checking of the current as it meets the sea is so marked that the bacterial content of the river is lessened about 50 per cent. During this period of partial subsidence the necessary supply is largely secured.

In removing the bacteria from a flowing stream by sedimentation, the organisms are not necessarily destroyed. They may be carried to the bottom by the precipitation of the inorganic matter and in the slimy ooze of the river-bed find conditions more or less suitable for de-

* Frank, *Zeit. f. Hyg.*, III. p. 355.

† *Cent. f. Bakt.*, XVIII. p. 265.

velopment. No data have yet been collected on this phase of the subject, but Russell found in studying the bacterial flora of the sea-bottom (Atlantic and Mediterranean) * that the germ content was much greater than that of the water, and to a considerable extent was made up of species not found in the water above. This would indicate that the high content of the mud is not entirely due to sedimentation.

172. **Sunlight.**—Direct sunlight has a potent germicidal effect on many bacteria, and Buchner † has ascribed a prominent part to this factor in explaining the phenomena of self-purification of waters. Experimental work has conclusively demonstrated that the germicidal effect is caused by the chemical and not the heat rays of the spectrum. Not only do the direct rays of sunlight destroy the bacteria, but even diffused light in some cases exerts a prejudicial influence.

Care must, however, be taken in interpreting these data, which have been secured for the most part in experiments carried on in various culture media; for it has been determined that such media in the presence of direct sunlight and air may decompose, and antiseptic substances as peroxide of hydrogen, be formed.

Some observers, however, as Frankland, ‡ have carried on their investigations in natural waters as well as in culture media; and have found, for instance, that anthrax spores are for the most part quickly killed in such waters, although other species retain their vitality for months; but they are destroyed less rapidly in water than in culture media.

The data collected as to the depth to which this disinfecting action of the light is effective are very contradictory. Buchner § found that the germicidal influence of the light was very marked when cultures were submerged at the depth of 4 to 5 feet, and demonstrable with typhoid in agar at 10 feet; but Arloing, ‖ Frankland, ¶ and Procacci ** have all found that an appreciable depth of water (a few inches to a foot or so) materially diminished the disinfecting action. The action is probably considerably less in rivers than in lakes owing to the increased turbidity of flowing streams.

Buchner's †† observations on the increase in bacteria in lake waters

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* *Zeit. f. Hyg.*, XI. p. 165.

† *Cent. f. Bakt.*, 1892, XII. p. 217.

‡ Proc. Roy. Soc., 1893, LIII. p. 316.

§ *Cent. f. Bakt.*, 1892, XI. p. 781; also XII. p. 217.

‖ *Arch. de Physiol.*, 1886, VII. p. 209.

¶ Proc. Roy. Soc., 1893, LIII. p. 204.

** *Annali d. Inst. d'Igiene Sper. di Roma*, 1893, III. p. 437.

†† Tiemann-Gartner, Das Wasser, p. 579.

during the night when compared with observations made at sundown are sometimes cited as confirmatory evidence of this disinfecting action, but there are too many disturbing factors that might enter in to mask the real effect to rely entirely on observations to prove this point. Indeed the observations by Prausnitz and others on the same river (Isar) showed that while frequently a marked decrease was noted in sunny days, they also observed the same on days in which the sky was completely overcast.

From the data already at hand it seems quite clear that the disinfecting action of direct light has been considerably overestimated. While it is unquestionably operative to some extent, it plays at most only a subordinate rôle.

173. **Vital Concurrence.** — Water contains so many other living forms than bacteria that it would be surprising if there were not a strong competition between the various forms of life represented in the same. Different observers have ascribed to green plant-forms (water-weeds, algæ, diatoms, etc.) a purifying power, but the evidence as to their effect in a polluted stream is far from conclusive. It is true that these chlorophyll-bearing organisms do not subsist directly on organic matter, and in some cases, as Schenck has noted, where polluted streams are readily purified, organisms of this class are not at all abundant; hence their purifying action is by no means satisfactorily proven.

The distinctively dangerous disease organism in water, i.e., the typhoid bacillus, is apparently affected by the presence of other bacterial forms in abundance. Jordan, Russell and Zeit * have shown that the typhoid disappears much more rapidly in a polluted than an unpolluted water, and Russell and Fuller † have determined that this disappearance is closely associated with intimate contact with sewage forms of bacteria. Whether this is due to by-products toxic to the disease organism or not is difficult to prove, but Frost has shown a distinct antagonism between the typhoid organism and several saprophytic forms.

This same condition is doubtless true with reference to the disappearance of *B. Coli* in flowing streams. Weston noted at New Orleans the nearly complete disappearance of *B. Coli* in the water of the Mississippi River, although heavily charged with silt and extensively polluted. For a considerable distance above the city, no surface pollution is added owing to the level system. Consequently spontaneous purification of polluted water became operative.

174. **Unsuitable Food-supply.** — The sewage bacteria, and to some

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* *Jour. Inf. Diseases*, 1904, I., p. 641. † Ibid., Sup. No. 2, Feb., 1906.

extent the soil organisms, do not find favorable conditions for rapid growth in ordinary waters. This is evident from the numerous experiments that have been made to determine the viability of such organisms as the typhoid, cholera, and colon forms (222). When these alone are added to water or in competition with other forms, they rapidly diminish in numbers. Still the evidence of pollution sometimes disappears in a flow of 6 to 8 hours, and in such cases it could hardly be due to their having been killed. In cases of retarded purification, as the Seine in France, where pollution is still recognizable after two to four days' flow, this factor might be more effective.

175. Aeration.—It is a popular belief that aeration greatly improves the character of water, but numerous experiments on the effect of oxygen and motion, singly and in conjunction with each other, fail to show any material effect. Leeds failed to find any difference in Niagara water above and below the falls. The experiments by Mills on artificial aeration also show but little effect.

176. Chemical Reaction.—Certain chemical combinations may take place in water that will tend to purify the same. The Schuylkill above Philadelphia is heavily charged with iron, salts, and acids (due to mine-drainage), but in flowing over a limestone reigon the acids in the water neutralize the lime salts, precipitating much of the lime and iron, making a soft and wholesome water from what was originally unfit for use.

177. Conclusion.—That flowing streams polluted or contaminated in any way do undergo a spontaneous purification there can be no question. The factors that have been considered above probably account for the most of such change, although the effect of each operative factor varies in different cases owing to the change in conditions.

Naturally no hard-and-fast rule can be given that will apply to all conditions, but the most definite conclusions that can be drawn from the data already at hand indicate that sedimentation and dilution play the more important rôle in the purification of waters. Undoubtedly sunlight and the action of other living forms are also operative to some extent, but the results already obtained lead to the belief that these are only of subordinate influence, especially in the case of streams.

The important problems for the engineer are: How soon does this purification take place? Can streams once polluted be used again with safety?

From available data it seems evident that a stream once polluted with any considerable amount of sewage is unsafe to use for a water-supply so long as there is any trace whatever of pollution remaining. It is impossible to set a distance limit, or even a time limit of flow (although this would be less objectionable), for such limits would vary much in each instance. It has been claimed in England that no stream is sufficiently purified by the time it reaches the sea to warrant its use, and it is well established that typhoid epidemics have been distributed for scores of miles down-stream. Just how long disease bacteria can retain their vitality in water has long been a disputed matter, but as long as a stream shows any evidence of pollution it certainly should be regarded as dangerous.

B. *Impounded Surface-waters* (*Lakes*, *Ponds*, *Reservoirs*).

178. The waters of an open expanse, such as a lake, are less likely to show marked pollution than flowing streams, because in relatively quiescent waters, solid matter, excepting the finest clays, cannot long remain in suspension and the factor of land contamination is of less prominence. In large bodies of water, as the Great Lakes, the effect of pollution is limited to shore regions, but under certain conditions may be considerably extended, as is to be noted along the south shore of Lake Superior, where the water is frequently rendered densely turbid for a distance of 6–10 miles from shore because of a stratum of tenacious red clay along the coast-line.

At certain seasons of the year, the water-supplies of towns along this shore, relying on lake-water, are greatly impaired. The following data collected by the writer at Duluth-Superior show the germ content of the polluted shore-line in comparison with the crystal-clear lake-water.

TABLE NO. 26.

NUMBER OF BACTERIA PER C.C. IN LAKE SUPERIOR AT DULUTH-SUPERIOR.

Depth at which Sample was secured.	Distance from Land at which Sample was secured.				
	Shore.	1¼ Miles.	5 Miles.	5 Miles.	10 Miles.
Surface	2457*	87	....	....	23
45 feet	....	....	....	44	....
54 "	....	16	....	50	....
60 "	....	....	11	....	5
80 "	....	....	20	....	....
90 "	....	....	....	....	6
Depth of water at different stations	3 feet	60 feet	80 feet	90 feet	100 feet
Appearance of water at surface	very turbid	cloudy	turbid	faintly turbid	clear

* Average of 14 different samples.

179. Vertical Circulation in Lakes.*—Owing to the fact that the maximum density of water is somewhat above the freezing-point (39.2° F., 4° C.), water in lakes is more or less subject to vertical currents that cause the upper and lower layers to mix under certain temperature conditions. In large lakes of the temperate type there is generally no circulation of the water, as the heavier cold water rests on the bottom. In smaller, shallower lakes there are periods of stagnation, in which there is no vertical circulation. These occur in winter and summer. Between these periods there is an "overturning," i.e., a vertical circulation due to temperature changes. In the spring, as the surface warms above the freezing-point, the water increases in density and therefore becomes heavier. This causes it to sink, thus producing vertical currents. In the fall the surface cools and the water is apt to be stirred by wind action until the warmer, lighter bottom water is forced to rise as the colder surface-water sinks.

In shallow lakes, small reservoirs, etc., the circulation of the water is going on at all times, except while the surface is frozen; but where reservoirs are 20 feet deep or so, the phenomenon of stagnation may at times occur.

This is a matter of some importance, as the temperature of a supply is affected by these changes. Moreover, if water is drawn from a low level in the reservoir, it may be derived from layers that have been stagnant for considerable periods of time.

180. Bacterial Content of Open Surface-waters.—The improvement in physical appearance of lake-waters in comparison with rivers reflects itself at once in the biological and chemical character of the same. Generally speaking, waters of this class contain far less bacteria than do running streams. While of course there is no marked uniformity in numbers, yet it is rare that waters of this type contain more than a few hundred organisms or at most more than a few thousand bacteria per c.c.; and these for the most part are harmless water saprophytes.

These organisms are more or less uniformly distributed throughout the entire mass of the water; but according to Nicholson's† studies made on Lake Mendota under the writer's direction, the lower strata are considerably richer in germ-life than the intermediate layers. The surface frequently contains more organisms due to the effect of dust.

In summer this bacterial distribution is apt to be obscured through the action of wind, light, variation in temperature, etc., but in winter, when the water is covered with a mantle of ice and these disturbing

* See Chapter V in Whipple's Microscopy of Drinking-water.

† Thesis, Univ. of Wis., 1900.

conditions are more or less thoroughly eliminated, this zonary distribution is rendered more apparent. In the mud or slime that collects on the bottom of lakes and ponds, the bacterial content is greatly increased.

Where surface-waters sustain a copious growth of algæ, as is very frequently the case, the bacterial content of the water during this state may be rendered abnormal through the development of organisms living on the organic matter that is derived from the death of the vegetable organisms.

181. **Natural Purification Processes.**—The marked diminution in germ content of lake-water as distance from shore increases indicates that the natural purification of quiescent surface-waters is also as marked as is that of flowing streams. Except in the case of inflow of streams of considerable size, the evidence of land-pollution does not extend far. The reason for this is, in the main, dilution and sedimentation. The disappearance of perceptible currents causes suspended matter to settle quickly, thereby reducing greatly the germ content. These organisms may be able to retain their vitality in the ooze for some time, but the larger proportion found in the lake mud are forms that have evidently developed in this habitat (171).

Lortet* even claims to have found a number of pathogenic bacteria at the depth of 120–150 feet in the mud of Lake Geneva, Switzerland. The ooze formed from the deposition of sediment in water gradually becomes more and more compacted, and, owing to the formation of ferrous sulfide, a black gelatinous precipitate is produced that cements the particles into a semi-solid sticky mass. Fuller has noted the formation of this material in the artificial subsidence reservoirs at Cincinnati.†

Direct sunlight is undoubtedly effective as a factor in purifying waters of this class, for quiescent waters are as a rule clearer, and therefore the actinic rays would be able to penetrate more deeply than in turbid flowing waters.

182. **Influence of Vegetation.**—The quality of surface-waters is sometimes affected by the copious development of vegetable life. This is particularly apt to occur in relatively shallow lakes where the growth of "water-weeds," as *Myriophyllum*, *Chara*, *Vallisneria*, *Ranunculis*, etc., may be so rank as to accumulate organic matter in large quantities. While these plant-forms have no direct relation to disease production, yet the decay of this vegetable material may seriously affect the quality of such water.

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* *Cent. f. Bakt.*, 1891, IX. p. 709.

† Rept. on Cincinnati Water Purification, p. 120.

183. Odors in Water-supplies.—It used to be thought that the presence of any appreciable odor in water was due entirely to the natural processes of decay, but in addition to these it is now known that a number of living organisms, both plant and animal, give off odors during their development, due to the presence of oils formed in the cells. Oils of different sorts can be detected by the sense of taste in extremely dilute solutions. According to Whipple* the odor of peppermint can be noted in water containing 1 part of oil to 50,000,000 parts of water; clove-oil, 1 to 8,000,000; cod-liver oil, 1 to 1,000,000. This explains why the odor from a relatively small number of some of these odoriferous organisms is so manifest. There are a number of plant and animal forms that appear so frequently in ponds and reservoirs of water-supplies as to warrant specific mention. Besides these there is a much larger number of other species that occur less frequently.

Of the aromatic odors formed, that produced by the diatom, *Asterionella*, is perhaps the strongest.† Where only a few of these organisms are present the odor is aromatic; where more abundant it recalls a geranium odor; and where very numerous a distinct fishy odor is apparent. Whipple has found by experiment that 50,000 cells of *Asterionella* would produce enough oil so that the dilution was only 1 : 2,000,000, an amount that is quite within the range of detection. Other diatoms are not infrequently found, but their odoriferous properties are less pronounced.

Grassy odors are caused mainly by the blue-green algæ, the *Cyanophyceæ*. The most distinctive member of this group is *Anabæna*. When abundant, the water has a taste resembling green corn. Vegetable odors are caused by the diatoms, *Synedra* and *Melosira*. With the former, 5000 cells per c.c. suffice to produce a distinct odor. Whipple has often found in the Brooklyn supply as many as 15,000–20,000 of these organisms per c.c.

Of all defects of this class in water, fishy odors are the most objectionable. One of the animal forms belonging to the protozoa, *Uroglena*, produces an odor resembling cod-liver oil; while another, *Synura*, recalls the odor of ripe cucumbers. Troubles of this character have appeared several times in the Boston water-supply. At first they were ascribed to *Spongilla*, the fresh-water sponge, but later they were traced to *Synura*, which was found to be developing in large numbers in Lake Cochituate under the ice.

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* Microscopy of Drinking-waters, p. 123.

† l. c., p. 125.

It might naturally be thought that these troubles, being due in the main to vegetable growth, would be more apt to prevail during the summer months than at other seasons, but such is not necessarily the case.* Of the algæ, *Anabæna* is apt to occur most frequently from July to September. *Pediastrum*, *Raphidium*, *Scenedesmus*, *Closterium*, and *Staurastrum* are most numerous in July and August, but the diatoms, as *Asterionella*, *Melosira*, *Synedra*, and *Tabellaria*, are often more abundant in early spring or late fall than at other seasons. The protozoan forms (*Dinobryon*, *Peridinium*, and *Trachelomonas*) occur most commonly in March, July, and August.

Troubles from bad or unpleasant odors in water-supplies are very much more apt to occur in open surface-waters than any other; hence impounded supplies may develop these abnormal conditions at times. Of 71 supplies examined by the Massachusetts Board of Health, a bad taste was noted in 45, and of these two-thirds gave serious trouble.

The introduction of the copper sulfate treatment has done much to make it possible to overcome the production of undesirable odors due to algæ, but this method should be used with caution. For further discussion see Art. 569.

C. *Ice Supplies.*

184. Influence of Freezing on Bacterial Life.—The quality of ice is dependent primarily upon the character of the water before it is frozen. It is true that some of the grosser solid impurities are expelled from water, especially if congelation takes place gradually, but it does not follow that ice made from polluted water is safe for human use.

Not only does the examination of ice show that it is generally poorer in germ-life than the subjacent water beneath, but experimental tests on the resistance of bacteria to freezing indicate that many forms and more particularly disease species are capable of retaining their vitality for many months.

Prudden,† Sedgwick,‡ and Park § have found that the typhoid bacillus is capable of retaining its vitality for at least three months when frozen, although there was a rapid diminution in number of organisms immediately after freezing.

In the process of freezing from 60–90 per cent of the contained organisms are killed, although many vegetative as well as spore-bear-

* Parker, Mass. Bd. Health, Exam. of Waters, 1890, p. 597.
† *Med. Rec.*, March 26, 1887.
‡ *Science*, March 23, 1900.
§ *Journ. of Boston Soc. Med. Sc.*, April, 1900.

ing forms are able to resist freezing for a while at least. While from experimental work it has been definitely shown that typhoid and other pathogenic organisms are able to retain their vitality for long periods of time when frozen, still there is no authenticated case in which typhoid epidemics have been traced to the use of impure ice, although intestinal disturbances are known to have been caused in this way.*

SUBTERRANEAN WATERS.

185. **Change in Quality Due to Percolation.**—That portion of the rainfall that finds its way into the soil is rapidly changed in quality by percolation through the various soil-layers. As it flows through the soil toward the ground-water level it loses the larger portion of the impurities derived from the air and the soil surface, but at the same time it absorbs other substances from the layers through which it passes, so that in general, the quality of subterranean waters is materially different from those of surface origin.

To some extent the gaseous content of rain-water is changed as it courses through the soil. The particles of suspended matter (soot, dust, and germ-life) that are absorbed from the air, together with the organic matter and bacteria derived from the upper soil-layers, are readily removed in the percolation process, so that at a depth of a few yards at most the germ-life of the surface of the soil and all its attendant impurities have been eliminated.

On the other hand the percolating water dissolves certain inorganic elements, and especially by virtue of the CO_2, which it has absorbed from the air, this solvent action is greatly increased.† In this way the salts of lime and magnesia are rendered soluble, making hard waters, while other mineral elements, such as the silicates, are also carried more readily into solution. This action increases materially the total solids of a water, more particularly those of an inorganic character. Subterranean waters therefore carry a load of soluble solids, while the solids of surface-waters are more largely in suspension. In regions rich in humus the ground-water may contain a large amount of organic as well as inorganic constituents.

Not infrequently such waters may also contain ferrous salts. In the presence of humus and absence of oxygen, the sulfates may be reduced to hydrogen sulfide, and the nitrogen compounds to ammonia. These iron-containing ground-waters are of not infrequent occurrence;

* Sedgwick, *Science*, March 23, 1900.

† Pure water dissolves about 1 part in 10,800 of carbonate of lime, while the same saturated with carbon dioxide is able to render soluble about 1 to 1000.

and in many cases they are otherwise desirable, but the presence of the iron impairs the quality of a supply for drinking and domestic use, not so much on hygienic grounds as because of its physical appearance. Moreover, in such waters, the so-called iron fungus, *Crenothrix polyspora*, is very apt to become established, in which case the iron is oxidized from a ferrous to a ferric form. Inasmuch as this organism does not require light for its growth, it is able to grow in covered reservoirs and pipes.

186. **Purification of Water in the Soil.**—By the operation of natural processes in the soil, water is purified in passing from the surface to the ground-water level. The forces concerned in this change are physical, chemical, and biological. The larger part of the suspended matter is removed by filtration in a purely mechanical manner. There is also an attraction for substances in solution, as is evident from the fact that the color of water due to dissolved matter is removed in part at least by percolation through soil.

Chemical changes may also be caused by the action of one substance or another, precipitating or dissolving the same, but the most effective transformations are those that are induced by biological causes, viz., the micro-organisms present in the soil-layers. In the upper layers, organic matter, vegetable or animal, undergoes fermentation, putrefaction, or decay, with the result that the nitrogenous organic substances are gradually converted into soluble condition, generally ammonia compounds. The carbonaceous elements are changed into carbon dioxide, water, and organic acids.

When material containing nitrogen has been converted into ammonia compounds, it is then acted on by the nitrifying bacteria, forming first nitrites and then nitrates.

These mineralizing processes, which are really oxidation changes, take place more rapidly in the superficial layers of the soil, where oxygen is more abundant. Temperature and character of the soil also exert an influence on rate of change.

In swampy regions containing a large amount of humus, and therefore so acid as to inhibit the development of the nitrate-producing bacteria, the nitrogenous material accumulates as ammonia products rather than as nitrates.

187. **Capacity of Soil for Purification.**—The purifying action of the soil is not unlimited, and under certain artificial conditions largely ceases to be operative. Naturally, the action of any pollution is intermittent, the offensive material being discharged on the surface at intervals, between which the natural purifying forces are operative. This

condition is essential to adequate purification. Under artificial conditions occasioned by man's presence, this intermittent action may be suspended. If, therefore, sewage is discharged continuously on to the surface of the soil, even though in small amounts, the action of the natural purifying processes is disturbed, and the result is that the soil becomes saturated with organic matter which is not converted into the harmless substances that would naturally be produced as a result of the operation of soil processes. It is in this way that the soil of thickly populated areas like cities loses its property of spontaneous purification, often to such an extent that the ground-water is rendered impure. Under such conditions, while the organisms of disease may be held back by the soil layers, the soluble products of organic decay are able to percolate into the ground, thus making it especially difficult to determine the wholesomeness of such water where reliance is placed on chemical examination alone.

188. Extent of Filtration Necessary to Effect Purification.—The distance through which the water must pass before it is sufficiently purified for potable purposes is a question of very considerable importance. Judging from the higher typhoid mortality rates of populations using shallow wells in comparison with those utilizing a supply from a deeper source, it is evident that efficient purification is often not reached in shallow wells. This may of course be due in some cases to direct pollution. Not infrequently it may happen, where the ground-water is subject to considerable oscillations in level, that at high stages this generally sterile water layer comes in contact with soil that is not bacteria-free. This condition might possibly arise in cities, especially where the land has been filled in, and where decomposing organic matter is some distance below the soil surface.

The depth necessary to insure efficient purification will also vary with the filtering power of the soil. Loose, sandy, or gravelly soil having larger pore-spaces will permit of more ready filtration than compact clay loams. Pfuhl * has determined, by adding a culture of some easily demonstrable organism like *B. prodigiosus* to water in a well, that there can be a lateral movement into the ground for 10 feet or more.

Again there is to be mentioned the possibility of direct rifts or channels existing in the soil or rock. Holes made by animals (rats and larger rodents), earthworms, crustacea, etc., frequently permit of direct passage of unfiltered water to considerable depths. A number of cases of wells and springs have been recorded where the germ content was so high and of such a character as to leave no doubt but that

* Pfuhl, *Zeit. f. Hyg.*, xxv. p. 549.

there was a direct connection with the surface.* Generally this condition is more likely to prevail along faulting cracks in rock layers than in soil, or in limestone regions where subterranean channels have been dissolved by the water. The classical case of the Lausanne, Switzerland, epidemic,† where the village well was infected from a polluted brook over a mile distant, but which had an underground connection with the well, is a striking illustration of the unreliability of natural purification through soil layers. Gaffky ‡ showed that the Wittenberg typhoid epidemic in 1882 was due to infection of an open well from vaults 50 feet distant. The stratum in this case was gravel.

Thoinot and Brouardel § traced a typhoid epidemic in Havre to pollution through 80 feet of chalk to a clay substratum where the water appeared as a spring. Such cases happily, however, are exceptional. In general, ground-water supplies are the most reliable of any. For individual use and for small municipalities, they will always remain the principal source of supply, and their use could undoubtedly be extended in some cases to larger cities.

189. Spring-waters.—In the popular mind springs are supposed to represent the purest of supplies, but under certain circumstances this type of ground-water may not be wholly pure. They are produced by percolating rain-water flowing along an impervious stratum until it finds an outcrop to the surface. Often in mountainous districts the depth and thoroughness of percolation over and through rock masses is so limited that the water may not equal in purity the normal ground-water. Generally, spring-waters before exposure to surface of soil are relatively deficient in micro-organisms, as they represent filtered waters, but as they appear at the surface, the water comes again in contact with organic matter and soil bacteria, and may thus receive a considerable quota of organisms from this source, although generally the germ content of unpolluted springs is below 100–200 per c.c. ‖

While spring-water usually has a low initial bacterial content, the organisms contained in such waters possess the property of very rapid multiplication during storage. According to Miquel ¶ this rapid but transitory power of development characterizes the bacteria of spring-waters in contradistinction to the slower and more persistent growth that occurs in impure waters.

* Tiemann-Gärtner, Das Wasser, p. 523.

† *Deutsch. Arch. f. Klin. Med.*, 1893, Bd. II.

‡ Gaffky, Mitt. a. d. Kais. Gesundheitsamte, 1884, II. p. 413.

§ Thoinot, Bacteriology, p. 62; also *Ann. Past.*, 1889, III. p. 145.

‖ Tiemann-Gärtner, Das Wasser, p. 492.

¶ Manuel pratique d'Analyse bact. d. Eaux, 1891, p. 146.

190. **Well-waters.**—It not infrequently happens that there may be several impervious geological layers superimposed on each other that serve to collect the water from different areas. Under such circumstances the upper stratum will retain the local ground-water of the region, while the more copious supply beneath is the result of percolation from a larger and perhaps distant source. Shallow wells often strike only the surface ground-water, which is sometimes of poor quality, while the water of deep wells which tap the larger, more universal supply in the rock is usually more thoroughly purified. Shallow wells dug in the soil and walled up dry are often to be found in the more crowded portions of cities. Generally these are sunk in soil that is more or less thoroughly impregnated with organic refuse, so that the water in the same is often in a polluted condition, not having been purified by its passage through a shallow, and at the same time contaminated, soil stratum.

Then, again, wells of this character practically serve as drainage-basins for the thickly populated areas above them, and when walled up dry, seepage from the soil is carried directly into the same. The influence of near-by closets and vaults is thus not infrequently to be observed. Cesspools are particularly dangerous, because they contain so much water which must find its way into the soil by percolation. Just how far wells of this character should be placed with reference to vaults and cesspools depends upon the character of the soil and the contour of the surface. In shallow wells where the ground-water layer may be lowered through pumping, the zone of influence may be considerably widened. Pfuhl places the average distance at 100 feet, but it is evident that no exact limit can be drawn. Where the ground-water lies near the surface the distance should be manifestly increased to a maximum limit. Not infrequently open wells of this class may be polluted directly from the surface, unless graded up so as to carry off the local drainage. Wells of such character frequently serve as disseminators of water-borne diseases. Their condition is at once betrayed by a chemical or bacterial test. They contain large numbers of bacteria, and the presence of gas-producing, indol-forming organisms at once indicates their impure condition. Such wells generally have a high chlorine content, as this element continues to increase with the growth of population, and the presence of nitrogen in nitrous and nitric forms and considerable quantity of the ammonias is further proof of pollution with organic refuse.

The better class of wells that are sunk into the permanent ground-water are either drilled or driven. In these the sides of the wells are made impervious to seepage by iron casing, so, that barring pollution from direct surface-drainage at the top, the only ordinary chance for contamination is in tapping an impure ground-water.

191. Bacterial Content of Wells.—While the ground-water is presumably free from bacteria, or at least very nearly so, water as it is taken from wells almost always has an appreciable germ content. In the case of shallow, dug wells where opportunity for infection from above or seepage from sides is present, and where the temperature of the considerable mass of water is such as to permit more rapid bacterial growth, it is not at all uncommon to find thousands of organisms per c.c. The infiltration of organic matter aids materially in the development of this germ life.

In the better type of wells, drilled or driven, the germ content is subject to wide variation. Normally where all opportunity for external pollution is excluded, the number of bacteria per c.c. is very small. Frankland found in some of the deep wells in the chalk in England only six bacteria per c.c., but often where the most careful precautions are taken in securing samples a much larger number is to be found. Sedgwick and Prescott* found the following numbers in a series of deep wells in Massachusetts examined by them:

	No. of Bacteria per c.c.
Well 100 feet deep	30
Well 193 " "	269, 254
Well 213 " "	101, 106
Well 254 " "	150, 135
Well 377 " "	48, 54
Well 454 " "	205, 214

These waters were characterized by the absence of liquefying bacteria and the abundance of pigment-forming species. Similar results have also frequently been recorded by others. The following summary from Tiemann-Gärtner's book on Water† gives an idea of average conditions.

TABLE NO. 27.

SUMMARY OF OBSERVATIONS ON BACTERIAL CONTENT OF WELLS.

Locality Observed.	Observer.	No. per c.c.
Mayence	Egger	34 wells in 64 had less than 100 53 " " " " " " 500
Stettin	Link	27 " " 47 " " " "
Steinberg	Rastall	9 " " 10 " " " "
Kaiserslautern	Bokorny	59 " " 78 " " " 100
Leitmeritz	Maschek	12 " " 59 " " " 500
Gotha	Becker	34 " " 53 " " " 100 50 " " 53 " " " 500
Kiel	Fischer	51 " " 179 " " " "
Höchst	Grandhomme	108 " " 118 " " " "

* 26 Rept. Mass. Bd. of Health, 1894, p. 435.

† Das Wasser, p. 489.

It is evident from the above that while the average conditions in well-water do not show a high germ content, yet even good wells often contain a considerable number of bacteria.

Of course it at times happens that even deep wells may receive ground-water that has not been wholly purified. Pfuhl * cites an instance where pollution occurred after passing through 180 feet of gravel, but these cases must be exceptional. From whence do these organisms then come? In all probability infection occurs at the time of digging the well. The machinery used in the digging is far from being bacteriologically clean, and in this way the water is seeded from the beginning. Many of the species are able to grow even in pure water, and the result is that some development occurs, so that various forms persist in the water. The following observations made by Hastings and the writer on newly drilled wells where chance for contamination was absolutely excluded point to this conclusion:

WELL NO. 1 DRILLED 110 FEET IN DRIFT (GRAVEL, SAND, ETC.), JULY, 1899.

Number of bacteria per c.c.	July 24, 1899	10,080
	Oct. 6, 1899	2,050
	Dec. 14 1899	380
	March 29, 1900	18

WELL NO. 2 DRILLED 60 FEET IN SAME FORMATION, NOV., 1899.

Number of bacteria per c.c.	Dec. 14, 1899	11,450
	March 29, 1900	570

Both of these wells contained an enormous number of liquefying bacteria at the beginning, but non-liquefying species predominated later.

Fränkel † has also demonstrated that this infection occurs from without by disinfecting a well with a mixture of carbolic and sulfuric acid; then by removing the chemicals by long-continued pumping. Wells so treated remained germ-free for 6–7 days, but ultimately became invaded from above.

192. Effect of Pumping.—Bacterial growth can go on at surprisingly low temperatures; and in deep wells in the ground where the water is in the neighborhood of 48–50° F., the conditions are such that multiplication of germ-life readily occurs. When a considerable volume of water is present in the well, the distribution of bacterial life throughout the same can readily occur. Under such circumstances the number of organisms per c.c. in the water can often be greatly reduced by pumping out the standing water and allowing fresh quantities of germ-free ground-water to percolate into the reservoir. Even with long-con-

* *Arch. f. öffentl. Gesundheitspflege in Elsass-Loth.*, 1895, XVI. Heft 2.

† *Zeit. f. Hyg.*, VI. p. 23.

tinued pumping it is practically impossible to remove all bacteria adherent to the sides of well and pump. Maschek records an instance where 31,500 gallons of water were pumped from a well in 12 hours, and the germ content was reduced from 2750 to 1064; in another case it fell from 458 to 68 when 1600 gallons were removed.

Gruber * carried on uninterrupted pumping experiments on a well for several days with the following results:

Beginning of test	several thousand per c.c.
8 days' pumping	472 "
14 " "	22 "
75 " "	10 "

193. Effect of Organic Nutriment on Growth of Water Bacteria.—The ability of bacteria to thrive in well-waters depends in large degree on the amount of nutriment they find in such a habitat. Rubner † added to a well a small quantity of meat extract and then determined its effect on germ-life. A quantity that was only able to increase the amount of oxygen consumed by 1–2 mg. caused the germ content to rise from 10,000 to 50,000 and finally to 170,000, at approximately which point it remained for some weeks. This indicates that if a well is so poorly constructed as to permit of the perçolation of soluble organic matter, the conditions are such as would favor growth of organisms in the same.

194. Artesian Wells.—In flowing wells where the flow is always outward it would be difficult to imagine how infection from outside might occur, and yet a bacteriological examination of such waters not infrequently reveals the fact that they may contain some bacteria, although generally much less than ordinary wells. Such a condition would naturally seem at variance with the idea that the ground-water is practically sterile; but when one recalls that in a number of these deep subterranean supplies crustacea and small fish ‡ have been found in some cases, it is evident that a deep supply does not necessarily mean that the water has really been filtered through a very deep layer of soil or rock. Russell found at Dubuque, Ia., in several artesian wells over 1500 ft. deep from 30–90 organisms in one, and 300–400 in another. Several analyses were made of these waters at different times with corroborative results.

It often happens that water from deep wells and springs contains

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* *Deutsche Vierteljahrschr. f. öffentl. Gesundheitspflege*, XXV. p. 415.

† *Arch. f. Hyg.*, XI. p. 365.

‡ In the artesian wells, 180 feet deep, situated at Biskra in the Sahara desert, mollusks and small fish are found at times.

nitrites in quantities that would be sufficient to condemn a water if it was from a shallow well or a surface-water. This condition may, however, have no significance, as it may be brought about by a reduction by various causes of the nitrates in the deeper layers of the soil.

EFFECT OF STORAGE AND DISTRIBUTION ON QUALITY.

195. **Improvement of Water by Storage.**—In most cases in municipal supplies it is necessary to store the water in reservoirs of varying size, so as to provide against contingencies. Under such conditions the water is sometimes subject to changes, some of which improve while others impair the quality of the supply.

The changes that result in an improved condition occur generally with waters of surface origin rather than with spring- or ground-waters. In the storage of waters of this class sedimentation is effective in eliminating much of the suspended matter, including living organisms, as well as a portion of the dissolved organic matter. Where considerable mineral matter is in suspension, as in many rivers, especially during flood seasons, the degree of purification by subsidence is even greater than where the suspended solids are less. St. Louis derives its supply from the Missouri River, which at some seasons of the year may contain nearly 2 per cent by volume of suspended solids. Nearly 95 per cent of this is precipitated in the storage-reservoirs during 24 hours, with the result that the germ content is greatly reduced.

The factors operative in the spontaneous purification of lakes are also of value in the changes induced by storage in reservoirs.

The color of waters, especially when due to organic matter, is lessened by storage, although this bleaching action of the sun's rays does not extend rapidly to any great depth.

196. **Impairment of Water by Storage.**—Surface-waters, however, may be impaired in quality by storage under certain conditions. A marked effect is apt to arise from the stagnation of the water. Under certain temperature conditions, the water in large reservoirs during quiescent periods * does not circulate vertically and therefore the lower layers become stagnant. If the bottom of such reservoirs contains considerable organic matter, as is generally the case where water is impounded in artificially made lakes, then the dissolved oxygen is rapidly exhausted, causing the death of all organisms incapable of leading an anaerobic existence. Such waters frequently acquire bad odors. The following observations by Whipple † show this variation.

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* This is especially liable to occur when reservoirs are covered with ice. (Drown, 24 Mass. Bd. Health, 1892, p. 333.)

† Whipple. Microscopy of Drinking-water, p. 137.

TABLE NO. 28.

DISSOLVED OXYGEN IN LAKE COCHITUATE, MASS.

	Per cent of Saturation. Aug. 16, '91.	Sept. 28, '91
Surface	79	90
10 feet deep	84	81
20 " "	30	33
40 " "	20	8
50 " "	0	0

If the organic matter in the upper soil layers is removed before impounding these surface-waters, this difficulty does not occur, a condition that generally obtains in lakes that have a gravelly or sandy bottom.

Ground-waters and those which have been purified by artificial filtration are not improved by storage in open reservoirs. In fact waters that have been thus purified by filtration through soil-layers and the germ content thereby greatly reduced are much more liable to deteriorate than grow better by storage.* A supply that is drawn from both ground and surface sources, as in the Brooklyn supply, is much more apt to give trouble than a pure ground-water, as the admixture of surface-water will generally seed the water with living organisms, which are able to develop rapidly in such waters.

When purified waters are allowed to stand, the bacteria are able to develop prolifically; and while this development has no special significance from a sanitary standpoint, because these organisms are generally the normal water bacteria, still it does not improve the water in any way.

Ground-waters, owing to their passage through the soil, contain considerable soluble mineral matter, and therefore such waters are well adapted to the development of some kinds of plant-life. Whipple † thinks this is less likely to happen in a new reservoir than in one which has been long in use. The accumulation of organic sediment on the bottom of the reservoir is very apt to facilitate the development of this type of microscopic life (183), of which the diatom, *Asterionella*, is perhaps the most undesirable representative on account of the marked odor that it produces. Waters of surface origin that have been filtered act in this respect like a ground-supply.

This development of algæ can be prevented by covering the reservoirs, as direct sunlight is necessary for the multiplication of these green

* Water-supply of Brookline, Mass., 19 Rept. Mass. Bd. Health, p. 89.

† Microscopy of Drinking Water, p. 141.

plant-forms * or by the application of the copper sulfate treatment (569). In large reservoirs this latter method is naturally most feasible, but it should be used with circumspection. The death of certain species often permits of the growth of other forms.

In covered reservoirs, however, certain ground-waters may also be affected. Fungi, bacteria, and animal forms, living as they do on organic material, do not need sunlight. Hence they might find in covered reservoirs a favorable habitat, but as animals generally live on algæ, their presence is determined by this fact. The most important organism of this class is *Crenothrix*, the iron bacterium, which often grows so luxuriantly in waters containing iron and organic matter as frequently to clog the service-pipes by the accumulation of vegetable growth. Sometimes this water-pest flourishes to such an extent as to necessitate a change in base of supply, as in Berlin, or the introduction of a method of treatment that will eliminate the iron before the water flows into the distributing-mains.

197. Effect of Distribution on Quality. — In distributing-mains changes in character of water may also occur. The temperature varies considerably during passage through pipes, and this has some effect on the living organisms of the same. The action of water on the pipes is considerable, especially if derived from a supply that is poor in lime and magnesia salts. Unless protected, the pipes are liable to rust and the so-called iron "tubercles" form on the inner surface. In "dead ends," owing to the stagnation in current, the water may acquire a distinct chalybeate taste and appear unsightly from flakes of iron-rust. This condition is much aggravated if the water itself contains iron in solution, in which case the iron bacteria (*Crenothrix*) are able to thrive. Certain kinds of waters, as those rich in CO_2 or organic acids, may exert a solvent action on service-pipes if they are made of lead to such an extent as to produce lead poisoning. This trouble occurs most frequently in connection with peaty waters.

Most other microscopic organisms are reduced in number in the distributing-pipes. If the source of supply is from reservoirs or surface bodies, it is apt to contain algæ which are unable to live in darkness. Such organisms therefore die and decay rapidly in the pipes, and if sufficiently numerous undesirable odors may be imparted to the water, besides furnishing food for bacteria. Many organisms are deposited by sedimentation, particularly in pipes on an "up-grade" or in high

* Sometimes a limited light through the roof of the reservoir cover will permit certain species, as *Chlorococcus*, *Asterionella*, *Melosira*, and *Synedra*, to develop, as was the case at Dedham, Mass., where the supply was drawn from a covered well.

buildings. Animal forms, living as they do on organic matter, are able to grow under such conditions, and in waters supplied from surface sources it is not uncommon to find the inner walls of the mains covered with a considerable layer of "pipe-moss" which may be made up of sponges, *Protozoa*, and *Bryozoa*. Ground-waters are not so likely to be troubled.

Changes occur in the bacterial content of water during distribution, but sometimes they are increased and again they may be diminished. There does not appear to be any well-defined law regarding their action.

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CHAPTER X.

COMMUNICABLE DISEASES AND WATER-SUPPLIES.

198. Relation of Water-supplies to Disease Dissemination.—The key-note of sanitary science, so far as applied to the investigation of water problems, is to be noted in the relation that exists between communicable or transmissible diseases and public water-supplies. That disease-producing germs may find their way into the human body through water and so possibly cause outbreaks of different maladies has been known from time immemorial. The ancient Romans appreciated this when they spent so much time and labor to bring their water-supplies through their magnificent aqueducts from beyond the reach of pollution.

The most important question to be considered in connection with any water-supply is: (1) whether it is wholly free from the possibility of distributing disease; (2) whether it is likely to remain in such a condition. These are questions of much more importance than economy in securing or distributing water, and should therefore first engage the attention of the sanitary engineer. Of the various maladies that are transmissible from person to person, only a limited number are likely to be distributed through the medium of water. These are known as water-borne diseases in contradistinction to those that are disseminated through the air or find an entrance by means of wounds.

199. The Germ Theory of Disease.—The germ theory of communicable diseases is now so definitely established that it is unnecessary to present proof in detail that the various maladies of this class are caused by the introduction of living organisms from outside of the body. In connection with this theory, two schools have arisen, one holding to the idea that each disease has a specific cause, an organism which alone is responsible for the occurrence of the pathological condition. The other adheres to the hypothesis that the production of a diseased state requires more than the introduction of the germ associated with the malady. According to this school the organism must find its way into a susceptible soil, under conditions which favor the production of the diseased state.

It is at once evident that these theories have a direct bearing upon questions relating to sanitary engineering. If the introduction of the specific germ of cholera is all that is necessary to provoke an attack of that disease, it is more than ever necessary that all cholera organisms should be prevented from finding their way into waters used as public supplies.

200. Specific Nature of Water-borne Disease Germs. — In the case of most water-borne diseases, it is quite generally admitted that the causal organism is more or less sharply differentiated from other bacteria. In one case, i.e., typhoid fever, the specific nature of the organism is not defined with so much certainty. *Bacillus typhosus*, the typhoid fever bacillus, is closely related to the common intestinal organism, *Bacillus coli communis;* and by some it is held that these are merely two varieties of the same germ.* The preponderance of evidence, however, is generally believed to be in favor of the specific nature of the two organisms; yet from the engineer's point of view it does not matter much, for any drinking-water that contains evident traces of intestinal discharges certainly should not be regarded as a safe supply, even if the possibility of human wastes finding their way into the same be wholly excluded.

201. Diseases Due to Parasitic Intestinal Worms. — Whether fecal matter from distinctively animal sources should be permitted to pollute drinking-water is a somewhat different question, for the diseases incident to man that are most frequently spread by means of the water-supply do not normally occur among animals, yet the possibility exists that larvæ and eggs of parasitic worms may find their way into water through discharges of animals. In a number of cases these parasites are common to both man and some of the lower animals, and hence the danger from this source is evident. Among the more common parasitic worms that affect man are the pork tape-worm (*Tænia solium*), the round worm (*Ascaris lumbricoides*), the thread-worm (*Oxyuris vermicularis*), and the worm causing pernicious anæmia (*Anchylostomum duodenale*). These worms, while affecting the human species, also find lodgment in some of the lower animals. In such cases their intestinal contents may contain eggs which may thus find their way into waters through pollution of the same with animal refuse, but in the aggregate the danger from this source is small.

* Hueppe, Prin. of Bact., English trans., p. 193.

INFECTIOUS DISEASES TRANSMISSIBLE BY WATER-SUPPLIES.

202. **Conditions Necessary for Infection.**—The danger of a water-supply serving as a vehicle for the transmission of disease rests (1) on the possibility of such organisms finding their way into potable supplies, and (2) on the ability of the bacteria so introduced to grow in such waters, or at least retain their vitality for sufficient periods of time to permit of infection.

If, under ordinary conditions, water is not a medium in which a pathogenic organism is able to live, then there is practically no danger of spreading such disease in this way. On the other hand, if the specific microbes are able to grow, or even to live for a considerable period, in waters that normally are used as public supplies, and these forms are also liable to be introduced into the same, then the danger from this source is well worth consideration.

But even though a disease germ may be able to live in water, it does not necessarily follow that danger to human beings exists on account of this. Not a few of the disease bacteria that are able to retain their vitality in water when placed under experimental conditions would not under normal circumstances find their way into supplies.

203. **Water-borne Diseases affect Intestinal Canal.**—Only those diseases that are able to establish themselves in the alimentary canal are at all likely to be transmitted in this way. This might include those that affect the throat, as diphtheria or scarlet fever, but, generally speaking, the danger is confined to those diseases that establish themselves in the intestines.

In some cases, as in typhoid fever, a disease can enter only through a single organ or kind of tissue, as the intestine in this instance; in other cases, as in anthrax or tuberculosis, the specific cause establishes itself in the body in several different ways. But it must be kept in mind that there are numerous human diseases that do not obtain a foothold in the body through the water that is drunk. These may therefore be practically neglected by the sanitary engineer in his work.

204. **The Most Important Water-borne Diseases.**—The most important diseases to consider in this connection are typhoid fever and cholera. These are the distinctively water-borne diseases; and while there are others that should be mentioned, yet practically the question of pollution with specific disease bacteria is confined to a discussion of the relation that these two maladies have to public water-supplies. Of these two, in this country under normal conditions, cholera is of much less importance, as it is distinctively an Oriental disease, whose

natural home is in India. Now and then, on account of the close intercommunication between Europe and the Orient, and the laxity at times of quarantine regulations, cholera breaks over its natural boundaries and devastates regions widely remote from its native home. Here in America the danger from the disease is much lessened, unless a widespread epidemic should break out in Europe.

Typhoid fever, on the other hand, is a disease that is naturally endemic to America as well as other countries, i.e., it occurs continually with more or less regularity. Neither of these two diseases is contagious in the strict sense of that term, i.e., contracted by mere contact with an affected individual. The germ causing the same does not travel of itself through the air as in the case of smallpox or scarlet fever, but it must be introduced into the susceptible organ, the intestine, through the medium of either the water which is drunk or the food which is eaten. That these diseases play such an important rôle in human affairs is a striking commentary on our hygienic methods of the present day. In caste-ridden India, where civilization has hardly yet emerged from the murky darkness of superstition, perhaps it is excusable that cholera should remain endemic; but among the civilized nations of Europe and America it is indeed humiliating to admit that such an easily preventable disease as typhoid fever is so thoroughly entrenched.

In addition to these two principal diseases that are very easily spread by means of polluted water, dysentery and diarrhœic disturbances should also be mentioned as traceable to a similar origin; but these troubles are often so imperfectly defined that they are not with certainty associated with any definite specific organism.

205. **Typhoid Fever.**—This disease is essentially an intestinal disease, the organism of which finds in the small intestine, especially in the lymph-glands of this organ, the most favorable location for development. The disease organism, *Bacillus typhosus*, multiplies rapidly in the intestine, and is evacuated in the dejecta and sometimes in the urine as well.* Carelessness in the disposition of these discharges may result in surface-waters becoming polluted with the same. This danger from feces has long been known, but it is only recently that the danger from infected urine has been thoroughly appreciated. Well-waters, particularly those that are from open and relatively shallow wells, are also liable to become infected.

* Gwyn (Johns Hopkins Hosp. Bull., June 1899) states that from 20 to 30 per cent of all typhoid cases show this condition. A most serious factor in this connection is their persistence for months in such large numbers after convalescence. Petruschky found as high as 170,000,000 typhoid organisms per cubic centimeter in the urine of a patient.

The disease organism is introduced into the body through the food and drink. Most frequently it gains entrance by means of polluted water, but quite often milk and solid food may also function as carriers of contagion. Even in milk it is often originally introduced from contaminated water that may have been used to rinse or wash the milk-utensils, as in the very severe Stamford, Conn., outbreak in 1893, where 386 cases of the disease developed in a period of a few weeks. Nearly all of these were on the route of a single milkman; and it was further shown that infection of the milk was caused by rinsing out the cans with *cold* water from an infected well, after they had been well scalded.

Within recent years it has been abundantly demonstrated that flies and insects also often function in distributing the disease by infecting food. To such a cause and not to polluted water-supplies were largely attributable many of the outbreaks in our military camps during the Spanish-American war.

The period of incubation, i.e., the time between infection and the appearance of the disease in the affected person, is somewhat variable, ranging from nine days to three weeks, the symptoms becoming characteristic in most cases in about two weeks. This long period of incubation must be taken into consideration in searching for the origin of infection. A person acquiring the disease through polluted water would therefore not show any evidence of the same for some time, and it is on this account easy to overlook the real source of infection. Not infrequently sporadic cases may be acquired in other cities and so disseminated by travelers (Fig. 26). Then, again, frequently the disease is rather light in character, so that the affected person is not confined to the house. The disease is often spread unwittingly by these "ambulating" or "walking" typhoid cases.

By many it is believed that putrid or offensive gases emanating from sewage or any other foul source predisposes the system to this disease. On the basis of this belief it is generally regarded that sewer-gas is very dangerous. Some experimental results obtained by Alessi* seem to indicate that such a view might be true, but this is contradicted by Abbott,† whose experimental investigations seem to indicate that such gases do not affect the health of animals. The mortality of laborers in the sewer systems of large cities or in connection with sewage-disposal plants does not sustain the view that inhalation of air over sewage is especially dangerous.

* *Cent. f. Bakt.*, 1894, xv. p. 228.

† Trans. of Cong. of Amer. Phys. and Sur., 1894, pp. 28–55.

The mortality-rate in typhoid fever varies considerably in different outbreaks, ranging from a few to over 20 per cent, and averaging on the whole about 10 per cent of the case-rate.

Although the disease at present is much more wide-spread than necessary (owing to our failure to regard hygienic measures that would limit its distribution), still it is diminishing rapidly in amount as is indicated by the data compiled from the Massachusetts vital statistics.

TABLE NO. 29.

DECREASE IN TYPHOID FEVER IN MASSACHUSETTS FROM 1871–1897.

	Typhoid Death Rate per 10,000 Population.	Percentage of Typhoid Deaths to Total Mortality.
1871–1875	8.2	
1876–1880	4.6	
1881–1885	5.0	2.7
1886–1890	4.16	
1891–1895	3.25	1.62
1896	2.77	1.46
1897	2.37	1.42

In the five large cities in the United States the percentage of typhoid deaths to total mortality has ranged as follows from 1870–95, inclusive:

TABLE NO. 30.

PERCENTAGE OF TYPHOID DEATHS TO TOTAL MORTALITY IN FIVE AMERICAN CITIES.

	Average for 25 yrs.	Lowest.	Highest.
New York	1.12	0.7	1.8
Brooklyn	0.75	0.6	1.1
Boston	1.93	1.2	3.0
Philadelphia	2.99	1.4	4.0
Chicago	3.66	1.08	7.2

206. Typhoid Fever and Sewage Pollution.—The history of almost every large city has been that with the growth in population and consequent increase in sewage the amount of typhoid fever has steadily increased. This is particularly striking in those cities that are situated on river systems or large bodies of water where surface-waters serve the dual purpose of public water-supply and sewage disposal.

On river systems, particularly in the more thickly populated regions of this country and Europe, cities frequently draw their public supplies from running streams that may have been polluted by the sewage of towns above them. With the natural growth in population the zone of pollution in the stream is constantly widening, and hence supplies from rivers which at one period might have been satisfactory are continually being endangered.

The increase in typhoid fever in such cases is generally gradual, but at intervals an especially severe outbreak in any one town will often be reflected in other outbreaks in towns situated lower down the river. This synchronous development of the disease proves the fact beyond dispute that the rise and fall of typhoid is often closely related to pollution of municipal supplies from sewage.

207. Mohawk Valley Epidemic.—A most instructive case of this simultaneous development of disease due to sewage pollution is seen

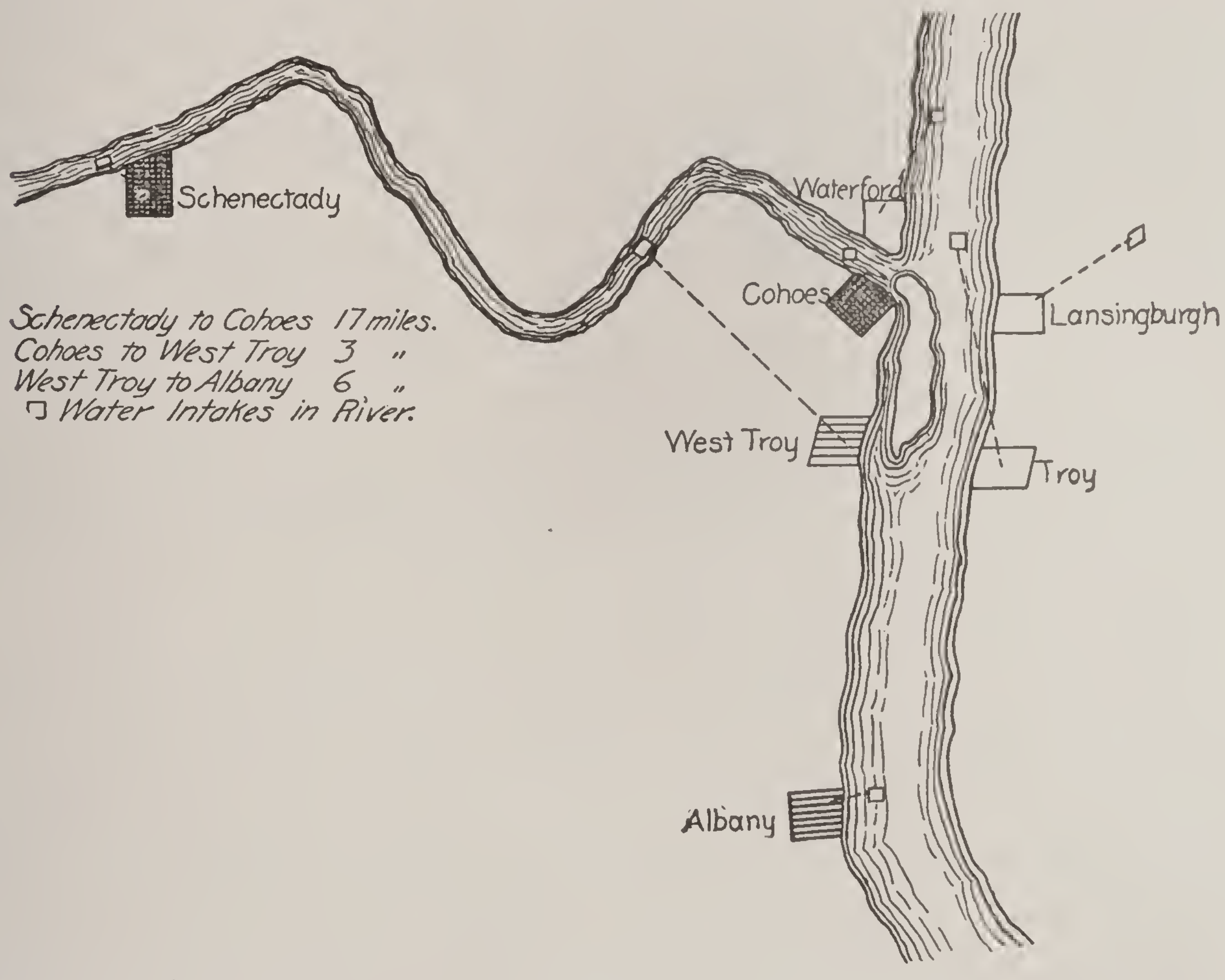

FIG. 25.—DISTRIBUTION OF TYPHOID FEVER IN MOHAWK-HUDSON VALLEY. (Adapted from Mason.)
Typhoid epidemics shaded (relative intensity of outbreak denoted by shading).

in the series of typhoid epidemics that occurred in the towns in the valley of the Mohawk and Hudson rivers in 1890–91.

In July, 1890, typhoid became epidemic in Schenectady and continued until April, 1891. Seventeen miles down the Mohawk is Cohoes, a city of about 22,000. Typhoid broke out here in October, 1890, and before April, 1891, there had been 1000 cases. The disease was exceptionally mild; but notwithstanding this the typhoid death-rate for the period of the epidemic was equal to an annual death-rate of 45 per 10,000 inhabitants, or about 12–15 times the normal.

West Troy, taking its supply also from the Mohawk above Cohoes (see map), suffered from an epidemic from November, 1890, till February, 1891, except for a brief period when the supply of the village of Green Island was used. Six miles below West Troy is Albany. Here again the disease became epidemic in December, lasting through the spring. Waterford, Lansingburgh, and Troy took their supply from other sources than the Mohawk, or the Hudson below the junction with the former stream. So far as this outbreak was concerned they escaped entirely.

The progressive development of the disease in all of those towns that used water from the Mohawk, and its absence in other towns situated on the Hudson that were supplied from other sources, shows conclusively the influence which the sewage pollution of Schenectady and other upper towns had on the distribution of the disease.

208. **Lowell-Lawrence Epidemic.** — A similar development was also noted in the case of the towns of Lowell and Lawrence on the Merrimack River in Massachusetts. In 1890–91 an especially severe outbreak of typhoid occurred in Lowell which was traced to the water-supply. The source of supply was the river-water, and Sedgwick showed that the probable origin of the polluted condition was attributable to several cases of the disease at North Chelmsford, a small village situated three miles above, on a tributary of the Merrimack. These cases occurred in August, September, and October.

The sewage of Lowell empties into the Merrimack, and after 8 hours' flow the river-water is utilized by the city of Lawrence, 9 miles below. Water polluted by the sewage of Lowell might thus reach Lawrence the same day. It would take several days (7–10) to pass through the supply-reservoir before it found its way into the service-pipes. From an inspection of Table 31 the direct relation between the outbreak in Lawrence and the polluted river-water derived from Lowell is evident.

TABLE NO. 31.

RELATION OF TYPHOID OUTBREAK IN LOWELL AND LAWRENCE.

Deaths from Typhoid in	Lowell.	Lawrence.
September, 1890	8	3
October "	10	3
November "	28	7
December "	26	19
January, 1891	19	19
February "	14	11
March "	10	6

209. **Pollution of Lake Towns.** — Pollution of water-supplies from sewage is not confined to river towns. Cities situated on lakes, even on our Great Lakes, frequently suffer from contamination of their supplies through disposing of their sewage in the same way. This is

noted in a striking manner in the case of Chicago, which takes its supply from Lake Michigan. Although a portion of its sewage has been pumped for a number of years into the old Illinois and Michigan Canal, still the pollution of the lake-water has been constantly increasing through the drainage of the Chicago River and also the numerous sewer-outfalls that empty directly into the lake. Through the custom of emptying into the lake the dredge-dumpings from the river, the water-supply has also been grossly polluted at times and has caused epidemics of typhoid.*

The earliest water-intakes were located only a short distance from shore. These have been gradually extended into the lake from 1 to 2 miles, but the endemic condition of typhoid fever in the city and the enormous increase in 1891 led to the extension of the main tunnel to 4 miles in 1892, after which the amount of typhoid rapidly decreased, as shown in the following figures:

TABLE NO. 32.

TYPHOID DEATH-RATES IN CHICAGO PER 10,000 POPULATION BEFORE AND AFTER THE FOUR-MILE EXTENSION OF THE WATER-INTAKE.

'86	'87	'88	'89	'90	'91	'92	Av. 7 yrs.
6.8	5.0	4.7	4.8	8.3	16	10.4	8.0
'93	'94	'95	'96	'97	'98	'99	Av. 7 yrs.
4.2	3.1	3.2	4.6	2.7	3.8	2.5	3.4

Even the relative immunity obtained by the four-mile tunnel was not of long duration, for in a few years it was not uncommon to find at times that the water was polluted. A heavy rainfall that would flush out the river would frequently pollute the lake out to the four-mile intake. Even with the inauguration of the Sanitary Drainage Canal in 1900, which removed the larger part of the sewage from the lake, pollution of the supply occurs from time to time due to the increased pollution yet discharged into the lake. This is being gradually remedied by the construction of a system of intercepting and large lateral sewers.

210. **Typhoid and Polluted Wells.** — Although the larger epidemics of typhoid fever are necessarily connected with impure municipal water-supplies, still it also frequently happens that polluted wells are the means of distributing the virus of the disease. The opportunity for

* Bull. of Chicago Health Dept., Aug. 1899.

infection is considerably greater in the case of private or public wells, but the spread of the disease is likely to be more restricted because of the smaller number of users; but on the whole the aggregate of typhoid cases that are infected in this way frequently exceeds that caused by polluted general supplies. It often happens that persons acquire the disease in other towns than where the disease first becomes manifest (see Fig. 26), but, excluding this, by far the larger amount of typhoid fever incident to polluted water that occurs in other than urban popu-

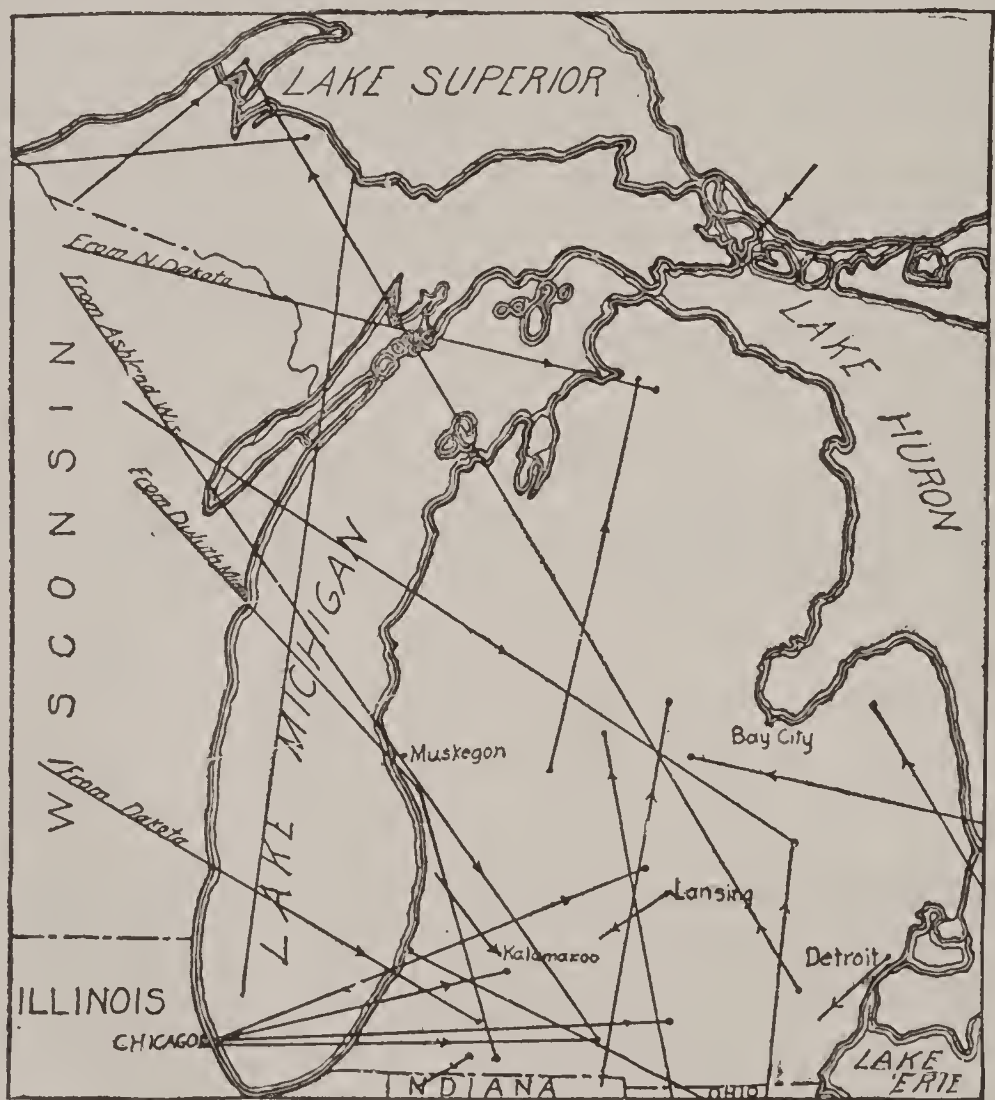

FIG. 26.—MOVEMENTS OF CONTAGIUM OF TYPHOID FEVER. (Mich. Board of Health.) Direction of arrows indicates movement of disease and shows how new foci are established by importing cases from without.

lations must come from infected wells. Even in cities a considerable proportion of this disease is attributed to the use of old wells. This is particularly the case in the more congested poorer quarters, where these older sources of supply are retained much longer than in the newer and better built portions of towns. In 1889 in Washington 626 fatal cases of typhoid occurred in families using water from about 300 different wells, a sanitary record for our capital city that is indeed humiliating.

The general decline in typhoid death-rates in cities coincides in a remarkable way with the introduction of public water-supplies, as has been noted especially in Massachusetts (Fig. 27).*

* 28 Rept. Mass. Bd. Health, 1896, p. 781.

211. Outbreaks Inaugurated from Single Cases.—While the contamination of municipal water-supplies is generally due to municipal sewage pollution, still it may at times happen that a single case of disease may be the means of inaugurating an outbreak, as in the Plymouth, Pa., case. This epidemic, consisting of over 1100 cases in a town of 8000 people, had its origin through the pollution of the

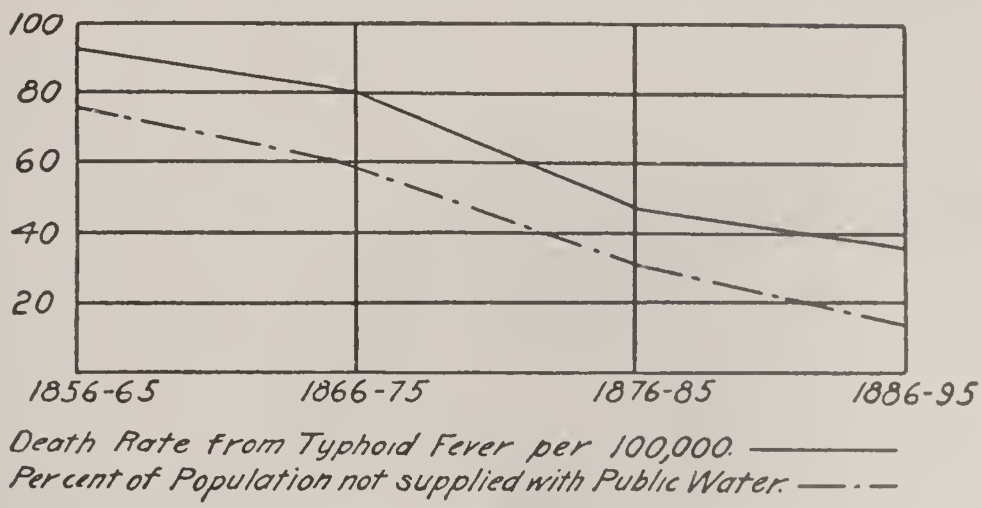

FIG. 27.—DECREASE IN TYPHOID DEATH-RATES COINCIDENT WITH INTRODUCTION OF PUBLIC WATER-SUPPLIES. (Mass.)

impounded drinking-water by the fecal discharges of a single patient. To prevent infection of the family vault, the dejecta were deposited on the surface of the snow. Soon after, heavy rains washed the frozen hillsides, the natural surface-drainage discharging into the stream that fed the supply-reservoir. Within about two to three weeks a very pronounced epidemic occurred that was confined closely to the patrons of the municipal water-supply.

212. Typhoid Rates an Index of Quality of Water.—A study of the death-rate or case-rate of typhoid fever in various towns and cities for a number of years illustrates in a striking way the relation that exists between this disease and the general character of the public water-supply. To some extent, typhoid may be introduced into a city from an external source (Fig. 26). This factor is generally more important in the smaller towns in which the transient population is relatively large, as in mining and lumbering regions. In some degree, the disease may also be traced to other causes than impure water, but by far the larger majority of cases are attributable to infection of this character. Cities deriving their supply from sources in which the probability of pollution is excluded have as a rule a very low typhoid death-rate; those, on the other hand, using surface-waters (impounded or streams) generally show an increase in this rate. Hill * has classified cities on the basis

* Public Water-supplies, p. 70.

of their death-rates from this disease into seven groups, beginning with those having a typhoid death-rate of ten or less per 100,000 population, and increasing each group by 10. The seventh class embraces all cities whose typhoid death-rate exceeds 60 per 100,000. It is a significant fact that reflects upon our sanitary methods in this country to observe that there is no American city in Class I or II with the exception of New York and Brooklyn. Hill has even suggested that

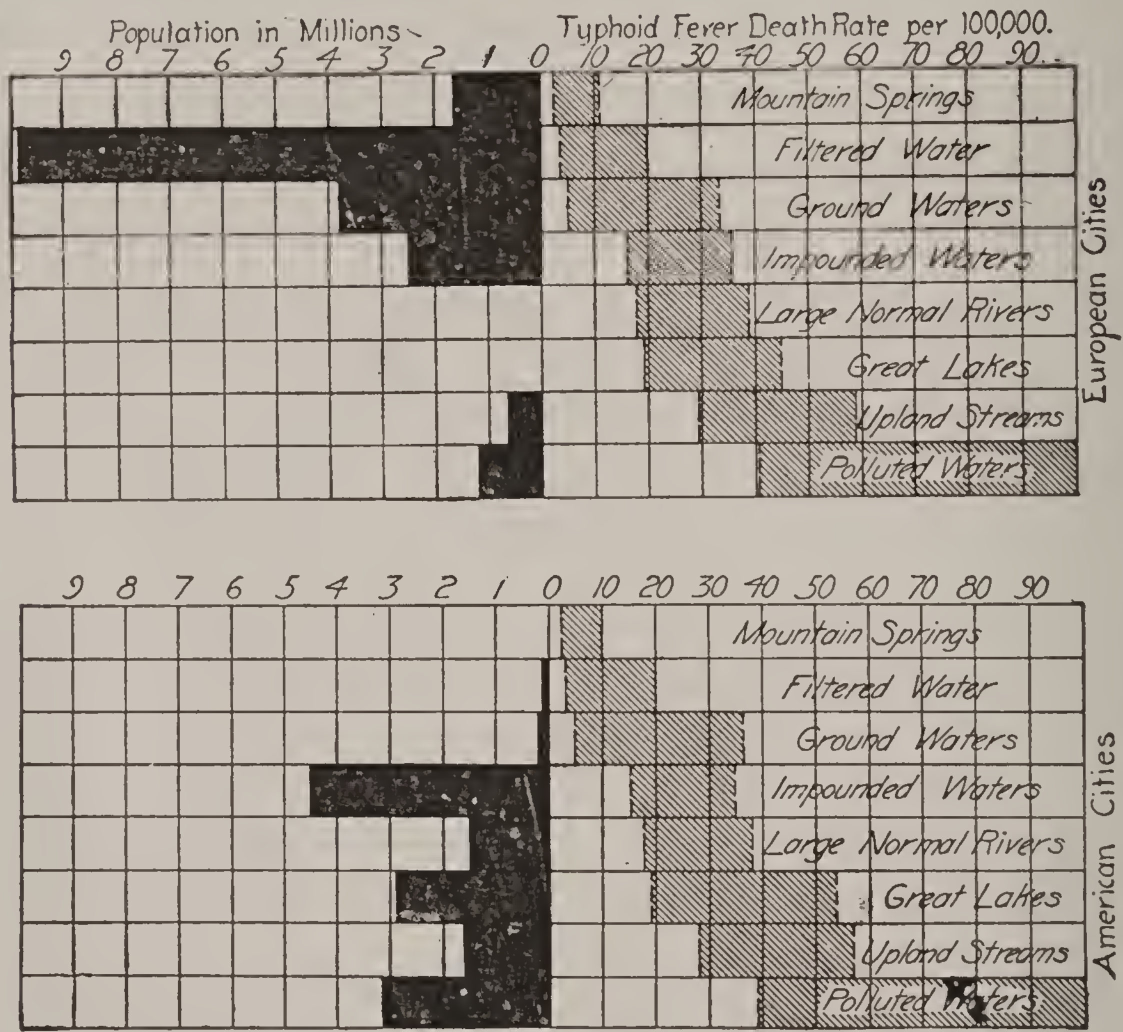

FIG. 28.—RELATION OF TYPHOID DEATH-RATE TO CHARACTER OF WATER-SUPPLY IN EUROPEAN AND AMERICAN CITIES; ALSO POPULATION SUPPLIED FROM EACH SOURCE. (Fuertes.)

contracts for supplies should be made on the basis of a certain death-rate from typhoid, but to determine the effect of any given improvement like this requires the collection of data for several years, and would therefore seem impracticable for this purpose.

Fuertes * arranges these statistical data on the basis of the kind of water furnished each municipality, as mountain spring, filtered water, ground-water, surface-water (streams, impounded waters, and lakes). In Fig. 28 the limits between which 75 per cent of the death-rates per 100,000 may be expected are shown for the different kinds of waters used; also the population using each class. Of the total population as

* Water and Public Health, p. 32.

charted (over 33,000,000), 20,000,000 are in European cities, the remainder in America. Over 75 per cent of the total European population here represented have a better supply than an impounded reservoir, like the Croton supply of New York, while over 75 per cent of the supplies furnished American cities are below this standard.

213. Diminished Typhoid Rates Incident to Improved Supplies.—From the typhoid death-rates it is very evident that the water-supplies of European cities are much better than those in America. This condition, however, has not always obtained, as in most cases European municipalities have had to pay the penalty of impure water by high death-rates before their supplies were bettered. The much denser population per square mile in Europe increases of course the amount of pollution in most surface-waters, and makes it thereby increasingly difficult for large cities to secure adequate supplies that are beyond the taint of suspicion. In mountainous regions pure natural waters can frequently be obtained, but in the cities situated on the seacoast and in the plains region sufficient natural supplies of pure surface-water are to be had only in exceptional instances. This has led to the purification of waters taken from available sources of supply. The diminished typhoid death-rates under these conditions, as compared with those that obtained before such improvements were made, indicate in the most conclusive manner the close relation that exists between the quality of water-supplies and public health, so far as water-borne diseases are concerned. These diminished typhoid death-rates, however, have not been gained entirely by securing an unpolluted or a purified water-supply, but in part through the introduction of improved systems of sewerage.

In Zurich the introduction in 1885 of new filters carefully controlled caused the following marked decrease in typhoid rates per 100,000 population:

TABLE NO. 33.

TYPHOID DEATH-RATES IN ZURICH, SWITZERLAND, PER 100,000 POPULATION, IN RELATION TO IMPROVEMENTS IN WATER-SUPPLIES.

Before Improvement.

'79	'80	'81	'82	'83	'84	'85	Av. 7 yrs.
33	80	43	48	27	174	110	73.6

After Improvement.

'86	'87	'88	'89	'90	'91	'92	'93	'94	Av. 9 yrs.
10	13	8	9	10	8	8.5	7.5	7	9.0

In the case of Munich the diminution in typhoid losses was coincident with the installation of the sewerage system, although the water-supply was not changed until several years afterward.

Fig. 29 shows the pronounced drop in the typhoid rate and the relation of the same to sewerage introduction.

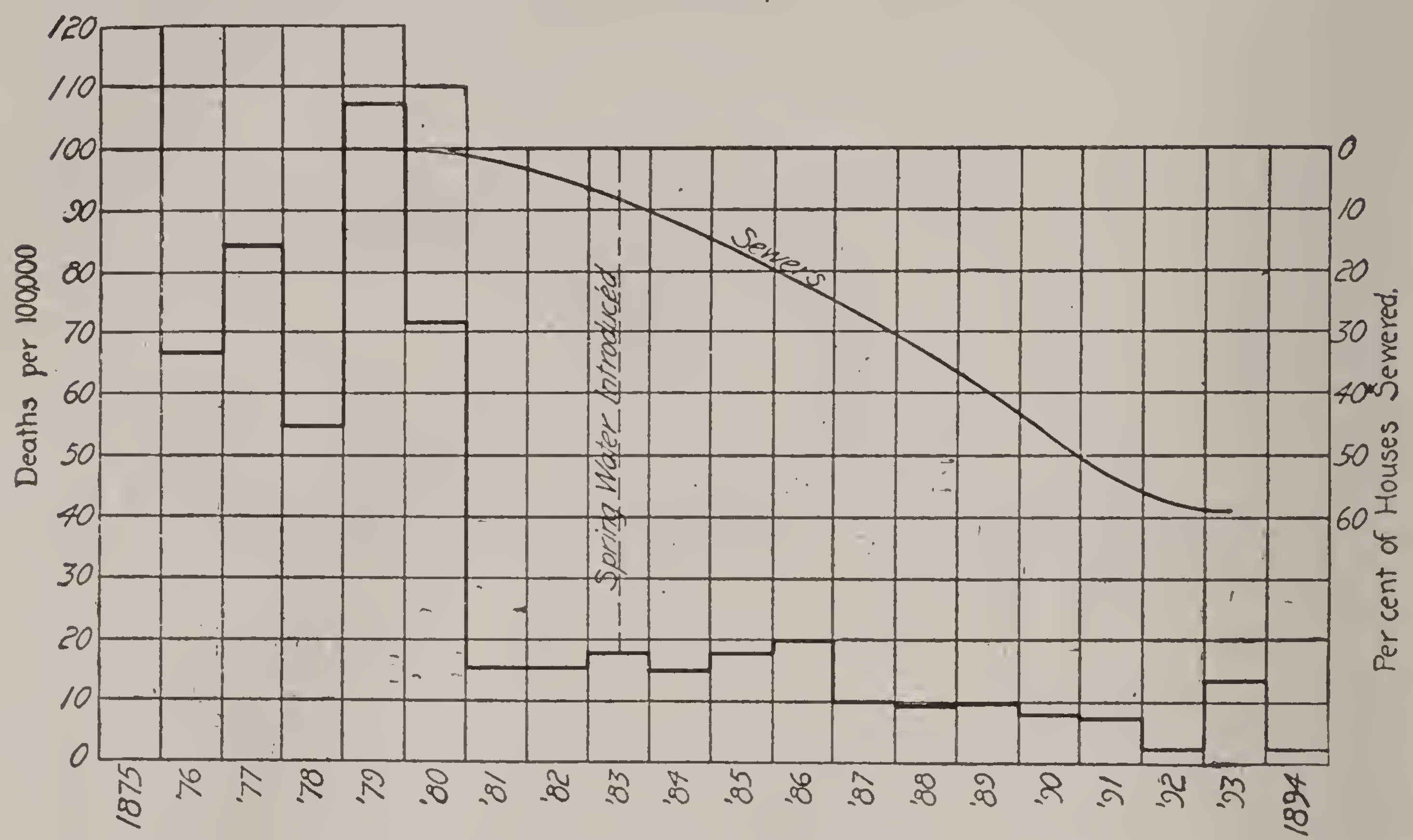

FIG. 29.—TYPHOID DEATH-RATE IN MUNICH; RELATION OF SAME TO INTRODUCTION OF SEWERAGE AND WATER-SUPPLY. (Fuertes.)

214. Seasonal Distribution of Typhoid Fever.—Typhoid fever does not rage with equal severity throughout the entire year. Usually the case-rate increases in late summer and fall, often reaching a maximum during the winter and then declining in the spring months. Fig. 30 indicates this unequal distribution. Of course during outbreaks of this disease this general rule does not obtain, as infection may rapidly pass from one person to another. Woodhead* attributes this higher case-rate in the fall to the higher temperature of the water, facilitating the growth of the typhoid organism, but this point is by no means thoroughly established. The ability of the organism to retain its vitality when frozen, even though it is not a

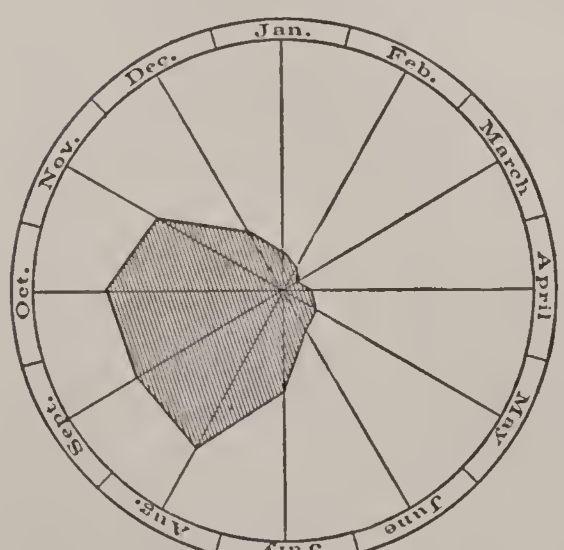

FIG. 30.—SEASONAL DISTRIBUTION OF TYPHOID FEVER. (Abbott.)

* Roy. Com. on Met. Water-supply, 1893, p. 506.

spore-producing germ (184, 223), shows how the disease may be spread even in winter.

215. Asiatic Cholera.—While cholera is a disease that is naturally "at home" in the Orient, particularly in India in the delta of the Ganges, still ever and anon it breaks over the boundaries that naturally limit it and becomes epidemic among western nations. Europe has been visited with this disease a number of times during the last century, the last outbreak occurring in Germany in 1892–3. It has less frequently invaded this country, although eight epidemics are recorded since 1832. The epidemic of that year and those of 1853–54 and 1873 were the most severe. Since the latter date, the disease has not occurred in this country.

Like typhoid fever it is primarily an intestinal disease, the organism associated with it developing luxuriantly in the intestine and therefore occurring in large numbers in the dejecta of cholera patients. This causative organism was discovered by Koch in 1884 in India, where he succeeded in isolating it from the intestinal contents of cholera patients; also finding the same in water from an open uncovered drinking-tank.

216. Cholera Outbreaks traced to Water-supplies.—In 1854 London was visited with a severe epidemic. The cholera death-rate in that portion of the city supplied by one company that drew its supply from the polluted Thames was 154 per 10,000, while in another quarter fed with an unpolluted supply there were only 17 deaths per 10,000.

The 1892–93 outbreak in Europe gave ample opportunity for the study of the disease in the light of modern methods. Although the specific organism had been found before in several cases associated with epidemics of the disease, many new data were gathered at this time and the relation of the cholera organism to water-supplies thoroughly confirmed by bacteriological examinations. In these studies it was also found that surface-waters not infrequently contain other bacteria of the spirillum type. Many of these closely simulate the cholera or comma organism, as it is sometimes called on account of its shape, but Pfeiffer's test and certain cultural methods permit of a ready differentiation (149).

The most striking illustration of the way in which the disease is spread by water-supplies is shown in the Hamburg outbreak. Hamburg, a city containing at that time a population of 640,000, and Altona, a city of 150,000 people, are situated on the River Elbe near its mouth. The two cities are practically one, as they merge into each other, although they have a separate city government. Hamburg at this time drew its water-supply from the Elbe some distance above the city. Altona, situated just below and forced also to use the Elbe water, took it at a point 8 miles down-stream, treating it by sand filtration because of its impure condition. Hamburg therefore received unfiltered Elbe water, subject of course to possible pollution; Altona used filtered river-water, taken from the stream after it had received the sewage of over 800,000 people. Cholera broke out in the fall of 1892, and during this epidemic there were 17,000 cases (16,800 in less than two months) in Hamburg with over 8600 deaths, while during the same time there were only about 500 cases with about 300 deaths in Altona. Hamburg with its unfiltered river-supply had a case-rate of about 263 per 10,000 and a death-rate of 134, while in Altona the

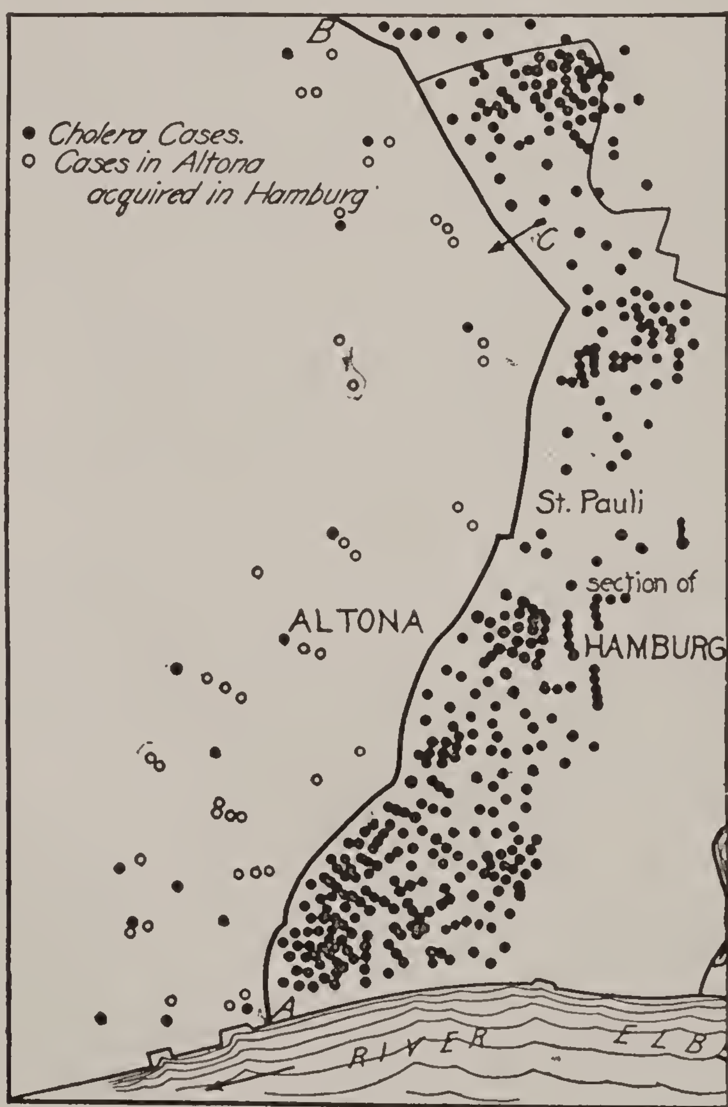

FIG. 31. HAMBURG-ALTONA EPIDEMIC OF CHOLERA IN 1892.

Deaths from cholera are shown in district 400 meters each side of Hamburg-Altona boundary. Section in Hamburg marked *C* was supplied with Altona water and wholly escaped the disease.

case-rate was 38.1 and the death-rate 21.3. Of the number in the latter city it must be remembered that the disease was contracted in many cases by people who worked in Hamburg but lived in Altona. One block of buildings in Hamburg, containing about 400 people, received its water-supply from Altona rather than Hamburg on account of local difficulties in connecting the main. This spot (*C* on map), although surrounded with cholera cases, remained free from the disease. Several of the large hospitals, garrisons, and prisons in Hamburg that used other water than the municipal supply escaped with little or no disease. The history of the epidemic shows in the most conclusive manner that the river-water was the means by which the disease was spread. In fact, the cholera spirillum was isolated not only from water taken from the Elbe, but also in one of the Altona filter-basins before the water was submitted to filtration.*

217. **Anthrax.**—This disease is not often disseminated by means of the drinking-water, but waters of surface origin may receive drainage from fields on which the disease may be present, and so become contaminated. This condition is especially liable to occur in those regions (Europe, Asia, and Africa) where the disease is severe. Here in this country it is not established except in a few localities (Lower Mississippi valley, Lower Delaware River, etc.).

Rivers are more apt to be the distributive agents so far as waters are concerned. On account of using hides and skins imported from infected regions refuse from tanneries, brush-factories, and morocco-shops disposed of in running streams may often be the cause of outbreaks along watercourses.

A striking instance came under the writer's attention in 1899. The Black River, for a distance of 10 miles below Medford, Wis., was polluted by tannery refuse. Stock (cattle and horses) contracted anthrax by drinking the river-water, by grazing on lowlands that had been subjected to overflow in the spring, and by eating hay that had been gathered from the inundated marshes. In caring for the affected stock several persons also became infected. The disease germ was introduced from a tannery in which Chinese hides were being handled.

Diatroptoff† notes the detection of the specific organism in the case of a well-water. The water from this well served to infect a herd of sheep. The writer in the Medford outbreak succeeded in isolating the disease organism from a pond that had become infected by surface drainage from fields on which cattle had died from anthrax.

218. **Other Water-borne Diseases.**—In addition to the above, a considerable number of other diseases are also distributed more or less frequently by the aid of water. In some cases the causal organism that produces the disease is not yet known, but the manner in which the outbreak is disseminated leaves no room for doubt as to the probability of water functioning in its spread. The winter outbreaks of cholera

* *Zeit. f. Hyg.*, XIV. p. 337. † *Ann. Past.*, 1893, VII. p. 286.

infantum that have occurred in Hamburg and Altona have been traced directly to the use of raw or imperfectly filtered Elbe water.*

219. **Gastro-intestinal Disturbances.**—As representing this class of diseases may be mentioned gastro-intestinal catarrhs. In some cases a diarrhœic condition may be produced as the result of the presence of suspended matter. In a number of instances, epidemics of intestinal catarrhs have been associated with the pollution of waters with organic matter from various sources. The Long Branch,† N. J., outbreak was ascribed to the use of peaty water, but it was not definitely shown whether the disturbance was due to the organic matter of peaty origin or to organisms that were present in such water.

Wright ‡ instances an outbreak in Buffalo that was confined entirely to persons who used water from a series of shallow wells in a certain region of the city.

Cameron § records an epidemic in a military school in Dublin where 150 persons were afflicted. The trouble was traced to a ground-water that was found to be rich in micro-organisms.

In 1894 a very extensive outbreak of an enteric disease appeared in Lisbon, ‖ Portugal. In three months over 15,000 people were afflicted. The symptoms of the disease appeared like cholera in many ways, but the fact that only one person died from the same indicated at least that the germ in its pathogenic properties was much different from true Asiatic cholera. The organism producing the outbreak was readily separated from fecal discharges of affected persons, and was also found in the water-supply of the city. It bore a striking resemblance to the comma bacillus of cholera. The protection afforded by the use of household filters demonstrated conclusively that the disease was distributed by the way of the water-mains.

220. **Dysentery.**—Although it is quite probable that dysentery may be caused by more than one kind of organism, the relation of diseases of this class to polluted waters is now quite generally accepted. The severe form of the disease that occurs in the tropics is ascribed to the development of an animal parasite, *Amœba coli*, while the disease as it appears in some other countries seems to be associated with certain bacteria. As these organisms have not been definitely determined in water-supplies, the supposed connection between them and water does not rest upon a thoroughly established scientific basis, but is based upon the distribution of the disease and other epidemiological data.

* Hazen. Filtration of Public Water-supplies, p. 228.
† Mason. Water-supply, p. 11.
‡ *Sanitary Record*, IV. p. 185.
§ *Dublin Jour. Med. Sc.*, I. p. 535.
‖ *Cent. f. Bakt.*, 1894, XVI. p. 401.

221. Malaria.— Regarding the spread of malaria by means of water-supplies, not much definite information that is scientifically established is at hand, although the general belief has been that the disease is sometimes spread in this way. The recent establishment of the mosquito theory of infection shows, however, that water is necessary for the continuance of the disease, although there is no evidence that the malarial parasite is introduced with the water that is ingested, even though such water might contain the larvæ of the spotted-wing mosquito (*Anopheles*) that is now known to be the means of distributing the disease.

VITALITY OF PATHOGENIC BACTERIA IN WATER.

222. Conditions Affecting Vitality.—While common experience has for centuries associated certain diseases with impure or polluted water-supplies, it has not been possible until the methods of bacterial investigation were employed to determine just how long a water once rendered impure through fecal or other pollution would remain dangerous to human health. Since the discovery of the specific organisms that are the inciting cause of different diseases, and the study of them under varying conditions in waters of diverse sources, as to how long such organisms are able to retain their vitality in natural waters, it has become possible to limit much more accurately this period of danger.

It is very important to recognize:

(1) Whether pathogenic bacteria once introduced into water are able to multiply therein; and,

(2) Supposing that conditions do not favor their growth, how long such pathogenic bacteria are able to retain their vitality and virulence.

Having once determined these conditions, it then becomes possible to state with some degree of accuracy the period during which water polluted with such germ-life is dangerous.

Any satisfactory answer to these propositions must take into consideration a number of conditions, both as to the organism and the influence of its environment, that will exert a varying effect on the vitality of any germ. The more prominent of these factors are as follows:

Natural variation in the organism itself, due to age, condition of culture, and previous history of the same; the number of disease germs present in the water; the condition of the water as a growth medium as to its composition, the amount of organic matter, the nature of the same, whether it is suspended or soluble, the presence of by-products of previous bacterial growth, the chemical reaction of the water itself; the effect of varying conditions in environment as to the temperature of the water, the amount of oxygen dissolved, the effect of light, and the state of water as to motion.

All of these factors exert more or less effect on the vitality of bacteria, and particularly on that of disease-producing microbes. Inasmuch as these conditions are not constant in all waters, it naturally follows that any specific organism will be subject to a good deal of variation in the length of time it will remain in a living condition in water. It is therefore not so surprising to find considerable difference in experimental results, which fact should lead to caution in deducing definite laws as to this question.

223. **Vitality of Typhoid Organism.**—The closer relation of typhoid fever to polluted water-supplies renders a determination of the vitality of this germ of more than ordinary importance. Although the typhoid-fever bacillus is not a spore-bearing form, nevertheless it is able to retain its vitality in drinking-waters for a considerable period of time, as is evidenced by the numerous outbreaks traceable to infected supplies.

Under ordinary conditions, from the direct experiments already made, it seems improbable that there is any considerable growth of the typhoid bacillus in potable waters, although, as Frankland * has shown, it is possible to acclimate the organism to such a dilute food-medium that it will actually grow in surface-waters; but under the circumstances in which it would naturally find its way into potable supplies there would be but scant opportunity for this acclimation process to occur. Where organic matter is present that is available as a food-supply, as in sewage-polluted waters, cell-multiplication may be possible,† but even here there are other retarding factors, such as the effect of bacterial by-products, that tend to prevent growth.

In order to determine the period through which the organism is able to survive in water, a large amount of data has been collected. The results, however, are so conflicting that it is impossible to closely define these limits.

Of the earlier work, Frankland's seems to have been most closely controlled. He studied the longevity of typhoid in polluted Thames water, a soft peaty water (Loch Katrine) and a hard, deep well water. Tests were made in raw, sterilized and filtered samples. The results obtained with sterilized and filtered are, of course, inapplicable to normal

* *Zeit f. Hyg.* 1895, XXI. p. 406.

† Olivier. *Comp. rend. d. sc. Soc. de Biol.*, 1889, No. 27.

conditions, but the prolonged vitality of the organism in all cases (20–51 days in sterilized and 11–39 days in filtered) compared with the effect in raw (9–33 days) indicates that the longevity of the organism is less in raw waters than in sterile waters. In this series the typhoid organism disappeared much more rapidly in surface waters than in unpolluted well waters.

All experimental work on the vitality of organisms carried on in glass containers is subject to a factor of error due to the protective action of the glass vessel on the organism as shown by Ficker.

Jordan, Zeit and Russell * have carried on, simultaneously but independently, a most extensive series of experiments on the influence of the waters of Lake Michigan and Illinois River on the vitality of typhoid. To avoid influence of glass containers, their experiments were made in diffusible membranes, as parchment and celloidin. Sacs made of this material and filled with respective types of waters (Lake Michigan, sewage from Chicago Drainage Canal, and Illinois River) were inoculated with freshly isolated typhoid organisms and immersed in these respective waters. The typhoid organism was recovered from these water samples by various differential culture methods, and in every case the presumptive cultures were crucially tested by the agglutination (Widal) test. The results uniformly indicated that the exposures in the sewage and sewage polluted river water resulted in the destruction of the typhoid more rapidly (3 days) than in Lake Michigan water (about 7 days).

Russell and Fuller † continued these investigations, using Lake Mendota water and dilute fresh sewage. They studied the permeability of the containing sacs, introducing still another type, agar membranes. and their results substantially confirmed those previously referred to. It is apparent from these investigations that the forces which result in the destruction of the typhoid organism operate much more rapidly in highly polluted than pure waters.

In solving so important a question as this, it is, of course, well to weigh carefully all possible sorts of evidence. As supplementing the experimental findings, epidemiological evidence would be of great value, where towns using the same stream for sewage disposal and water supply might have successive epidemics of this disease. Such findings, however, are not frequent.

The evidence is practically unanimous that this organism persists longer in cold waters than in those of summer temperature. At Lawrence the rate of decrease was noted as follows when the typhoid

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* *Journ. Inf. Diseases*, 1904, I. p. 641. † *Journ. Inf. Diseases*, Supp. No. 2, Feb., 1906.

bacillus was exposed in Merrimack River water kept near the freezing-point.

Day of analysis................	1	5	10	15	20	25
Number per c.c..............	6120	3100	490	100	17	0

The spread of the disease from Lowell to Lawrence during the winter, and the Plymouth, Pa., case in which typhoid dejecta were exposed in the snow from January to March to a minimum temperature of — 22° F., indicate that low temperatures are certainly ineffective agents in the destruction of this organism.

224. **Cholera.**—In the experimental results obtained as to the vitality of the cholera spirillum in natural waters, the data are even more conflicting than with typhoid. In order to encourage growth, Bolton * found that about 400 parts of organic matter per 1,000,000 were necessary. This explains why the germ lives longer in a polluted than in a pure water. Trenkmann † has determined that the vitality of the cholera spirillum is considerably prolonged where the organism is grown in solutions containing sodium chloride. This is of interest as explaining the presence of the organism in brackish waters (river Elbe at Hamburg,‡ harbor at Marseilles).

In general, the experimental results indicate that the cholera spirillum is unable to retain its vitality in potable waters for as long a time as the typhoid bacillus. In the majority of experiments cited, the duration of vitality was only 1–3 days. On the other hand, some reputable observers claim to have found it in ordinary water several months after infection. In Cologne sewage Stutzer and Burri § found it lived from 7–13 days.

At low temperatures it retains its ability to grow, as has been determined experimentally, as well as empirically in the winter epidemics that occurred at Nietleben and Altona in 1893.‖

Owing to the fact that the period of incubation with this disease is quite short (1–5 days), it is more often possible to detect the presence of this organism in polluted supplies than it is with typhoid. (See Literature of this chapter.) For such determinations the differential media that have been devised may be successfully used (149).

225. **Anthrax.**—The problems presented in the case of this disease organism differ materially from those previously noted, in that *Bacillus*

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* *Zeit. f. Hyg.*, I. p. 1.

† *Cent. f. Bakt.*, 1893, XIII. p. 313.

‡ The chlorine content of river is greatly increased by the waste waters from the Stassfurt salt-works (See Aufrecht, *Cent. f. Bakt.*, 1893, XIII. p. 353.)

§ *Hyg. Rund.*, IV. p. 208.

‖ Koch. *Zeit. f. Hyg.*, 1893, XIV. p 393.

anthracis is able to form spores, and hence is much more resistant. Spores, however, are only produced in contact with air and where the temperature is at least 60° F.

There is little probability of the pollution of waters by anthrax from human sources, but it not infrequently happens that this disease organism finds its way into water from animal sources, and inasmuch as the same germ is able to produce anthrax in both man and animals, the origin of the same is a matter of no little moment. Tanneries, brush-factories, etc., are particularly liable to distribute the disease germ by the way of the water, owing to the fact that hides, hair, and wool are frequently infected. In such cases the disease germ is more apt to be in the spore rather than the vegetative form and therefore will be much more resistant.

In the sporeless stage the organism is able to live but for a short time. Two to five days mark the ordinary limits of existence in surface-waters, the organism degenerating more rapidly in summer than in winter. Under summer conditions the germ may sporulate, in which condition it is able to live over from one year to the next. In lowlands subject to overflow, the conditions seem to be the best for the perpetuation of the vitality of the organism.

226. Conclusion.—Experimental tests have been made with other kinds of pathogenic bacteria, but the results are only of general scientific interest. In summarizing, all bacteria of disease are killed out sooner or later in waters. Ordinarily the amount of organic nutriment contained in water is not sufficient to encourage rapid development, and the consequence is that most forms are sooner or later starved out.

LITERATURE.

A large amount of literature showing the relation of communicable diseases to water-supplies is in existence, but for the most part it is widely scattered in various hygienic, bacteriological, engineering, and other journals. In a few such works, as

Hill's Public Water-supplies,
Fuertes' Water and Public Health,
Abbott's Hygiene of Transmissible Diseases,
Sedgwick's Principles of Science and Public Health, and
Whipple's Typhoid Fever,

some of the more classic examples are given. Typhoid epidemics may be classified according to their respective vehicles of transmission as due to

(1) Water-supplies.
(2) Milk-supplies.
(3) Food (shell fish, oysters, etc.)

while more recently flies have been shown to be actively associated with the distribution of infected material, as in the typhoid fever outbreaks in the military camps during the Spanish-American war, yet the larger percentage of epidemics of typhoid fever are caused by infection in various ways of water-supplies. See Schüder, Zeit. f. Hyg., 1901, XXXVIII. p. 343, in which there is collected literature relating to 650 typhoid epidemics.

TYPHOID.

1. *Epidemics arising from Infected Ground Water-supplies*, (springs or wells) are usually more or less circumscribed in their distribution. Characteristic epidemics are noted in the

Wittenberg, Germany, outbreak, which was due to infection of a well supplying garrison. (See Gaffky. Mitt. a. d. kais. Gesundheitsamte, 1884, II. p. 410.)

Lausanne, Switzerland, 1872. Infection of town supply (spring) through imperfect filtration of soil.

Deutsche Arch. f. klin. Med. 1893, Band XI.

2. *Epidemics caused by Accidental Infection of a Satisfactory Supply.*

Baraboo, Wisconsin, infection of a pure supply from wells by passage of distributing pipes through polluted water. (See Kirchoffer, W. G., *Eng. News*, Nov. 27, 1902; also Russell, H. L. Report Wisconsin State Board of Health. 1903.)

Butler, Pa. Infection of filtered supply due to temporary discontinuance of filter operations. Soper, G. A., *Eng. News*, Dec. 24, 1903.

3. *Epidemics caused by Contamination of Supplies of Surface Origin.*

Ithaca, N. Y. (See Soper, G. A., Jour. N. E. W. W. Assn., December, 1904.)

Plymouth, Pa., 1885. One-ninth of entire population of 9,000 stricken with disease due to pollution of open public reservoir with fecal discharges from single typhoid case. (See Sedgwick, Principles of Sanitary Science, p. 200.)

Pittsburg, Allegheny and vicinity, *Eng. News*, Feb. 25, 1904.

Philadelphia, Pa., Annual Report, Dept. Public Safety of Phila., 1898.

Lowell–Lawrence, Mass., 24th Report Mass. Board of Health, 1892.

Washington, D. C., *Eng. News*, Nov. 8, 1906.

4. *Epidemics due to Infection of Milk-supplies.*

For most complete recent résumé, see Milk and Its Relation to Public Health, Bulletin 41, Hygienic Laboratory, Public Health and Marine Hospital Service of the U. S., 1908.

Stamford, Conn., 1895. Three hundred and eighty-six cases developed within six weeks, of which 97 per cent came from a single milk supply, milk being infected by rinsing out the cans with cold water from a shallow contaminated well. (See Smith, H. E., Conn. State Board of Health Report, 1895, p. 161.)

Montclair, N. J., 9th Annual Report Montclair Board of Health, 1903.

Palo Alto, Cal. Of 900 people supplied with milk from one dairy 232 had typhoid fever. (See Modern Medicine, Osler, Vol. II. p. 85.)

Springfield and Somerville, Mass., 24th Report Mass. Board of Health, 1892, p. 715.

CHOLERA.

Hamburg–Altona, Germany, 1892. The most striking case on record of the value of sand filters in checking disease outbreaks.

Koch. Wasserfiltration und Cholera. Zeit. f. Hyg., 1893, XIV. p. 393, also ibid., XV. p. 89.

Reincke. Ber. d. medic. Inspect. d. Hamburg. Staates f. 1892, p. 28.

Gaffky. Arb. a. d. kais. Gesundheitsamte, X. pp. 1–129.

Cholera in Germany other than in Hamburg in 1892–93.

Arb. a. d. kais. Gesundheitsamte, X. pp. 129–273.

Korber. Dorpat outbreak. Zeit. f. Hyg., 1895, XIX. p. 161.

Koch. Nietleben outbreak. Zeit. f. Hyg., 1894, XV. p. 123.

Cholera in Germany in 1894.

Arb. a. d. kais. Gesundheitsamte, XII. pp. 1–285.

DIARRHOEAL EPIDEMICS.

Hamburg–Altona, 1880, 1888, 1892.

Reincke. Ber. d. med. Inspect. d. Hamburg. Staates für 1892, p. 10.

Bockendahl. Generalber. ü. d. öffentl. Gesundheitswesen für Schl. Hol., 1870, p. 10.

(Abstracts of both of these articles in Hazen's Filt. Pub. Water-supplies, p. 226.)

PART II.

THE CONSTRUCTION OF WATER-WORKS.

CHAPTER XI.

GENERALITIES PERTAINING TO WATER-WORKS CONSTRUCTION.

227. In the preceding chapters there have been discussed the various matters relating to the requirements of a water-supply and the capabilities of the various sources as regards quantity and quality. In the remaining portions of this work there will be considered in detail the design and construction of the various parts entering into a system of water-works.

Questions of quantity and quality are of prime importance in the selection of a source of supply, but that the solution may be the best it is also necessary to consider the question of cost, a matter which depends upon the extent and character of the various parts of the works involved. With two or more sources at hand, each of which will furnish water of sufficient quantity and equally good quality, the problem resolves itself into one of economy as measured by the first cost plus the capitalized cost of operation. The problem is, however, rarely so simple as this, questions of future enlargement, differences in quality, possible future pollution, and financial resources of the community being some of the elements which render the question a complicated one. Thus a complete general knowledge of the problem becomes a prerequisite to an intelligent selection of source as well as to the actual construction of the works.

Before passing on to the details of water-works construction it will be of assistance to get a general view of the subject, and to that end we will here briefly describe the various general features, the arrangement of the various parts of a system, and the standards by which the economy of various methods and arrangements can be compared.

GENERAL ARRANGEMENT OF WATER-WORKS.

228. Classification.—The various constructive features of a water-supply system are divided into three groups: (1) Works for the collection of water; (2) Works for the purification of water; (3) Works for the conveyance and distribution of water.

229. Works for the Collection of Water.—These are divided according to the nature of the source into: (A) Works for taking water from large streams and natural lakes; (B) Works for the collection of ground-water; (C) Works for the collection of water from small streams by means of impounding reservoirs.

A. Works for taking water from large streams or lakes vary in character from a simple cast-iron pipe extending a short distance from shore, to the expensive tunnels and cribs of some of the large cities on the Great Lakes. The location of these works is determined very largely with respect to the quality of the water obtainable. Wherever, as is often the case, it is desired to draw a supply from a lake which at the same time receives sewage from the city, the question is one involving great difficulties.

B. Works for the collection of ground-water consist of various forms of shallow wells, artesian wells, filter-galleries, etc. The location of works of this class is determined, primarily, by the location of the water-bearing strata. If these are extensive, it will usually be convenient and economical to place the wells at relatively low elevations in order that the water may be readily reached by pumps, or perhaps in order that a flowing well may be secured. In the case of shallow wells the location is often affected by the possibility of local contamination, an element usually absent in the case of deep wells.

C. Water collected in impounding reservoirs from streams of comparatively small watersheds depends for its good quality chiefly upon the scarcity of population upon the watershed. Suitable areas are therefore more likely to be found in the more rugged parts of the country and at the higher elevations, and usually at considerable distances, sometimes as great as 50 or 75 miles, from the population to be served. The location of such impounding reservoirs is also largely dependent upon questions of construction, such as the location of the dam, length and cost of aqueduct or conduit, and, what is of great economic importance, whether the water can be conveyed and distributed entirely or partly by gravity.

230. Works for the Purification of Water.—These vary in kind according to the nature of the impurities to be removed. Thus in the case of surface-waters the sediment, bacteria, etc., are removed more or less completely by settling-basins and various forms of filters; disagreeable gases by aeration. In the case of ground-waters iron may be removed by aeration and filtration; hardness by chemical precipitation, etc. In these ways waters otherwise very undesirable can be greatly improved or made entirely satisfactory, but of course at a considerable expenditure of money. It will often happen, therefore, that a source of good quality but expensive will need to be compared with another poor in quality but capable of being made fairly comparable with the other at no greater total cost. Not infrequently the possibility of the future deterioration of a surface supply and the consequent necessity for artificial purification must also be considered.

231. Works for the Distribution of Water.—These include aqueducts and conduits for conveying water from a distant source, pumps and pumping-stations, local reservoirs for equalizing the flow or for storage, and the pipes for distributing to the consumers. Conduits may be open channels, masonry conduits not under pressure, or closed pressure conduits, such as pipes of wood, iron, or steel, and sometimes tunnels. The form is determined chiefly by considerations of cost. Pumps are used in a great variety of forms and situations, and may be operated by steam, gas, electricity, wind, or by hydraulic power. There are deep-well pumps for drawing water from depths not reached by suction, low-lift pumps for raising water from a river into settling-basins or on to filters, or from wells into a low reservoir; and high-lift pumps for forcing the main supply into the distributing pipes or into an elevated distributing reservoir. Local reservoirs are used for receiving water from long conduits and regulating the flow in the distributing system, for equalizing the flow and pressure in pumping systems, and as settling-reservoirs. The pipe system includes distributing mains, fire-hydrants, service-pipes, shut-off valves, regulating-valves, etc.

232. Arrangement of Works.—The arrangement, extent, and cost of the various features of a water-works system depend greatly upon the nature of the source, its distance from the district to be served, and its elevation above that district.

In describing the various arrangements of water-works systems it will be convenient to consider them in two classes: first, those drawing from a distant source; and second, those drawing from a near-by source.

Where the water is obtained from a distant source we may have:

(*a*) gravity systems in which water from an impounding reservoir or lake (rarely from other sources) is led into a conduit through which it flows down to the city; or (*b*) systems in which the water is pumped from ground-water sources, or from rivers or lakes, by low-lift pumps into a gravity conduit, or by high-lift pumps directly into a pressure conduit to the city. At the city it passes into a small reservoir and thence by gravity to the consumer, or it may be pumped from the reservoir to a higher level in order to get the necessary pressure for distribution. Where the differences of elevation in the city are great it may be economical to have two or more zones of distribution. If the water is to be purified, the necessary works may be located at any convenient point between the source and the city. If placed at the source, a set of low-lift pumps will probably need to be established; if at any other point along the conduit, such pumps will seldom be required.

A near-by source is usually at so slight an elevation above the city that high-lift pumps are required to furnish the necessary pressure for distribution. With a ground-water source a set of low-lift pumps may often be used to elevate the water into a low equalizing reservoir, whence it is drawn by the high-lift pumps. If the source is a lake or large stream and filters are used, low-lift pumps will usually be required to pump the water upon the filters, although gravity may sometimes be used for this. If the source is an impounding reservoir, it is occasionally at so high a level that a gravity system may be employed.

The most expensive arrangement is in general a distant source at a low elevation where purification is required, such as an impure water brought from a distance. The cheapest is a near-by source of pure water at a high elevation, such as a spring-water or artesian water under pressure. In the nature of things, comparisons between such sources will seldom need to be made. Systems requiring careful comparison are usually various near-by sources requiring pumping and possibly purification with various remote sources of pure water usually located at a high elevation.

233. **Systems of Operation.**—According to the arrangement of the works there are several so-called "systems" of distribution: (1) by gravity; (2) by pumping to reservoir; (3) by pumping to stand-pipe or tank; and (4) by pumping direct. In (1) the water is conveyed entirely by gravity. In (2) it is elevated to a distributing-reservoir, whence it flows by gravity into the pipe system. In (3) a small stand-pipe or tank is substituted for the reservoir, while in (4) the water is pumped directly into the mains. In all these methods the pipe system remains essentially the same.

In many cases a reservoir or standpipe is so arranged that it receives only the surplus water when the rate of pumping exceeds the demand, and returns this surplus at times when the demand exceeds the rate of pumping. This may be considered a combination of (2) and (4) or (3) and (4), and is sometimes called the direct-indirect system. Again, it is often desirable in the case of a reservoir or stand-pipe system to so arrange the piping that in case of fire the reservoir may be shut off and an increased pressure furnished directly by the pumps.

The number of works in the United States operated under the various systems are given in Table No. 34, compiled by Flynn.*

TABLE NO. 34.

NUMBER OF CITIES AND TOWNS IN THE UNITED STATES SUPPLIED WITH WATER BY THE VARIOUS SYSTEMS NAMED.

System.	Northeastern States.	Southeastern States.	North Central States.	Western States.	Total.
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
Direct and to 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
Direct and to stand pipe	41	20	115	185	361
Direct and to reservoir and stand-pipe	3	2	18	10	33
Total	550	278	692	822	2342
Natural Pressure	0	9	2	10	21
Grand total	1159	339	714	1069	3281

234. Comparison of the Various Systems.—In comparing these various systems, their relative advantages and disadvantages should be considered in three respects: safety or reliability of operation, economy, and convenience. The first element is the most important, particularly

* *Eng. News*, 1898, XL. p. 10.

for large cities; for in such a case the entire community depends so absolutely upon the maintenance of the public water-supply that a failure for even a day would be a calamity. In smaller cities and towns it would be of much less importance, but yet a very vital factor in determining the value of a water-supply to the community. This element of safety cannot readily be measured in dollars and cents, but the experience of many places having an imperfect plant, and the losses resulting therefrom, show that it is a matter justifying a large measure of consideration.

In the matter of economy, differences are more readily measured. In comparing the convenience of two systems we should consider the amount and uniformity of pressure in the two cases, convenience in the operation of pumps and in the making of repairs and renewals, use of hose *versus* fire-engines, etc. All of these involve more or less also the question of economy.

235. *Safety.*—In respect to safety or reliability of operation the gravity system undoubtedly ranks first. The nature of the structures is such as to render them little liable to accident, and if a reservoir of from 5 to 10 days' capacity is provided to receive and distribute the water from the conduit, thus allowing time for repairs, or if the conduit is in duplicate or of masonry underground, this system is exceedingly safe and reliable.

Next to the gravity system in point of safety is the system of pumping to an elevated distributing-reservoir holding several days' supply. If at the same time considerable reserve pumping capacity is furnished to enable ordinary repairs to be made without drawing largely from the reservoir, this system is not far inferior to the gravity system. Certain rare though possible contingencies, such as a shortage of fuel or a boiler explosion, must, however, be considered as tending to place this system second in point of safety. Hydraulic power is in this respect more reliable than steam-power.

Many water-works have in place of a reservoir a small tank or stand-pipe holding at most but a few hours' supply, dependence being placed entirely upon the pumps for any continued excessive draught. This arrangement is manifestly inferior to the second system, and should not be used if a suitable site can be found for an elevated reservoir. In many places where the stand-pipe or the direct-pressure system has been in use, elevated reservoirs have subsequently been constructed.

The system of direct pumping depends for its efficiency entirely upon the ability of the pumps to follow all variations in consumption and to respond at any instant to demands for fire purposes. It ranks last in reliability, and should not be considered except for localities of level topography, where it becomes a question between this and the stand-pipe system. For small or moderate-sized cities an elevated tank holding at least one hour's fire consumption is an important element of safety and greatly to be desired. For large cities the fire rate does not call for such a large relative increase in pumping capacity and it can therefore be more readily met by the pumps. The total quantity used is, moreover, large, and small tanks would be of little value. The pumping machinery in large works is also more likely to be at all times in good working condition than is the case with small plants.

Where a stand-pipe is used for ordinary domestic pressure and dependence is chiefly placed on direct pumping for fire purposes, the stand-pipe may still be of considerable value in furnishing a fire pressure suitable for certain of the lower districts of the town or for small fires in the residence portion.

236. *Economy and Convenience.*—The relative economy of different systems for a given city is a matter depending entirely upon the local conditions. Compared to a pumping system, the gravity system is very economical of operation, and the depreciation of the plant is also likely to be small. If the source is quite remote, the expense of conduit becomes an important item, and beyond a certain distance the high initial cost will outweigh the economy of operation. As a gravity system is usually fed from an impounding-reservoir, there is also involved in this case the expense of reservoir construction.

Comparing the various forms of pumping systems, a large distributing-reservoir, while adding to the first cost, is an element of economy in enabling the pumps to be more uniformly and economically operated and in reducing slightly the necessary size of the piping. In small works the pumps can thus be operated a convenient number of hours each day, such as 8, 10, or 12. A large reservoir will also require a less amount of idle pumping capacity for reserve than either the direct or the stand-pipe system. With respect to the convenient operation of pumps the stand-pipe or tank system is better than the direct; and in very small works it may effect a considerable saving by enabling them to be operated for but a part of the time.

As regards uniformity of pressure the gravity and reservoir systems are equally good. Direct pumping is the least desirable, but cannot be said to be entirely disadvantageous, as the pressure can be more readily modified to suit the requirements. Thus the high pressure necessary for fire extinguishment may be furnished only so long as it is needed, while at other times a much less pressure may be used, a matter of considerable economical importance. A similar advantage exists in the combined stand-pipe and direct-pressure system, with the additional one of a more flexible operation of the pumps at ordinary times.

237. Existing Works as Affecting Choice.—The problems of the future are mostly those of enlarging and improving present supplies. The kind and condition of the system already in existence will therefore often be of controlling influence in arriving at the best design for the new works. It will often happen that the present source has become polluted, and the question arises whether it be best to abandon it, to purify the water, or to use the water for other than domestic purposes. Combinations may thus be made between old and new systems or sources, and these may be either operated together or independently, each one serving a separate district, a separate zone of elevation, or a separate service.

238. The Dual System.—It has been proposed that where it becomes very expensive to furnish a water suitable for drinking purposes, a double system be adopted. One system would furnish water of the purest quality for drinking and culinary purposes, while the other would supply water for other domestic purposes, and for commercial and public use. The former would be perhaps relatively expensive, but as the quantity required would be only 5 or 6 gallons per head per day, the total expense would not be great. It would also often be much easier to find a good water in the quantities required for this purpose than for the entire supply. For example, a city of one million inhabitants would require only 5 or 6 million gallons per day of pure water, a quantity that could very often be obtained from good ground-water sources, such as would be entirely inadequate to supply all the water required. The larger quantity could then be obtained from cheaper sources, making the total expense in many cases less than under the usual single system.

The chief objection to this double system is the fact that there would be in all houses impure as well as pure water, and unless the former be very bad, unfit in fact for washing purposes, many persons would be careless or indifferent as to its use and thus the benefits of a pure water to a community would be very largely neutralized. That such would be the case is indicated by the experience at Lawrence, Mass., where, after the introduction of filter-beds to purify the city water-supply, a considerable number of cases of typhoid fever still remained, most of which were traced to mill operatives who used raw canal water at the mills, although city water was readily obtainable.

A more practical dual system would be one which would supply the purer water for all domestic purposes, and the other and cheaper water for certain commercial and public uses. Such a system is in use in a number of foreign cities with resulting economy. In Paris, for example, where the use of water for street cleaning is so great, a separate and cheap supply is used for this purpose. Special high-pressure fire systems are of great value in large cities, and since 1900 such systems have been installed in several places. See Art. 757 for further data. The merits of salt water for street sprinkling may make it advantageous in towns located on tide-water to construct a separate sea-water pipe system. It will also often happen that a number of large commercial consumers of water are so located that they may be economically supplied with cheap water through a separate system.

PRINCIPLES OF ECONOMIC CONSTRUCTION.

239. **The General Problem.**—In fixing upon a design the engineer must constantly keep before him the question of economy,—economy in the long run and generally speaking. The consideration of this question always involves matters relating to the future that can be only approximately determined, and on that account it is often very difficult to make a correct decision. Thus the cost of operation may, and often does, change materially; so also the interest rates, and the cost of material, and even the methods of construction.

The expense of any works to any community is made up of three parts: (1) first cost; (2) cost of operation and repairs; and (3) depreciation, or the cost of renewals not included under ordinary repairs. The problem of the engineer as regards economy is to secure a minimum sum total of these three items of expense. In addition he must usually consider the question of annual payment into a sinking fund.

240. **Methods of Comparing Cost.**—Different systems may be compared as to economy either by comparing the value of the capital invested and required to keep the plant in operation, or by comparing the annual expense, including the depreciation and interest on the investment. The former method is frequently employed in comparing designs of structures where the first cost is of relatively great importance, but for certain purposes it is important to look at the matter from the standpoint of annual charges, especially in connection with the financial management of the works, provision for payment of bonds, etc. Both methods should give the same relative result.

241. Method of Capitalization.—To correctly state the cost of a works in terms of capital we have: (1) the first cost; (2) the annual cost of operation and maintenance capitalized at the current rate of interest; (3) the capital which must be added to make good the depreciation. The last sum must be such an amount as will, when put at compound interest, provide a sum at the expiration of the life of the structure sufficient to renew it and also to leave a sum equal to the original amount for further future provision. These three sums will then equal an amount sufficient to construct, operate, and perpetually maintain the plant.

Many parts of a water-works system can be kept in perfect service indefinitely by the ordinary repairs. These are not subject to depreciation. Other parts must be renewed from time to time. What are considered as repairs, what as renewals, and what as new improvements will depend upon the method of keeping accounts in the particular water-works considered, but this detail will be left for subsequent discussion (Chapter XXIX).

That part of the cost represented by items (1) and (2) is readily stated and requires no further comments. The capital necessary to provide for depreciation may be determined as follows: Let $P =$ sum required; $C =$ cost of renewal, assumed equal to the first cost; $r =$ rate of interest; and $n =$ years of life of the structure. Then placing P at compound interest we must have

$$P(1 + r)^n = C + P,$$

whence

$$P = \frac{C}{(1 + r)^n - 1}. \quad \quad (1)$$

If $O =$ annual cost of operation and maintenance, we then have the total capitalized sum

$$S = C + \frac{O}{r} + P$$

$$= C + \frac{O}{r} + \frac{C}{(1 + r)^n - 1}. \quad \quad (2)$$

In permanent structures the term P drops out and the problem reduces to a comparison of the first cost plus the capitalized cost of operation, $C + \frac{O}{r}$, a very simple matter. With structures not permanent the term P must be considered, and this requires a knowledge of the durability of the various kinds of structures, and some judgment as to their continued usefulness independent of their durability.

As an example of the method of computation of the total capital required suppose that a certain structure costs $50,000; that the operating expenses are $5000 per year, and that the life of the structure is thirty years. If money is worth 5 per cent, the total capitalized cost will be, substituting in eq. (2),

$$S = 50,000 + \frac{5000}{.05} + \frac{50,000}{(1.05)^{30} - 1} = \$50,000 + \$100,000 + \$15,000$$
$$= \$165,000.$$

If this same structure would last forty years, the last term becomes $\frac{50,000}{(1.05)^{40} - 1} = \$8300$, and the total capital $158,300; if fifty years, the last term becomes $5200, and the total capital $155,200. It is to be noted from these figures that to make the structure last forty instead of thirty years would justify an initial expenditure of $15,000 — $8300 = $6700; to make it last fifty years, an additional expenditure of but $3100, and to make it last indefinitely, only $5200 more. Compared to first cost and cost of operation, it is thus evident that the question of extending the life of a plant by adding to the first cost, when that life is already twenty-five or thirty years, is a very minor consideration. This example shows also that no great accuracy is necessary in estimating the life of a plant when it is of a fairly permanent character.

Many other elements enter in to modify these mathematical results. For example, inconvenience of renewal, and reliability of operation, tend greatly to increase the value of a permanent structure; while future improvements in methods and processes tend in the opposite direction.

242. Method of Annual Expense.—The annual expense will be equal to the interest on the first cost, plus the cost of operation, plus the annual depreciation. It is evidently equal to the interest on the total capitalized cost as previously found, or to $Cr + O + Pr$. The last term of this expression may be called the *annual* depreciation, and by substituting from eq. (1) we have, letting D = annual depreciation,

$$D = Pr = \frac{Cr}{(1 + r)^n - 1}, \quad \ldots \ldots \quad (3)$$

in which C = cost of renewal (= first cost), r = rate of interest, and n = life of plant in years. The annual *rate* of depreciation per unit of cost $= \frac{r}{(1 + r)^n - 1}$.

The annual depreciation may be otherwise expressed as the annual payment which if placed at compound interest will accumulate a fund equal to C at the end of n years. Formula (3) assumes the payment to be made at the end of each year. Table No. 35 gives the annual payment required to accumulate $1.00 at the end of various periods of time and at various rates of interest, calculated according to eq. (3). From this the annual rate of depreciation can be directly determined. Thus if the life of a structure is estimated at thirty years and the interest rate is 3 per cent, the annual depreciation will be 2.1 per cent of the first cost.

Provision for a Sinking Fund. — Where bonds are issued to cover the first cost of a works it is usually considered good policy to provide a sinking fund for the entire liquidation of the debt at the end of some long period, such as thirty or forty years. While this question does not strictly enter into a determination of the true economy of a structure, yet the matter of required annual payment for several years to come is usually of the most vital importance to the present generation. To what extent a sinking fund should be considered in comparative estimates is not easy to say. It depends on what the policy of the water department is likely to be. Probably very few departments provide funds to fully cover depreciation, and also a sinking fund. The latter is indeed likely to be the chief consideration, and the matter of renewals left to the future. In some cases, especially those relating to the improvement of supplies of large cities, it will be well to make full allowance for both depreciation and sinking fund, but in most cases it would appear to be a fairer basis of comparison and a sufficient precaution against unforeseen contingencies, to omit the sinking fund and to consider the permanent parts of the plant as subject to a slight depreciation. If account is to be taken of annual payments into a sinking fund, the amount necessary to accumulate any given sum can easily be determined from Table No. 35.

243. Depreciation of Structures. — The rate of depreciation, or the life of various parts of a water-works, is very various. With certain parts, such as dams, masonry aqueducts, and the like, the life with ordinary repairs is practically indefinite. Brick and stone buildings are subject to a small depreciation. Pipes buried in the ground have a life not yet well determined and which depends much on the character of the soil. Cast iron, properly coated, appears to depreciate very slightly, if at all. Its life is variously estimated at from fifty to one hundred years or more. Probably seventy-five years would usually be a safe figure. Carefully protected riveted pipe will also have a very long life,

TABLE NO. 35.

AMOUNTS NECESSARY TO INVEST AT VARIOUS RATES OF COMPOUND INTEREST TO ACCUMULATE $1.00 AT THE END OF VARIOUS PERIODS OF TIME. THE PAYMENT IS ASSUMED TO BE MADE AT THE END OF EACH YEAR.

Year.	Rates of Interest in Per cent.				
	2 Per cent.	2½ Per cent.	3 Per cent.	3½ Per cent.	4 Per cent.
1	$1.00000	$1.00000	$1.00000	$1.00000	$1.00000
2	0.49505	0.49382	0.49261	0.49140	0.49020
3	0.32675	0.32514	0.32353	0.32193	0.32035
4	0.24262	0.24082	0.23902	0.23725	0.23550
5	0.19218	0.19025	0.18835	0.18648	0.18463
6	0.15853	0.15655	0.15451	0.15267	0.15077
7	0.13451	0.13250	0.13051	0.12854	0.12661
8	0.11651	0.11458	0.11246	0.11048	0.10853
9	0.10252	0.10046	0.09843	0.09644	0.09449
10	0.09133	0.08926	0.08723	0.08524	0.08329
11	0.08218	0.08011	0.07808	0.07609	0.07411
12	0.07456	0.07250	0.07047	0.06848	0.06655
13	0.06812	0.06605	0.06403	0.06205	0.06015
14	0.06260	0.06054	0.05853	0.05657	0.05467
15	0.05782	0.05577	0.05380	0.05183	0.04994
16	0.05365	0.05160	0.04961	0.04768	0.04582
17	0.04997	0.04793	0.04595	0.04404	0.04220
18	0.04670	0.04470	0.04271	0.04082	0.03899
19	0.04378	0.04176	0.03981	0.03794	0.03614
20	0.04116	0.03915	0.03722	0.03536	0.03356
21	0.03878	0.03678	0.03487	0.03304	0.03128
22	0.03663	0.03465	0.03270	0.03094	0.02920
23	0.03467	0.03270	0.03081	0.02902	0.02731
24	0.03287	0.03091	0.02905	0.02727	0.02560
25	0.03122	0.02928	0.02743	0.02567	0.02401
26	0.02970	0.02777	0.02594	0.02421	0.02257
27	0.02821	0.02640	0.02460	0.02285	0.02124
28	0.02699	0.02509	0.02329	0.02161	0.02000
29	0.02578	0.02389	0.02211	0.02045	0.01888
30	0.02465	0.02278	0.02102	0.01937	0.01783
31	0.02360	0.02174	0.02000	0.01837	0.01686
32	0.02261	0.02077	0.01905	0.01744	0.01595
33	0.02169	0.01986	0.01816	0.01657	0.01511
34	0.02082	0.01901	0.01732	0.01576	0.01431
35	0.02000	0.01821	0.01654	0.01499	0.01358
36	0.01923	0.01745	0.01580	0.01426	0.01288
37	0.01851	0.01674	0.01511	0.01361	0.01224
38	0.01781	0.01607	0.01450	0.01298	0.01163
39	0.01717	0.01544	0.01383	0.01240	0.01106
40	0.01655	0.01484	0.01326	0.01183	0.01052
41	0.01597	0.01427	0.01271	0.01130	0.01002
42	0.01542	0.01373	0.01219	0.01080	0.00954
43	0.01489	0.01322	0.01170	0.01033	0.00909
44	0.01439	0.01273	0.01123	0.00987	0.00866
45	0.01391	0.01226	0.01080	0.00945	0.00826
46	0.01345	0.01183	0.01036	0.00905	0.00788
47	0.01302	0.01141	0.00996	0.00866	0.00752
48	0.01260	0.01097	0.00957	0.00830	0.00718
49	0.01220	0.01066	0.00918	0.00796	0.00686
50	0.01182	0.01026	0.00886	0.00763	0.00655

TABLE NO. 35. — *Continued.*

Year.	Rates of Interest in Per cent.				
	2 Per cent.	2½ Per cent.	3 Per cent.	3½ Per cent.	4 Per cent.
51	$0.01146	$0.00991	$0.00853	$0.00732	$0.00626
52	0.01111	0.00957	0.00822	0.00703	0.00598
53	0.01077	0.00925	0.00791	0.00674	0.00572
54	0.01045	0.00894	0.00763	0.00647	0.00547
55	0.01014	0.00866	0.00735	0.00622	0.00523
56	0.00984	0.00837	0.00709	0.00597	0.00501
57	0.00956	0.00810	0.00683	0.00573	0.00479
58	0.00929	0.00784	0.00659	0.00550	0.00458
59	0.00902	0.00759	0.00635	0.00529	0.00439
60	0.00877	0.00735	0.00613	0.00507	0.00420
61	0.00852	0.00712	0.00592	0.00489	0.00402
62	0.00828	0.00690	0.00571	0.00471	0.00385
63	0.00806	0.00669	0.00552	0.00453	0.00369
64	0.00784	0.00648	0.00533	0.00435	0.00353
65	0.00763	0.00628	0.00515	0.00418	0.00339
66	0.00742	0.00609	0.00497	0.00403	0.00325
67	0.00722	0.00591	0.00480	0.00388	0.00311
68	0.00704	0.00573	0.00465	0.00373	0.00298
69	0.00685	0.00556	0.00449	0.00359	0.00286
70	0.00667	0.00540	0.00434	0.00346	0.00274
71	0.00649	0.00524	0.00419	0.00333	0.00263
72	0.00633	0.00508	0.00405	0.00321	0.00252
73	0.00616	0.00493	0.00392	0.00309	0.00242
74	0.00601	0.00479	0.00379	0.00298	0.00232
75	0.00585	0.00465	0.00367	0.00287	0.00223
76	0.00571	0.00452	0.00355	0.00277	0.00214
77	0.00557	0.00439	0.00343	0.00266	0.00205
78	0.00542	0.00426	0.00333	0.00257	0.00197
79	0.00529	0.00414	0.00321	0.00247	0.00189
80	0.00516	0.00403	0.00311	0.00239	0.00181
81	0.00503	0.00391	0.00301	0.00230	0.00174
82	0.00491	0.00380	0.00292	0.00222	0.00167
83	0.00479	0.00369	0.00282	0.00214	0.00160
84	0.00468	0.00359	0.00273	0.00206	0.00154
85	0.00456	0.00349	0.00264	0.00199	0.00148
86	0.00445	0.00340	0.00256	0.00192	0.00142
87	0.00434	0.00330	0.00248	0.00185	0.00136
88	0.00424	0.00321	0.00240	0.00178	0.00131
89	0.00414	0.00312	0.00233	0.00172	0.00126
90	0.00405	0.00304	0.00225	0.00166	0.00121
91	0.00395	0.00295	0.00219	0.00160	0.00116
92	0.00386	0.00287	0.00212	0.00154	0.00111
93	0.00377	0.00280	0.00205	0.00149	0.00107
94	0.00368	0.00271	0.00199	0.00144	0.00103
95	0.00360	0.00265	0.00193	0.00138	0.00099
96	0.00351	0.00258	0.00187	0.00134	0.00095
97	0.00343	0.00251	0.00181	0.00129	0.00091
98	0.00336	0.00244	0.00175	0.00125	0.00087
99	0.00328	0.00237	0.00170	0.00121	0.00084
100	0.00320	0.00231	0.00165	0.00116	0.00081

but not so long as cast iron. Large, well-made machinery may have a life of thirty or forty years, or perhaps longer, but improvements in the design of machinery would usually limit its useful life to twenty-five or thirty years. The life of boilers will usually range from fifteen to twenty years. Light machinery will have a life of fifteen or twenty years, and in some cases of great wear even less than this.

In many problems, especially in the appraisal of water-works properties, the determination of the present value of a depreciated plant is necessary. Considering the plant as one to be indefinitely maintained on a steady financial basis, the present worth of a depreciated plant must be such a value as will, when added to the present value of the annuity set aside for its depreciation, just equal its first cost; that is, neglecting fluctuations in cost, the present value of the plant and that of the annuity set aside for its replacement should at all times be a constant quantity. This is what is usually called the "sinking fund basis." The present worth can readily be obtained from Table 35. Thus, suppose the life of a structure be forty years, and the rate of interest three per cent. Required, the value of the plant twenty-five years after its construction. From the table, the annuity required to cover depreciation is $0.01326 per dollar for forty years. The annuity required to produce $1.00 at the end of twenty-five years is $0.02743. The value of the annuity of $0.01326 at the end of twenty-five years is therefore equal to .01326 ÷ .02743 = $.483. This amount may be considered the amount of depreciation per dollar at the end of twenty-five years, and the present worth is therefore equal to $1.00 — .483 = $.517 per dollar, or 51.7 per cent of the first cost. For further discussion see Chapter XXIX.

244. **Provision for the Future.** — In the preceding discussion the matter of permanence only has been mentioned. Another very important element needs to be considered at all points of the design, namely, what provision is it economical and expedient to make for the future? The questions of future population and consumption have already been discussed, and it is here assumed that these have been settled. There remains then to be determined the capacity of the various parts of the work. Obviously, those portions of the works that can be added to as easily at one time as another, such as pumps, filters, etc., should be built for but little more than present requirements. Future requirements, however, should be regarded rather more than strict economy would suggest owing to delays and difficulties in securing appropriations, etc. Other parts, such as pump-houses, land for filter-beds, pumping-mains, etc., should be designed for a longer time in the future, as these are obtained more cheaply per unit

of capacity at first than by duplication. Still other parts, such as masonry conduits, dams, tunnels, etc., should be built for a still greater capacity, as the extra cost and depreciation will both be relatively small.

The question to be answered is, how much will it pay to spend now in securing capacity in the various parts of the works in order to avoid the expenditure of a larger sum in the future? It is a simple problem in compound interest, and is solved by determining what sum of money placed at compound interest will amount to the sum saved at the time the increased capacity becomes necessary. If A = amount saved at the end of n years by incurring a certain expense B at the present time, and r = rate of interest, then in order that the one plan may just balance the other we have $A = B(1 + r)^n$, or

$$B = \frac{A}{(1 + r)^n}. \qquad (4)$$

If the structure under consideration is subject to depreciation, the amount A must be figured on the basis of the actual value of the depreciated plant at the given time in the future.

To arrive at a proper solution it is necessary to take into consideration many other elements than merely the mathematical one of interest. All parts of a works should be made to correspond, or be so designed that future enlargements of one part will correspond to the more permanent structures of another part. Thus a metal portion of a conduit may well be made just one-half or one-third the capacity of the masonry portion of the same, the former providing for ten or fifteen years in the future, while the latter provides for thirty or forty years. There must also be considered the question of the financial ability of the community to incur a large expense or a large annual payment, the debt limit, etc. These factors will often necessitate the construction of a plant which might not be the ideal one with unlimited capital at hand.

245. Estimates of Cost.—To be of much value an estimate of cost must be made from classified estimates of the different kinds of work to be done and material to be furnished. In work of the nature of most of that involved in water-works construction the risk is not great, and close estimates can be made of the cost of various classes of work by inquiry of local contractors and by comparison with the cost of similar work already executed. The contract prices quoted in the technical papers will be of assistance in this connection. The analysis of the cost of various classes of masonry given in Baker's "Masonry Construction" will also be found useful. Prices of metal parts, such as pipes, valves, machinery, etc., vary so greatly from time to time that any statement of them would be of little permanent value. Approximate prices can always be obtained on short notice by correspondence with manufacturers.

Although the classified estimate is the only reliable way of estimating the cost of a works, yet it is very useful to an engineer, when considering the possibilities of different schemes, to have in mind certain rough general figures of cost of the different parts of a system. Such, for example, as the cost of small pipe systems per mile, cost of reservoirs of different kinds per 1000 or 1,000,000 gallons capacity, cost of filters per million gallons capacity, etc. In what follows such general figures of the cost of works have in many cases been given, but it must be remembered that they can be used only in making rough preliminary estimates.

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CHAPTER XII.

HYDRAULICS.

246. Purpose of the Chapter.—In the present chapter it is proposed to give in as brief a form as possible such hydraulic formulas as are of frequent use in the design of water-works, together with diagrams and tables of coefficients based upon the latest and most reliable experiments.

247. Units of Measure.—The unit of length most frequently used in hydraulics is the foot. The unit of volume is the cubic foot or the United States gallon. The unit of time usually employed in hydraulic formulas is the second, but in many water-supply problems the minute, the hour, and the day are also often used. The unit of weight is the pound, and that of energy the foot-pound.

1 U. S. gallon = 231 cubic inches = 0.1337 cubic foot;

1 cubic foot = 7.481 U. S. gallons;

1.2 U. S. gallons = 1 imperial gallon.

248. Notation.—The following general notation will be used in the present chapter without further explanation:

w = weight of a cubic foot of water assumed equal to 62.5 pounds;

g = acceleration of gravity = 32.2 feet per second per second;

h = head of water;

p = pressure of water;

r = hydraulic mean radius;

s = sine of slope of hydraulic grade-line or of free water-surface.

Q = rate of discharge or flow;

v = velocity.

249. Weight of Water.—The weight of distilled water at different temperatures is given in Table No. 36.

The weight of ordinary water is greater than that of distilled water on account of the impurities contained. For ordinary purposes a cubic foot of fresh water may be taken equal to 62.5 pounds. Sea-water will weigh about 64 pounds per cubic foot.

TABLE NO. 36.

WEIGHT OF DISTILLED WATER.

Temperature, Fahrenheit.	Weight, Pounds per Cubic Foot.	Temperature, Fahrenheit.	Weight, Pounds per Cubic Foot.
32°	62.42	140°	61.39
39.3	62.424	160	61.01
60	62.37	180	60.59
80	62.22	200	60.14
100	62.00	212	59.84
120	61.72		

250. Pressure of the Atmosphere.—In problems pertaining to the operation of suction-pipes it is important to know the atmospheric pressure corresponding to various elevations above sea-level. Data pertaining to this point are given in Table No. 37 in terms both of mercury barometer and of water barometer. The figures given are average values.

TABLE NO. 37.

ATMOSPHERIC PRESSURE AT DIFFERENT ELEVATIONS.

Elevation above Sea-level Feet.	Pressure in Pounds per Square Inch.	Height of Mercury Barometer. Inches.	Height of Water Barometer. Feet.
0	14.7	30.00	34.0
500	14.5	29.47	33.3
1,000	14.2	28.94	32.8
2,000	13.7	27.92	31.6
4,000	12.7	25.98	29.4
6,000	11.8	24.18	27.4
8,000	11.0	22 50	25.5
10,000	10.3	20.93	23.7

251. Vapor Tension of Water.—Where the temperature of the water is high the elevation of the water barometer will be considerably reduced below that given in the previous table on account of the pressure of the water vapor. Table No. 38 gives values of this vapor tension or pressure in feet of head for various temperatures.

252. Pressure of Water.—(1) *Pressure at a Point.*—The pressure of water per unit of area at a distance h below the free surface is

$$p = wh. \qquad (1)$$

TABLE NO. 38.

VAPOR TENSION OF WATER.

Temperature, Fahrenheit.	Pressure in Pounds per Square Inch.	Pressure in Feet of Water.
32°	.09	.21
40	.12	.28
60	.26	.60
80	.50	1.15
100	.95	2.19
120	1.69	3.91
140	2.89	6.68
160	4.74	11.0
180	7.53	17.4
200	11.56	26.7
212	14.70	34.0

If h is expressed in feet, and p in pounds per square inch, we have

$$p = 0.434h, \qquad (2)$$

and

$$h = 2.304p. \qquad (3)$$

Pressures are very commonly stated in terms of the head h, in which case h is called the *pressure-head.*

(2) *Pressure on a Surface.*—The pressure of water on a plane surface is always normal to that surface. The amount of the pressure is

$$P = wAh, \qquad (4)$$

where A = total area of surface, and h = head of water at its centre of gravity. The component of the pressure in any given direction is equal to the normal pressure upon that projection of the given surface at right angles to the given direction.

(3) *Centre of Pressure.*—The centre of pressure upon a submerged plane surface is given by the formula

$$y = \frac{I}{S}, \qquad (5)$$

where y = distance of centre of pressure from the intersection of the plane with the water-surface; and I and S are respectively the moment of inertia and the statical moment of the area about this intersection.

(4) *Bursting-pressure in Pipes.*—The bursting-pressure per lineal unit in a pipe of diameter d, due to a water-pressure p, is

$$P_1 = pd. \qquad (6)$$

If t = thickness of pipe, the resulting tensile stress per unit area is

$$a_1 = \frac{P_1}{2t} = \frac{pd}{2t}. \qquad (7)$$

In a closed cylinder under pressure the force tending to rupture it transversely is

$$P_2 = \frac{p\pi d^2}{4}, \qquad (8)$$

and the stress per unit area is

$$a_2 = \frac{P_2}{\pi dt} = \frac{pd}{4t}, \qquad (9)$$

or one-half that given by equation (7). This is also the stress in a sphere of diameter d.

Formulas (6) to (9) assume that the water in the pipe or cylinder is under a uniform pressure, or, in other words, that the diameter of the pipe is small as compared with the pressure head of the water.

FLOW OF WATER THROUGH ORIFICES.

253. Form and Proportion of Orifices.—In making use of an orifice for measuring water it is desirable that it be made in such a way as to be in effect an orifice in a thin plate; that is to say, it should be so arranged that the water in passing out will touch the inner edge only. Furthermore, it is important that the inner edge of the orifice shall be flush with the inner surface of the tank, and that the latter should extend as a plane surface for a considerable distance in each direction from the orifice. To secure complete contraction the orifice should be placed at a distance from the sides and the bottom of the tank not less than three times the width of the orifice; and in order that the effect of the velocity of approach may be inappreciable the area of the orifice should not exceed one-twentieth of the cross-section of the tank.

254. Flow through Small Orifices.—The theoretical velocity of water flowing through an orifice is, at the point of contraction, $v = \sqrt{2gh}$, where h = head of water at centre of orifice. The actual velocity is a little less than the theoretical, and we have

$$v = c\sqrt{2gh}, \qquad (10)$$

in which c is a coefficient, found by experiment to be equal to .97 to .98. The ratio of the area of the cross-section of the contracted vein to the area of the orifice is called the coefficient of contraction. It ranges from .6 to .7, but usually has a value of from .62 to .64. Finally, if A = area of orifice, the discharge is

$$Q = c_d A \sqrt{2gh}, \qquad (11)$$

in which c_d = coefficient of discharge = about .62. Many experiments have been made to determine the value of c_d directly.

255. Large Rectangular Vertical Orifices.—The discharge is given by the formula

$$Q = c \cdot \tfrac{2}{3} b \sqrt{2g}(h_2^{\frac{3}{2}} - h_1^{\frac{3}{2}}), \qquad (12)$$

in which c is again a coefficient, b = width, h_2 = head on lower edge of orifice, and h_1 = head on upper edge. If the head h at the centre of the orifice is greater than four times the height of the orifice, then it is sufficiently exact to write

$$Q = c \cdot bd \sqrt{2gh}, \qquad (13)$$

in which d = vertical dimension of the orifice. For square orifices $b = d$, and if h is greater than $4d$, then

$$Q = c \cdot d^2 \sqrt{2gh}. \qquad (14)$$

Table No. 39 contains values of coefficients for square orifices as deduced by Hamilton Smith from the results of many experiments.

TABLE NO. 39.

COEFFICIENTS FOR SQUARE VERTICAL ORIFICES (SMITH).

Head, h, in Feet.	Side of the Square in Feet.						
	0.02	0.04	0.07	0.1	0.2	0.6	1.0
0.4	....	0.643	0.628	0.621			
0.6	0.660	.636	.623	.617	0.605	0.598	
0.8	.652	.631	.620	.615	.605	.600	0.597
1.0	.648	.628	.618	.613	.605	.601	.599
1.5	.641	.622	.614	.610	.605	.602	.601
2.0	.637	.619	.612	.608	.605	.604	.602
2.5	.634	.617	.610	.607	.605	.604	.602
3	.632	.616	.609	.607	.605	.604	.603
4	.628	.614	.608	.606	.605	.603	.602
6	.623	.612	.607	.605	.604	.603	.602
8	.619	.610	.606	.605	.604	.603	.602
10	.616	.608	.605	.604	.603	.602	.601
20	.606	.604	.602	.602	.602	.601	.600
50	.602	.601	.601	.600	.600	.599	.599
100	.599	.598	.598	.598	.598	.598	.598

256. Circular Vertical Orifices.—The discharge from large circular vertical orifices is given by the formula

$$Q=c\,.\,\tfrac{1}{4}\pi d^2\sqrt{2gh}\left(1-\frac{1}{128}\frac{d^2}{h^2}-\frac{5}{16,384}\frac{d^4}{h^4}-\frac{105}{4,194,304}\frac{d^6}{h^6},\text{ etc.}\right), \quad (15)$$

where $d =$ diameter of orifice, and $h =$ head of water at its centre. If h is greater than $3d$, then we may write

$$Q = c\,.\,\tfrac{1}{4}\pi d^2\sqrt{2gh}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (16)$$

Table No. 40 gives values of c as deduced by Hamilton Smith.

TABLE NO. 40.

COEFFICIENTS FOR CIRCULAR VERTICAL ORIFICES (SMITH).

Head, h, in Feet.	Diameter of Orifice in Feet.						
	0.02	0.04	0.07	0.10	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	0.590
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

FLOW OF WATER OVER WEIRS.

257. Sharp-crested Weirs. — For measuring the flow of a small stream a weir is very often employed. Such weirs are usually rectangular and are made sharp-crested, that is, of a form such that the water touches the inside edge only. The back of the weir should be a vertical plane surface. If the ends of the weir are placed at some distance from the sides of the channel, the stream of water will be contracted laterally. For complete contractions this distance should be at least three times the height of the weir. If the ends are flush with the sides of the channel, there will be no end contractions.

The depth of the water on a weir should be measured sufficiently far above the weir to eliminate the effect of the surface curve.

The formula for discharge from a rectangular weir may be written from equation (12) by putting $h_1 = 0$. It is

$$Q = c \cdot \tfrac{2}{3} l \sqrt{2g} H^{\frac{3}{2}}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (17)$$

where l = length of weir and H = height of water on the weir. If the channel is small, the velocity of approach will have an appreciable effect upon the discharge. If h = head due to this velocity $= \dfrac{v^2}{2g}$, then this factor is commonly taken account of by the use of the equation

$$Q = c \cdot \tfrac{2}{3} l \sqrt{2g} (H + nh)^{\frac{3}{2}}, \quad . \quad . \quad . \quad . \quad . \quad (18)$$

in which n is a coefficient varying from 1 to 1.5. The velocity of approach can be estimated with sufficient accuracy by first determining the value of Q with the term nh omitted, then using this value of Q to determine v, and then a more accurate value of Q, etc. From a careful analysis of the experiments of Francis, Fteley and Stearns, and others, Hamilton Smith has adopted values for n of 1.4 for weirs with end contractions, and $1\frac{1}{3}$ for weirs with contractions suppressed. From his study of the experiments referred to he has also derived the values for the coefficient c as given in Tables Nos. 41 and 42.

TABLE NO. 41.

COEFFICIENTS FOR CONTRACTED WEIRS (SMITH).

Effective Head in Feet.	Length of Weir in Feet.						
	0.66	1	2	3	5	10	19
0.1	0.632	0.639	0.646	0.652	0.653	0.655	0.656
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

TABLE NO. 42.

COEFFICIENTS FOR SUPPRESSED WEIRS (SMITH).

Effective Head in Feet.	Length of Weir in Feet.						
	19	10	7	5	4	3	2
0.1	0.657	0.658	0.658	0.659			
0.15	.643	.644	.645	.645	0.647	0.649	0.652
0.2	.635	.637	.637	.638	.641	.642	.645
0.25	.630	.632	.633	.634	.636	.638	.641
0.3	.626	.628	.629	.631	.633	.636	.639
0.4	.621	.623	.625	.628	.630	.633	.636
0.5	.619	.621	.624	.627	.630	.633	.637
0.6	.618	.620	.623	.627	.630	.634	.638
0.7	.618	.620	.624	.628	.631	.635	.640
0.8	.618	.621	.625	.629	.633	.637	.643
0.9	.619	.622	.627	.631	.635	.639	645
1.0	.619	.624	.628	.633	.637	.641	.648
1.2	.620	.626	.632	.636	.641	.646	
1.4	.622	.629	.634	.640	.644		
1.6	.623	.631	.637	.642	.647		

Mr. James B. Francis from his elaborate series of experiments at Lowell * on a weir 10 feet long, and operating under heads of 0.4 to 1.6 feet derived the formula for weirs without end contractions,

$$Q = 3.33 l H^{\frac{3}{2}}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (19)$$

and with contractions

$$Q = 3.33(l - 0.2H)H^{\frac{3}{2}} \quad . \quad . \quad . \quad . \quad . \quad (20)$$

If there be but one contraction, $0.1H$ is to be used instead of $0.2H$. To take account of the velocity of approach, the formulas become

$$Q = 3.33 l[(H + h)^{\frac{3}{2}} - h^{\frac{3}{2}}], \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (21)$$

and

$$Q = 3.33(l - 0.2H)[(H + h)^{\frac{3}{2}} - h^{\frac{3}{2}}], \quad . \quad . \quad . \quad (22)$$

in which $h = \frac{v^2}{2g}$. In these formulas the unit of length is the foot.

The most elaborate series of experiments on weirs ever made is that which has been carried out by Bazin, and which is reported in the Annals des Ponts et Chaussées for the years 1888–98. The formula deduced by him for sharp-crested weirs with no end contractions is in foot units

$$Q = \left(.405 + \frac{.00984}{h}\right)\left(1 + .55\left(\frac{h}{p + h}\right)^2\right) lh \sqrt{2gh}, \quad . \quad . \quad (23)$$

* Lowell Hydraulic Experiments. New York, 1871.

in which p = height of weir above the bottom of the channel, and h is the actually observed head on the weir. The velocity of approach is taken account of in the formula. This formula gives slightly larger values for the discharge than the Francis formula.*

258. Submerged Weirs.—As the result of an analysis of the experiments of Francis and of Fteley and Stearns, Mr. Herschel † adopts the following formula for discharge, for submerged, sharp-crested weirs without end contractions:

$$Q = 3.33l(nH)^{\frac{3}{2}}; \quad \ldots \ldots \quad (24)$$

where l = length of weir;

H = height on the weir on the up-stream side;

n = a coefficient, depending on the ratio of the head on the down-stream side H' to the head H.

The values of n are given in Table No. 43.

TABLE NO. 43.

VALUES OF n FOR SUBMERGED WEIRS. (HERSCHEL.)

$\frac{H'}{H}$	n	$\frac{H'}{H}$	n	$\frac{H'}{H}$	n	$\frac{H'}{H}$	n
.00	1.000	.20	0.985	.45	0.912	.70	0.787
.02	1.006	.25	0.973	.50	0.892	.75	0.750
.05	1.007	.30	0.959	.55	0.871	.80	0.703
.10	1.005	.35	0.944	.60	0.846	.90	0.574
.15	0.996	.40	0.929	.65	0.819	1.00	0.000

259. Weirs of Various Forms.—In many cases it is desirable to determine the flow of a stream by measurements taken of the height of water flowing over some dam or weir; and, on the other hand, in the design of waste-weirs some method of estimating their capacity is essential. The law of flow over such weirs is very complicated, and the only accurate way of determining the constants for any particular case is by means of experiments on a section of the same form as the one in question. If this is impossible, the best substitute for it is to use constants which have been determined for a weir agreeing as closely in form as may be to the one under consideration.

Here again Bazin's work is of the greatest value. He employed in his experiments weirs of a great variety of form. The heights used were one metre and one and one-half metres; and the heads employed reached a maximum of about one-half metre. The end contractions

* See a valuable paper by G. W. Rafter, including exhaustive discussion, in Trans. Am. Soc. C. E., Dec. 1900.

† Trans. Am. Soc. C. E., 1885, XIV. p. 194.

were suppressed in all cases, and below the weir the sides of the channel were continued so that in general there was not perfectly free access of air beneath the sheet of water. In Table No. 44 are given values of the coefficient C in the formula $Q = Clh^{\frac{3}{2}}$, for several forms of weirs, as deduced from Bazin's experiments. The head h is in all

TABLE NO. 44.

VALUES OF THE COEFFICIENT C IN THE FORMULA $Q=Clh^{\frac{3}{2}}$, FROM BAZIN'S EXPERIMENTS.

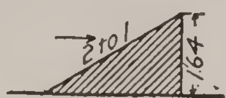
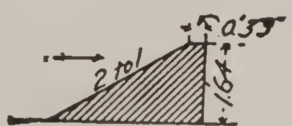
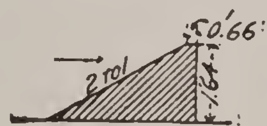
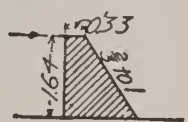
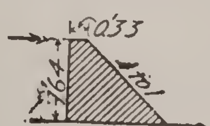
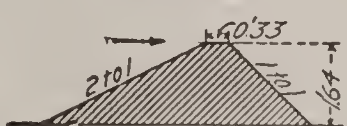
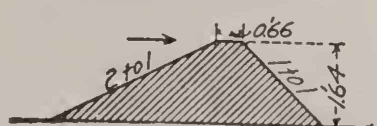
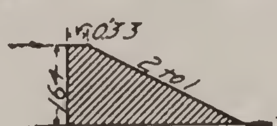
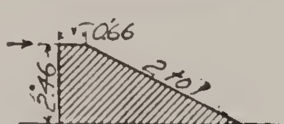
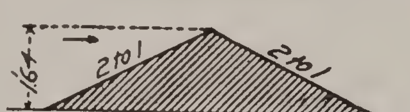
No.	Section of Weir.	Height on Weir in Feet.					
		0.4	0.6	0.8	1.0	1.2	1.4
1		4.06	4.20	4.20	4.18	4.15	4.12
2		3.40	3.70	3.88	3.98	4.05	4.10
3		3.18	3.37	3.54	3.68	3.80	3.88
4		3.26	3.70	4.06	4.31	4.00	3.80
5		3.41	3.74	4.00	4.20	4.18	4.24
6		3.20	3.68	4.02	4.26	4.42	4.20
7		3.47	3.72	3.87	4.01	4.13	4.23
8		3.20	3.38	3.56	3.74	3.85	3.95
9		3.14	3.44	3.67	3.83	3.83	3.86
10		2.68	2.90	3.13	3.35	3.52	3.64
11		3.74	3.81	3.88	3.93	3.98	4.02
12		3.86	3.87	3.92	4.02	4.08	4.15

TABLE NO. 44.—*Continued.*

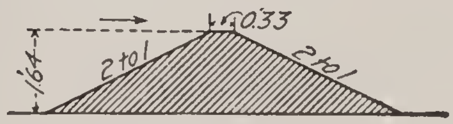
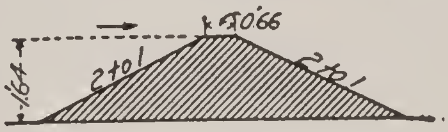
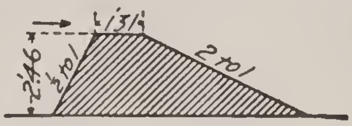
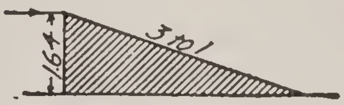
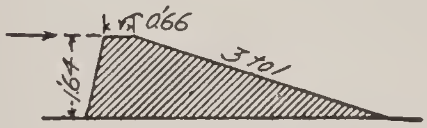
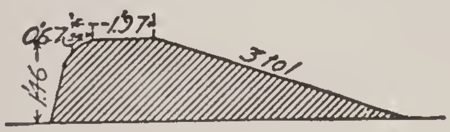
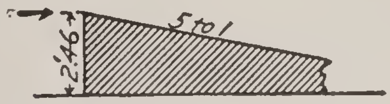
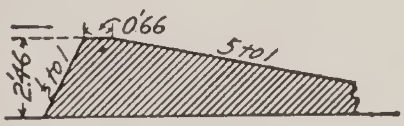
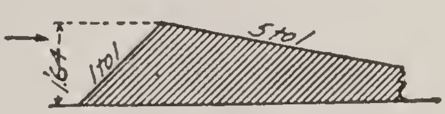
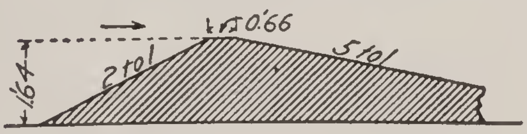
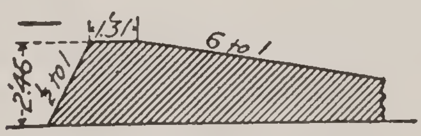
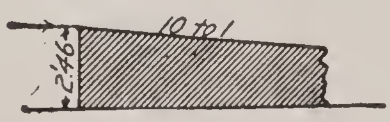
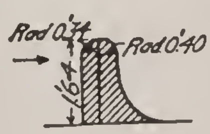
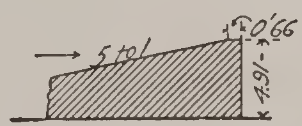
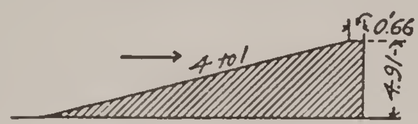
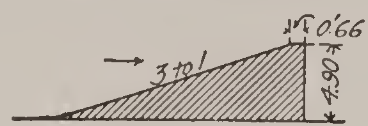
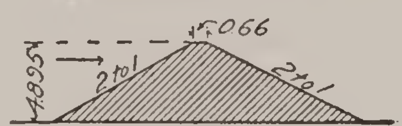
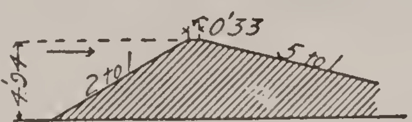
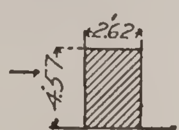
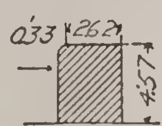
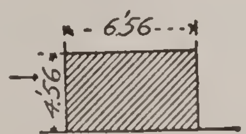
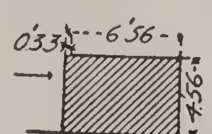
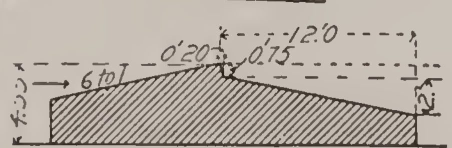
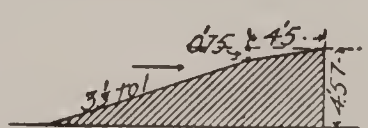
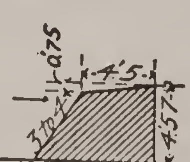
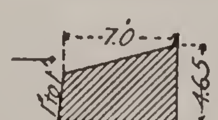
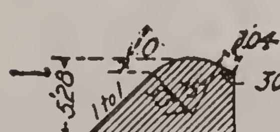
No.	Section of Weir.	Height of Weir in Feet.					
		0.4	0.6	0.8	1.0	1.2	1.4
13		3.44	3.64	3.80	3.94	4.04	
14		3.18	3.34	3.49	3.65	3.79	3.90
15		2.77	2.83	2.92	3.05	3.16	3.29
16		3.08	3.33	3.49	3.54	3.60	3.64
17		3.24	3.40	3.60	3.76	3.86	3.97
18		3.04	3.12	3.19	3.26	3.36	3.48
19		3.06	3.10	3.14	3.18	3.20	3.23
20		2.83	3.00	3.15	3.27	3.38	3.47
21		3.56	3.54	3.57	3.62	3.66	3.70
22		3.12	3.23	3.34	3.47	3.56	3.64
23		2.80	2.85	2.90	2.97	3.05	3.14
24		2.86	2.88	2.93	2.96	2.98	3.00
25		3.40	3.76	4.08	4.36	4.56	4.56

TABLE NO. 45.

VALUES OF THE COEFFICIENT C IN THE FORMULA $Q = Clh^{\frac{3}{2}}$, FROM THE CORNELL UNIVERSITY EXPERIMENTS.

Number.	Section of Weir.	Height on Weir in Feet.							
		1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0
1		3.51	3.37	3.33	3.31	3.29	3.23	3.16	3.14
2		3.51	3.37	3.33	3.31	3.29	3.25	3.20	3.21
3		......	3.76	3.68	3.68	3.70	3.75	3.83	
4		......	3.68	3.71	3.81	3.90	4.00	4.06	
5		3.81	3.61	3.68	3.65	3.72	3.80	3.93	
6		......	2.85	2.89	3.03	3.16	3.30	3.50	3.71
7		......	3.12	3.20	3.31	3.45	3.63	3.78	3.92
8		......	......	2.45	2.49	2.51	2.56	2.59	2.67
9		2.93	2.85	2.82	2.90	2.90	2.92	2.95	
10		3.29	3.23	3.34	3.42	3.45	3.47	3.52	
11		3.51	3.52	3.47	3.55	3.62	3.68	3.72	
12		3.59	3.62	3.54	3.52	3.57	3.55	3.69	
13		3.81	3.61	3.57	3.63	3.62	3.67	3.71	3.80
14		3.37	3.20	3.50	3.60	3.70	3.84	3.95	

cases the actually observed head on the weir. The coefficients as given were obtained from curves plotted from Bazin's results.*

More recently Mr. G. W. Rafter and Prof. G. S. Williams have made for the United States Board of Engineers on Deep Waterways an important series of experiments at the Cornell University Hydraulic Laboratory.† In these experiments the height of water on the weir was carried to a value of about 5 feet, the weir being 6.58 feet long and from 4.6 to 4.9 feet above the bottom of the channel. Free access for the air beneath the sheet of water was provided. In Table No. 45 are given coefficients for several of the forms experimented on, for use in the formula $Q = Clh^{\frac{3}{2}}$, where h is again the actually observed head on the weir. These values were deduced from the data and discharge curves given in Prof. Williams' discussion of Mr. Rafter's paper above referred to.

It is important to note that, on account of the difference in conditions with respect to the entrance of air below the sheet of water in the two series of experiments above described, it appears that in those forms having a steep down-stream face the coefficients from Bazin's experiments are much the higher. For slopes greater than one-to-one the differences are small. Note also that weirs with wide crests give lower coefficients than those with narrow crests; also that low weirs give higher coefficients than high weirs.

FLOW OF WATER THROUGH PIPES.

260. General Relations between Velocity and Pressure.—Let $ABCD$, Fig. 32, be any pipe in which there occurs a steady flow of water from

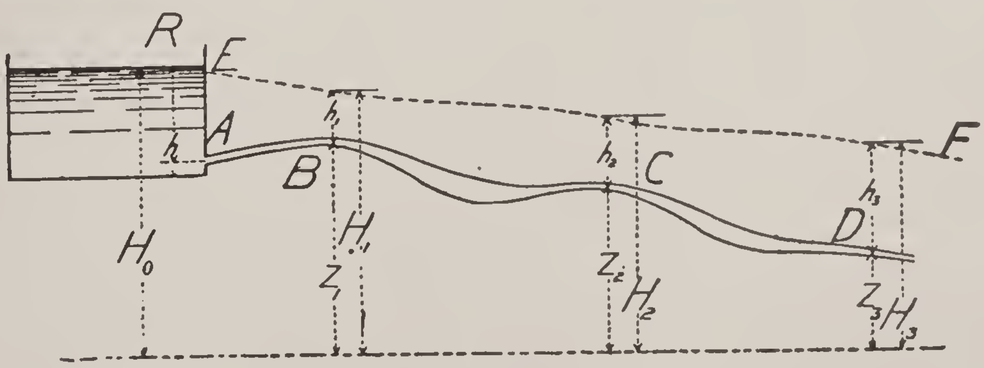


FIG. 32.

a reservoir. Let h_1, h_2, etc., be the pressures in this pipe at points B, C, D, etc.; let z_1, z_2, z_3, etc., be the elevations of the pipe at these points above any given plane, and H_0 the elevation of the surface of the water in the reservoir above this plane. Let v_1, v_2, v_3, etc., be the

* See Rafter's paper in Transactions Am. Soc. C. E., Dec. 1900, for a compilation of Bazin's coefficients. Also W. S. Paper No. 200, U. S. G. S., 1907.

† Trans. Am. Soc. C. E., Dec. 1900, pp. 266, 316.

velocities at the several points; they will be inversely proportional to the respective cross-sections of the pipe. The total energy, potential and kinetic, of any given volume of water, referred to the datum plane and expressed in terms of head or elevation is, $h + z + \frac{v^2}{2g}$, or $H + \frac{v^2}{2g}$. If there be no loss of energy in passing from point to point, then

$$h + z + \frac{v^2}{2g} = \text{a constant}, \quad . \quad . \quad . \quad . \quad . \quad (25)$$

which is Bernoulli's theorem.

In the flow of water through pipes some loss of energy takes place, or rather is dissipated as heat because of the internal friction in the water, so that the value of the expression written above, instead of remaining constant, becomes continually smaller as the water advances. If h_f = loss of head due to friction between points B and C, we may write

$$h_1 + z_1 + \frac{v_1^2}{2g} = h_2 + z_2 + \frac{v_2^2}{2g} + h_f. \quad . \quad . \quad . \quad (26)$$

In the figure the line EF, the ordinates to which measured from the pipe represent the pressures in the pipe, is called the hydraulic grade-line of the pipe. If the pipe is of uniform size, then $v_1 = v_2 = v_3$, etc., whence $h_1 + z_1 = h_2 + z_2 + h_f$; that is, the difference of elevation of the hydraulic grade-line at any two points represents the head lost in friction between these two points. As the elevation of the hydraulic grade-line is independent of the elevation of the pipe, it is convenient to refer directly to the datum plane and to write $H_1 = H_2 + h_f$. If the pipe lies above the hydraulic grade-line at any point, the pressure there will be less than atmospheric and the pipe will act as a siphon, provided its elevation above the hydraulic grade-line does not exceed the height of the water-barometer as given on page 214.

The usual problem to be considered in the flow of water through pipes is to trace out the various losses of head which take place between the water at the reservoir and at any point on a pipe-line, or, in other words, the head required $(H_0 - H_1)$ to overcome friction between A and B and to cause a velocity of flow equal to v_1. The relation is

$$H_0 - H_1 = \frac{v_1^2}{2g} + h_f.$$

Or if between any two points in a pipe, it is

$$H_1 - H_2 = \frac{v_2^2}{2g} - \frac{v_1^2}{2g} + h_f.$$

261. Nature of Fluid Friction.—The resistance to the flow of water through pipes may be considered as made up of two parts: (1) that due to the friction of water on the inner surface of the pipe,—a function of the viscosity of the water; and (2) a loss of energy resulting from eddies produced in the water by irregularities in the surface of the pipe. Until quite recently the great importance of the second factor has not been appreciated. Where water flows through a pipe with a rough inner surface, such as a riveted steel pipe, a great disturbance is caused in the stream of water, which may extend entirely through the mass; and it is the internal work of friction resulting from these eddies, and the churning up of the water caused by projecting rivets and plates, that constitutes by far the larger portion of the total energy consumed in the flow. The total loss of head in a pipe such as here mentioned is thus very much greater than that which occurs in a smooth pipe whose diameter is equal to the clear diameter of the rough pipe. With very rough channels almost the entire loss of head is chargeable to this element of internal friction due to eddies. In speaking, therefore, of the fluid friction in pipes it is necessary to bear in mind that it is principally internal friction, and to a very slight extent a friction of the water upon the surface of the channel.

The relative importance of resistance due to viscosity and that due to internal friction depends much upon the velocity of the flow and the diameter of the pipe, as well as upon the roughness of the channel. For low velocities and small diameters the resistance is largely due to viscosity, in which case it varies closely with the velocity. At ordinary and high velocities it is due largely to internal friction and varies nearly as the square of the velocity. These relations are, however, much influenced by the roughness of the channel.

262. General Formulas.—Owing to the variation in the general law due to variation in conditions, as noted in the foregoing article, it is impossible to derive a formula which will apply to all cases. The best that can be done is to select a form of expression which will approximately represent the law, and then to make use of coefficients by which the results of experiments may be conveniently expressed and utilized. The approximate law commonly used is that the loss of head varies with the square of the velocity, with the length of the pipe, and inversely with its diameter. Variations due to differences in the character of the surface of the pipe, and deviations from the assumed law due to other causes, are taken account of in the coefficient. The form of expression employed in theoretical discussions and to some extent in practical problems is

$$h = f \cdot \frac{l}{d} \cdot \frac{v^2}{2g}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (27)$$

in which $h =$ loss of head;
$f =$ friction factor;
$l =$ length of pipe;
$d =$ diameter.

The value of f in this expression is an abstract number, and therefore the same for any system of units. Tables giving values of f for smooth cast-iron pipes are given in various works on hydraulics; these values vary from about .01 for large pipes and high velocities to .03 for small pipes and low velocities.

In practice the use of a coefficient f is somewhat inconvenient. A more commonly used form of expression, and one which is applied also to open channels, is the Chezy formula:

$$v = c\sqrt{rs}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (28)$$

in which $r =$ hydraulic mean radius;
$s =$ hydraulic slope $= \frac{h}{l}$;
$c =$ a coefficient.

In the case of a pipe flowing full, $r = \frac{1}{4}d$.

In the above formula the value of c will be different for different systems of units. In most cases the foot and second are assumed to be used. Note that the coefficient c in this formula is equivalent to $\frac{16.04}{\sqrt{f}}$ of formula (27). Various expressions and values for c have been proposed by different investigators for different materials and under different conditions.

263. **Coefficients and Formulas for Cast-iron Pipes.**—The greatest attention has been given to the flow of water through smooth pipes of a character similar to new, or nearly new, cast-iron and smooth wrought-iron pipes. The most reliable and thorough examination of experiments of this character is probably that by Hamilton Smith.* The results of his investigation are given in the form of tables and diagrams of values of c in the Chezy formula for various velocities and diameters of pipes.

* Hydraulics. N. Y., 1886.

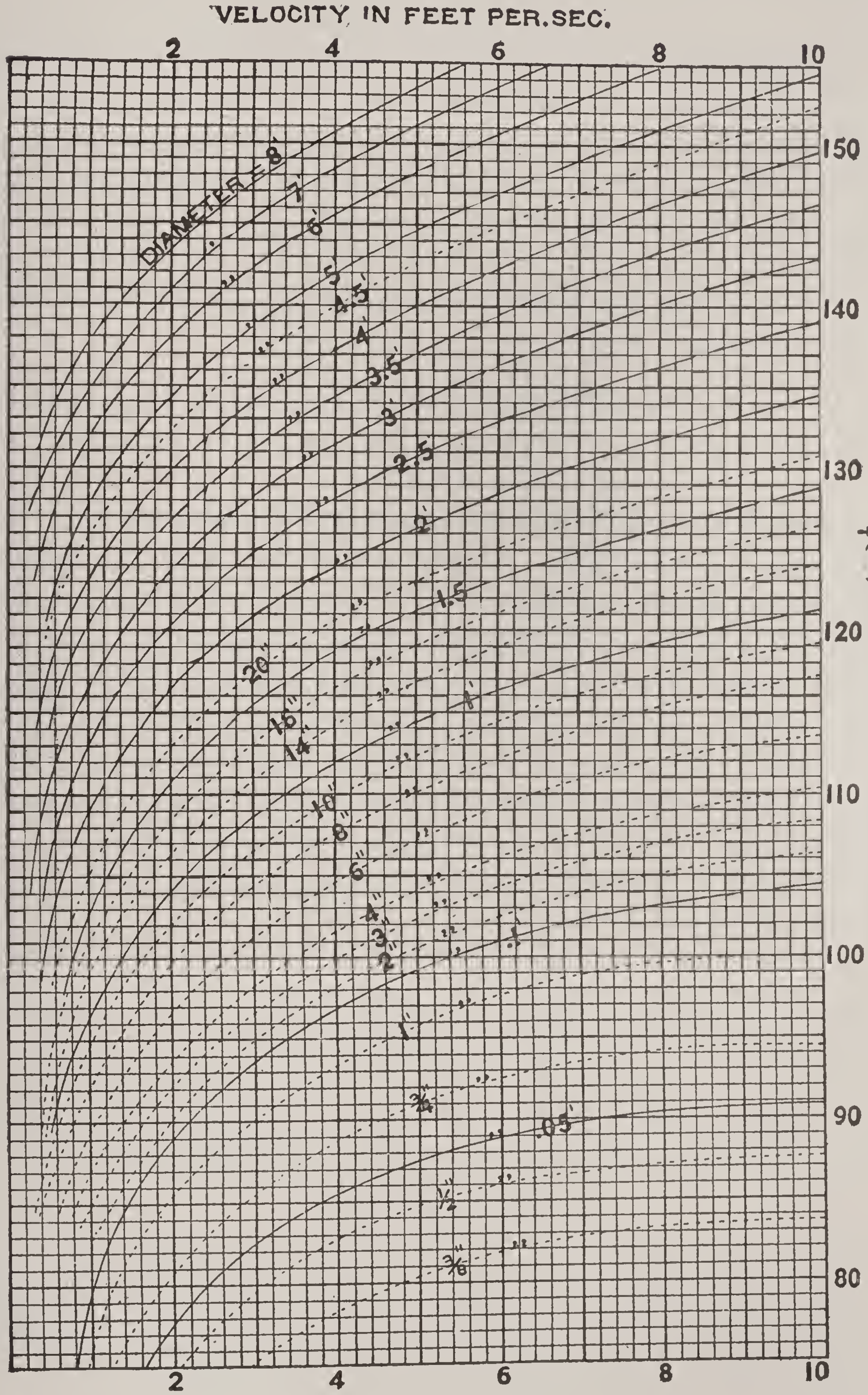


FIG. 33.—COEFFICIENTS FOR CAST-IRON PIPES. (Coffin.)

The diagram Fig. 33 is taken from Coffin.* It is slightly modified from the one given by Smith, the dotted lines being interpolated by Coffin. This diagram probably represents the best available information on the subject, but for large sizes and for very small sizes it must be considered as rather uncertain. For practical use also, the diagram is somewhat inconvenient, as it will frequently happen that both v and c are unknown, the head being the known factor. In this case the problem must be solved by making two or three successive approximations.

A formula which has been used extensively for cast-iron pipes is the Darcy formula. It is

$$v = \frac{1}{\sqrt{\alpha + \frac{\beta}{d}}} \sqrt{rs}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (29)$$

where α and β have the following values for English units:

For new cast-iron pipes $\alpha = .00007726$; $\beta = .00000647$.
For old or rough iron pipes $\alpha = .0001545$; $\beta = .00001294$.

It is to be noted that the values of these constants for old pipes are taken just double those for new pipes, which results in giving for a particular velocity double the loss of head in the former case as in the latter. Darcy's formula was derived from experiments on comparatively small pipes, and it is probably true that for large pipes and high velocities it gives somewhat too small values for the velocity. Levy has modified Darcy's formula slightly, but the modification is too small to be of any practical consequence.

Instead of using a variable coefficient it has been proposed by some to adopt a formula of the form $v = cr^n s^m$. This is in some respects a more convenient form of expression than the Chezy formula, and for any particular class of pipes it can be made to fit the experiments quite as well. Two formulas of this class deserve mention, that of Lampé and that of Flamant.† Lampé's formula is, in English units,

$$v = 77.68 d^{.694} s^{.555}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (30)$$

where $d =$ diameter of the pipe and $s =$ slope. Flamant, after making a thorough examination of all available experiments, proposes the following formulas: ‡ For cast-iron pipes slightly incrusted, such as would nearly always be the case after a few years of service,

$$v = 76.28 d^{\frac{5}{7}} s^{\frac{4}{7}}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (31)$$

* The Graphical Solution of Hydraulic Problems. New York, 1897.

† For discussion of recent formulas of this type see *Eng. News*, 1901, XLVI. p. 98; *Eng. Record*, 1903, XLVII., pp. 321, 667.

‡ Annales des Ponts et Chaussées, 1892, II. pp. 301–350.

and for new cast-iron pipes

$$v = 86.38 d^{\frac{5}{7}} s^{\frac{4}{7}}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (32)$$

Besides the above formulas and sets of coefficients, Kutter's formula for c (given in Art. 282) is frequently applied to pipes, although derived from experiments on open channels. The value of n is taken according to the nature of the surface, it being usually assumed about .011 for smooth pipes.

264. Comparison of Various Formulas.—In the following table a brief comparison is made of various formulas.

TABLE NO. 46.

COMPARISON OF VARIOUS FORMULAS FOR THE FLOW OF WATER IN SMOOTH PIPES.

Diam. in Inches.	Slope.	Velocities in Feet per Second.						
		Smith.	Lampé.	Flamant.		Darcy.		Kutter. $n = .011$.
				New Pipe.	Pipes in Service.	New Pipe.	Rough Pipe.	
2	.004	1.07	1.05	1.02	0.90	1.20	0.85	0.79
	.05	4.54	4.25	4.34	3.83	4.24	3.00	2.78
	.20	9.68	9.17	9.59	8.46	8.47	5.99	5.57
6	.001	1.00	1.04	1.02	0.90	1.18	0.83	0.96
	.01	3.67	3.73	3.80	3.35	3.72	2.63	3.12
	.1	12.80	13.38	14.13	12.47	11.77	8.32	9.85
12	.0005	1.08	1.14	1.12	0.99	1.22	0.86	1.14
	.005	3.96	4.11	4.18	3.69	3.86	2.73	3.73
	.025	9.56	10.03	10.51	9.27	8.64	6.11	8.35
36	.0001	1.01	1.00	0.98	0.87	0.97	0.68	1.06
	.001	3.57	3.60	3.66	3.23	3.07	2.17	3.62
	.01	12.60	12.92	13.64	12.03	9.72	6.87	11.50
60	.00005	1.01	0.97	0.95	0.84	0.89	0.63	1.04
	.0005	3.51	3.49	3.55	3.13	2.82	1.99	3.57
	.005	12.50	12.54	13.22	11.66	8.92	6.31	11.45

An examination of the table shows that the results by Smith's diagram, by the formula of Lampé, and by Flamant's formula for new pipes all agree very closely. Flamant's formula for pipes in service gives velocities about 10 per cent lower than Smith's diagram. The loss of head would be about 20 per cent higher. Darcy's formula gives, in the case of pipes of large size, results considerably below the others. Kutter's formula, on the other hand, gives relatively low results for small pipes.

265. Diagram Recommended for Use in the Design of Distributing Systems.—Among the various formulas mentioned, that of Flamant for cast-iron pipes in service is considered to be the most suitable for use in the design of ordinary distributing systems. It gives a slight margin of safety, which, in case the pipes are properly coated, will probably cover the deterioration for ten or fifteen years. This formula has the further advantage of being easily solved by the use of logarithms, and can also be readily solved graphically. The diagram on page 243, constructed after the principles laid down by M. Lalanne and M. Daries,* is based on this formula. It is very simple and offers little chance for error. On the four vertical lines are shown the four quantities, discharge, diameter, loss of head or slope, and velocity. The intersections of any straight line with these four vertical lines indicate corresponding values of these four quantities; so that any two being given, the other two are determined by the application of a straight-edge.†

In the case of the design of large and important conduits no formula should be accepted without question, but a special investigation of the matter of coefficients should be made. Much aid in estimating values for such coefficients will be obtained from the extensive Table No. 1 of Trautwine's translation of Ganguillet and Kutter, which contains a large collection of data of experiments, and calculated values of c and n.

266. Effect of Age of Service on Loss of Head.—In the diagram recommended for use some 10 per cent reduction of velocity, or about 20 per cent increase in head, has been allowed for slight deterioration of the pipe. In some cases this would doubtless be sufficient to cover a period of ten or even twenty years, while in other cases it would undoubtedly be too small an allowance. Uncoated cast-iron pipe becomes very badly tuberculated within ten years or less, and the reduction in carrying capacity of such pipe has been shown by experiments to be very great,—75 per cent, or more, in the case of 4- and 6-inch pipe. Properly coated cast-iron pipe, such as is universally used at the present time, corrodes very much less rapidly. Many cases have been reported of tar-coated pipe which has remained perfectly clean and bright for twenty or thirty years. On the other hand, there is ample evidence that tar-coating does not always entirely prevent the formation of tubercles. The extent of this action is doubtless influ-

* Nouvelle Annals de la Construction, Aug. 1897, XLIII.

† See also logarithmic diagram based on Levy's formula in *Eng. News*, 1899, XLII. p. 4. The Hazen-Williams slide rule is a very convenient device for such calculations. It is based on the formula $v = cd^{.63} s^{.54}$.

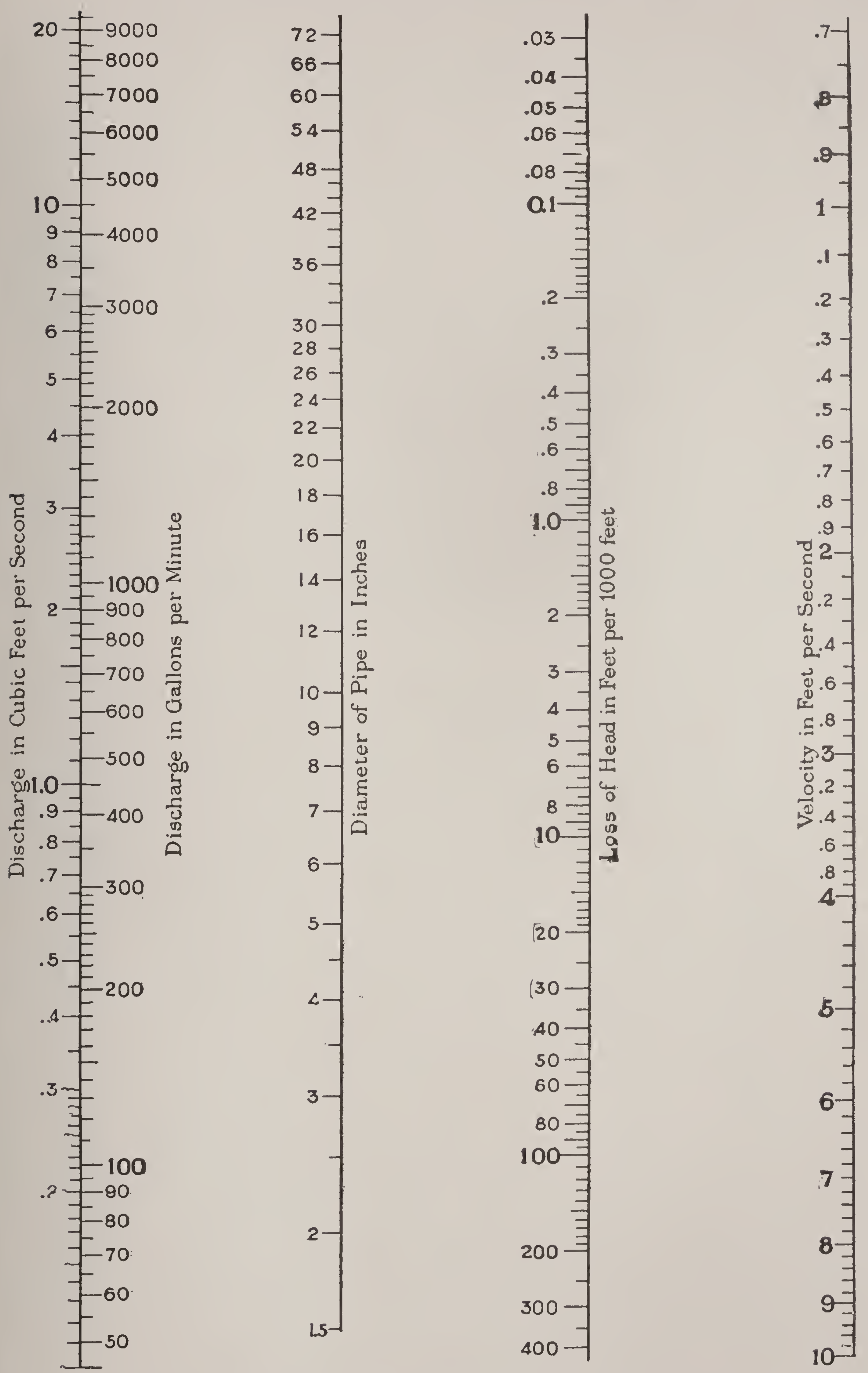


FIG. 34.—DIAGRAM FOR CALCULATING CAST-IRON PIPES.

enced by the quality of the water, but unless the contrary is known it will be necessary to assume that more or less incrustation will take place. Mr. FitzGerald reports that tar-coated pipe laid in Boston will become tuberculated in ten or fifteen years. In some cases considerable organic growth has become attached to the interior surface of the pipe, and this acts greatly to reduce the carrying capacity.

Tuberculation or incrustation affects the carrying capacity of a pipe in two ways: first, it reduces the cross-section, and, second, it increases the roughness of the pipe. The total effect on velocity will be very much greater in small pipes than in large pipes. Comparatively few reliable experiments have been made on old tar-coated pipes, most of the experiments on old pipes being on the uncoated pipe. Forbes * found in an eighteen-year-old tar-coated pipe 14 and 16 inches in diameter a value of c equal to from 90 to 93, about 25 per cent less than for new pipe. Experiments by FitzGerald † on a 48-inch pipe sixteen years old (tar-coated) gave a value of $c = 108$ ($n = .014$). The value of c for this same pipe when new was 140 to 144. The velocity in the old pipe was thus about 24 per cent less than in the new pipe.

Of the various formulas for old pipes, that of Darcy has probably been the most frequently used. As already noted, it simply gives twice the loss of head, or seven-tenths the velocity, as for new pipe. Mr. E. B. Weston suggests a series of coefficients whereby the increase in loss of head due to age is placed at about 16 per cent each five years over what it is at the beginning, but no allowance is made for differences in size of pipe. Coffin has constructed a diagram, based upon experiments on old pipes, which gives an increase in loss of head for increase of service of 15 to 25 per cent for each five years for ordinary velocities of 2 to 5 feet per second. The effect is made greater the greater the velocity. Some indication of the effect of age on riveted pipes will be found by a study of Table No. 48, page 246.

267. Friction Loss in Service-pipes.—The diagram on page 243 is not suitably arranged for calculating sizes of small pipes such as are used for service connections; and furthermore, since service-pipes are usually made of lead or of galvanized iron, they are little subject to corrosion, and the velocity which might be obtained from the diagram would be rather low. For the design of such pipes Smith's coefficients given in the diagram of Fig. 33 will give results sufficiently close for all practical purposes, although for pipes less than 1 inch in diameter

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* *Jour. New Eng. W. W. Assn.*, 1892, VI. p. 164.

† Trans. Am. Soc. C. E., 1896, XXXV. p. 241.

the results are probably somewhat too low. Table No. 47 is calculated from these coefficients. For very smooth pipes, such as those of lead or brass, Mr. E. B. Weston proposes the following formula for the friction factor f of eq. (27):

$$f = 0.0126 + \frac{0.0315 - 0.06d}{\sqrt{v}}, \quad . \quad . \quad . \quad . \quad (33)$$

in which the foot-unit is to be used. Tables based on this formula are given in Weston's "Friction of Water in Pipes." This formula gives velocities somewhat greater than Smith's coefficients.

TABLE NO. 47.

LOSS OF HEAD IN SMALL PIPES.

Velocity, Ft. per Sec.	½-inch Diam.		1-inch Diam.		1½-inch Diam.		2-inch Diam.		2½-inch Diam.		3-inch Diam.	
	Discharge, Gallons per Min.	Loss of Head, Feet per 100 Feet.	Discharge, Gallons per Min.	Loss of Head, Feet per 100 Feet.	Discharge, Gallons per Min.	Loss of Head, Feet per 100 Feet.	Discharge, Gallons per Min.	Loss of Head, Feet per 100 Feet.	Discharge, Gallons per Min.	Loss of Head, Feet per 100 Feet.	Discharge, Gallons, per Min.	Loss of Head, Feet per 100 Feet.
1	......	......	2.5	.82	......	......	9.8	.35	15.3	.27	22.1	.22
1½	......	......	3.7	1.61	8.2	1.01	14.7	.72	23.0	.56	33.1	.45
2	......	......	4.9	2.63	11.0	1.67	19.6	1.19	30.7	.92	44.2	.76
2½	1.53	10.00	6.1	3.87	13.8	2.44	24.5	1.73	38.3	1.36	55.2	1.12
3	1.84	14.2	7.4	5.33	16.6	3.37	29.4	2.39	46.0	1.88	66.2	1.55
3½	2.15	17.7	8.6	6.95	19.3	4.44	34.3	3.14	53.7	2.47	77.3	2.02
4	2.45	22.4	9.8	9.31	22.1	5.56	39.2	3.98	61.3	3.13	88.3	2.56
4½	2.76	27.6	11.0	10.9	24.8	6.89	44.1	4.91	69.0	3.85	99.4	3.14
5	3.06	33.3	12.3	12.3	27.6	8.33	49.1	5.94	76.7	4.66	110.4	3.81
5½	3.37	39.7	13.5	15.5	30.4	9.88	53.9	6.42	84.3	5.53	121.4	4.52
6	3.68	46.7	14.7	18.2	33.1	11.5	58.8	8.22	92.0	6.45	132.5	5.27
6½	3.98	54.2	15.9	20.9	35.9	13.3	63.8	9.51	99.6	7.46	143.5	6.10
7	4.29	62.6	17.2	24.0	38.6	15.2	68.6	10.8	107.3	8.53	154.6	7.01
7½	4.60	71.3	18.4	28.9	41.4	17.3	73.6	12.4	115.0	9.70	165.6	7.93
8	4.90	80.8	19.6	32.7	44.2	19.5	78.5	13.9	122.6	10.9	176.6	8.94
8½	5.21	90.6	20.9	36.9	46.9	21.9	83.4	15.6	130.3	12.2	187.7	10.00
9	5.51	101.6	22.1	41.2	49.7	24.4	88.3	17.3	138.0	13.6	198.7	11.1
9½	5.82	112.9	23.3	45.9	52.5	27.2	93.2	19.3	145.6	15.1	209.7	12.3
10	6.13	124.8	24.5	48.0	55.2	30.2	98.1	21.3	153.3	16.6	220.8	13.7

268. Coefficients for Riveted Pipes.—The friction loss in riveted pipes depends upon the thickness of the plates and the manner of making the joints. Experiments on this class of pipes are not sufficiently numerous to enable any general expression to be formulated, so that in the design of such pipes the selection of coefficients must be made by reference to the experimental data. In general it is found that the coefficient c changes little with change in diameter or velocity, and in this respect exhibits considerable difference from its variation in cast-iron pipe. For ordinary velocities the value of c for new pipe appears to range between 100 and 115. Probably a value of 110 would be as great as should be used in almost any case. Kutter's formula for c is very often used, the value of n being taken equal to .013 to .015.

To aid in the selection of a coefficient, all the most important experiments on large riveted pipe are given in Table No. 48. Further data regarding these experiments will be found in the various publications referred to. The table is mostly taken from a similar one given in the paper by Profs. Marx, Wing, and Hoskins in Trans. Am. Soc. C. E., 1898, vol. XL. p. 471.

TABLE NO. 48.

VALUES OF THE COEFFICIENT c FOR RIVETED PIPES.

Number of Experiment.	Diameter of Pipe in Inches.	Velocities in Feet per Second.										
		1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0
		Values of Coefficient c.										
1	11	......	......	......	......	......	......	......	107.1	......	......	110.6
2	14	87										
3	15	......	......	......	......	......	......	......	......	111.6		
4	16	......	......	......	......	......	......	......	110.0*			
5	24	......	......	......	......	......	78.5*					
6	33	......	......	......	......	......	......	123.2*				
7	35	......	......	......	......	......	126.8*					
8	36	86	90.8	95.2	99.4	103.3	107.0	110.6	114.0	117.2	120.4	123.6
9	36	......	......	......	......	......	......	......	......	106.3*		
10	38	......	......	......	......	......	116.6*					
11	38	......	......	......	......	......	......	109.2*				
12	42	......	......	......	115.9*							
13	42	96.0	103.0	107.9	111.0	112.6	113.0	112.8	111.8	110.8	110.2	110.0
14	42	101.0	102.8	104.3	105.5	106.4	107.2	107.8	108.2	108.4	108.5	108.5
15	48	101.2	105.4	108.8	111.2	112.8	113.4	113.2	112.4	112.0	111.7	111.6
16	48	78.0	84.6	89.6	92.4	93.0	93.2	94.0	94.2	94.4	94.7	94.9
17	48	97.2	100.8	103.3	104.9	105.3	104.8	104.0	103.7	103.7	103.7	103.7
18	48	97.1	98.7	100.3	101.6	102.2	103.6	104.2	104.7	105.1	105.2	105.2
19	72	110	111	110	108	108	110	111				
20	72	81.6	92.0	98.0	101.3	102.4	103.2	103.8	104.3	104.7	105.0	
21	103	116.6	112.7	110.3	108.8	107.7	106.9	106.2	105.6			

* Most of the values in the table were obtained from plotted curves. Those marked with an asterisk are from single observations and are inserted in the table under the velocity corresponding most nearly with the observed velocity.

The following is a brief description of the experiments the results of which are tabulated above:

Nos. 1 and 3. By Hamilton Smith, North Bloomfield, Cal. Sheet iron with taper joints; asphalt and tar-coating; 5 years old.

No. 2. By A. McL. Hawks. Cylinder-joints; asphalt coating. Tested when 3 years old, and also when 6 years old with same results. Trans. Am. Soc. C. E., 1899, XLII. p. 155.

No. 4. By A. L. Adams. Astoria pipe-line. Cylinder-joints; asphalt coating; new pipe. Trans. Am. Soc. C. E., 1896, XXXV. p. 226.

No. 5. By Geo. W. Rafter on the old Rochester conduit. Cylinder-joints; 14 years old. Trans. Am. Soc. C. E., 1891, XXVI. p. 20.

Nos. 6, 7, and 12. By. I. W. Smith on the Portland conduit. Cylinder-joints; asphalt coating; new pipe. Trans. Am. Soc. C. E., 1891, XXVI. p. 203.

Nos. 8 and 9. By Clemens Herschel on the conduit of the East Jersey Water Company from Belleville to South Orange. No. 8 made with new pipe; No. 9 with pipe 4 years old. Cylinder-joints; asphalt coating. Herschel's "115 Experiments."

Nos. 10 and 11. By Kuichling on different sections of the Rochester conduit. Cylinder-joints. The section of pipe in experiment No. 10 was coated partially with asphalt and partially with the Sabin coating. Average plate thickness = .27 inch. Section in experiment No. 11 coated with Sabin coating; average plate thickness .31 inch. Age of pipes about $1\frac{1}{2}$ years. These same sections gave in November and December, 1898, coefficients of 112.9 and 105.9 respectively. Annual Reports of Executive Board of Rochester, 1895–98.

Nos. 13 to 18. Experiments by Herschel. No. 13, Kearney extension of the East Jersey Water Company's pipe-line. Taper joints; new pipe; coating "unusually smooth." No. 14, on conduit No. 2 of the East Jersey Water Company; new pipe. No. 15, on conduit No. 1; cylinder-joints; asphalt coating; new pipe. Nos. 16 and 17, on portions of the same conduit as No. 15, but 4 years later. No. 18, on portion of conduit No. 2; taper joints; new pipe. "115 Experiments."

No. 19. Experiments by Marx, Wing, and Hoskins on the conduit of the Pioneer Electric Company, Ogden. New pipe; butt-joints; asphalt coating. Trans. Am. Soc. C. E., 1898, XL. p. 471.

No. 20. Same as No. 19, but 2 years later. Proc. Am. Soc. C. E., Feb. 1900, p. 108.

No. 21. By Herschel on Holyoke flume. Cylinder-joints; paint coating washed off; rather rusty. "115 Experiments."

269. Friction Loss in Wood-stave Pipe.—Very few experiments have been made on this class of pipes. Such information as is available is collected in the paper by Marx, Wing, and Hoskins, already referred to, in which are also described some experiments by the authors on the Ogden 72-inch wooden-pipe line. The few experiments made previous to these indicated a value of n in Kutter's formula of about .010 for 18- to 30-inch pipes. The experiments on the Ogden pipe-line, however, gave a value of c varying generally from 115 to 125; (n = .014 — .013). Experiments by Noble on 44- and 54-inch pipes gave values of c equal to 110—115 (n = .013) for the 44-inch, and of 115—129 (n = .013 — .012) for the 54-inch pipe. Velocities ranged from 3.5 to 4.8 feet per second in the former case and from 2.3 to 4.7 feet per second in the latter case.*

270. Measurement of Flow through Large Pipes.—The quantity of water flowing through pipes may be measured by means of weirs or

* Trans. Am. Soc. C. E., 1902, XLIX. p. 143.

orifices, or by noting the rate of filling of a reservoir, or by the use of meters. In Chapter XXIX various kinds of meters are described and discussed with particular reference to their use on service-pipes. For accurately measuring the quantity of water flowing through large pipes, as in the making of tests, probably the best form of meter is the Venturi. This meter simply consists of a contracted section of pipe, *AB*, Fig. 35, with pressure-gauges at *A* and *C*. If v_1 and v_2 are the

FIG. 35.—VENTURI METER.

velocities at *A* and *C*, h_1 and h_2 the pressures, a_1 and a_2 the areas, then, neglecting friction, we have, from eq. (26), page 226,

$$h_1 + \frac{v_1^2}{2g} = h_2 + \frac{v_2^2}{2g}. \quad \ldots \ldots \quad (34)$$

But if $Q =$ theoretical discharge, then $v_1 = \frac{Q}{a_1}$ and $v_2 = \frac{Q}{a_2}$.

Substituting and reducing, we have

$$Q = \frac{a_1 a_2}{\sqrt{a_1^2 - a_2^2}} \sqrt{2g(h_1 - h_2)}. \quad \ldots \quad (35)$$

If $q =$ actual discharge, then $q = cQ$, where c is a coefficient determined by experiment and nearly equal to unity.*

271. **Minor Losses of Head.**—*Loss of Head at Entrance.*—This is expressed by the formula

$$h = \left(\frac{1}{c^2} - 1\right)\frac{v^2}{2g}, \quad \ldots \ldots \quad (36)$$

where $v =$ velocity in the pipe, and c is the coefficient of discharge of a short tube having the same form as the end of the pipe. For various forms at entrance we have the following values:

	c.	$\frac{1}{c^2} - 1$.
Pipe projecting into reservoir	.72	.93
End of pipe flush with reservoir	.82	.49
Conical or bell-shaped mouth	.93 to .98	.15 to .04

* See paper by Herschel on Venturi Meter, Trans. Am. Soc. C. E., 1887, XVII. p. 228.

272. *Loss Due to Sudden Enlargement.*—This is given by the formula

$$h = \left(\frac{a_2}{a_1} - 1\right)^2 \frac{v_2^2}{2g}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (37)$$

in which a_1 and a_2 are the cross-sections of the smaller and larger pipes respectively, and v_2 is the velocity in the larger pipe.

273. *Loss Due to Sudden Contraction.*—This is given by

$$h = \left(\frac{1}{c'} - 1\right)^2 \frac{v^2}{2g}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (38)$$

in which c' depends on the ratio of the two diameters and is given by Merriman as follows:

Ratio of smaller to larger diameter....	0	.2	.4	.6	.8	1.0
c'....	.62	.63	.64	.67	.72	1.0

274. *Loss of Head at Bends* * is equal to $h = n\dfrac{v^2}{2g}$ . . . . (39)

for a 90° bend, in which n has the following values according to the ratio of the radius of pipe r to the radius of curvature R (Weisbach):

$\frac{r}{R}$........	.1	.2	.3	.4	.5	.6	.7	.8	.9	1.0
n..........	.13	.14	.16	.21	.29	.44	.66	.98	1.41	1.98

275. *Loss of Head in Valves.*—Weisbach's experiments on small gate-valves gave values for n in the expression $h = n\dfrac{v^2}{2g}$ as follows:†

Ratio of height of opening to diameter..	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{1}{4}$	$\frac{1}{8}$
Values of n........................	.07	.26	.81	2.1	5.5	17	98

In applying the above formula v is the velocity in the pipe.

For a throttle-valve placed at various angles θ with the axis of the pipe, Weisbach found the following values of n:

θ...	5°	10°	20°	30°	40°	50°	60°	65°	70°
n...	.24	.52	1.5	3.9	11	33	118	256	750

Experiments on large gate-valves have been made by Kuichling and by J. W. Smith. The following table gives values of the coefficient c in the expression $Q = cA\sqrt{2gh}$ as deduced by Kuichling from these two sets of experiments.‡ In this expression A is the area of the opening, h is the head lost in the valve, Q is the rate of discharge.

* Williams found that for large pipes the total resistance in a 90° bend increased with increased radius of bend beyond four or five pipe diameters. The total resistance in a length of pipe of 80 diameters (including a 90° bend) of moderate radius was found to be from 20 to 30 per cent in excess of the same length of straight pipe. (See his valuable paper in Trans. Am. Soc. C. E., 1902, XLVII. p. 1.)

† See values for 4-in. valves in *Eng. News*, 1902, XLVII. p. 302.

‡ Trans. Am. Soc. C. E., 1895, XXXIV. p. 243.

TABLE NO. 49.

COEFFICIENTS FOR LARGE GATE-VALVES.

Ratio of height of opening to diameter	.05	.1	.2	.3	.4	.5	.6	.7	.8
Ratio of area of opening to total area	.05	.10	.23	.36	.48	.60	.71	.81	.89
Coefficient c for 24-inch valve	1.7	1.0	.72	.70	.77	.92	1.2	1.6	
Coefficient c for 30-inch valve	1.2	.9	.83	.82	.84	.90	1.05	1.35	2.1

Experiments at the Ohio State University in 1899 on various kinds of small valves showed that gate-valves when wide open gave a coefficient of discharge equal to from .5 to .7, and globe-valves usually from .3 to .4.*

276. Hydraulics of Fire-streams.—Table No. 50 contains data pertaining to the loss of head in fire-hose, and the character of fire-streams under different pressures and for different-sized nozzles. The data are taken from much more extensive tables given by Freeman in his elaborate paper on the hydraulics of fire-streams.†

TABLE NO. 50.

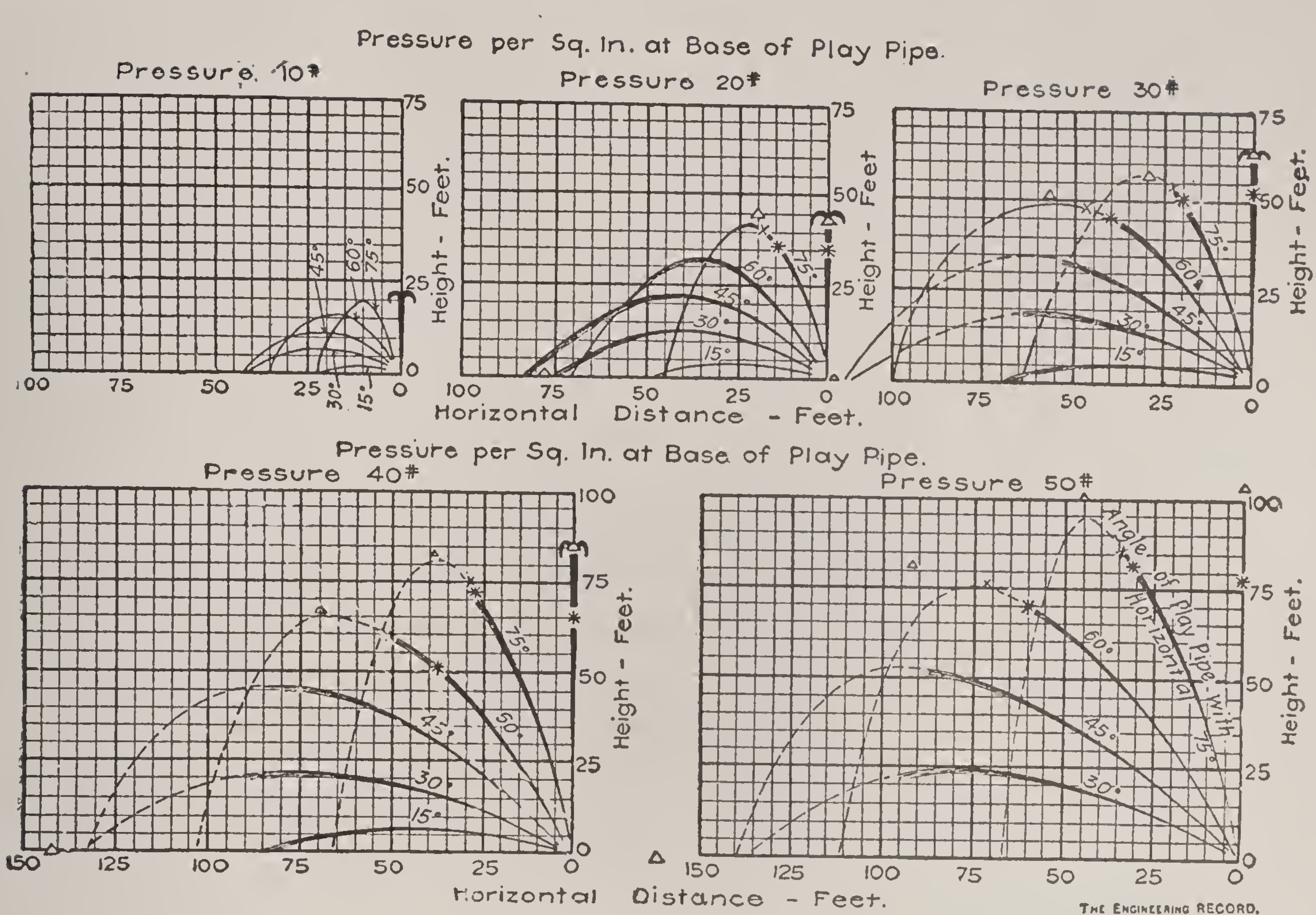
HOSE AND FIRE-STREAM DATA.

Pressure at Nozzle (Base of Play-pipe).	1-inch Smooth Nozzle.					$1\frac{1}{8}$-inch Smooth Nozzle.					$1\frac{1}{4}$-inch Smooth Nozzle.				
	Discharge in Gallons per Minute.	Loss of Head per 100 Feet of Ordinary Hose.	Vertical Height of Jet for Good Fire-streams.	Maximum Horizontal Distance for Good Fire-streams.	Extreme Drops at Level of Nozzle.	Discharge in Gallons per Minute.	Loss of Head per 100 Feet of Ordinary Hose.	Vertical Height of Jet for Good Fire-streams.	Maximum Horizon'al Distance for Good Fire-streams.	Extreme Drops at Level of Nozzle.	Discharge in Gallons per Minute.	Loss of Head per 100 Feet of Ordinary Hose.	Vertical Height of Jet for Good Fire-streams.	Maximum Horizontal Distance for Good Fire-streams.	Extreme Drops at Level of Nozzle.
lbs.		lbs.	ft.	ft.	ft.		lbs.	ft.	ft.	ft.		lbs.	ft.	ft.	ft.
20	132	5	35	37	77	168	8	36	38	80	209	12	37	40	83
30	161	7	51	47	109	206	12	52	50	115	256	19	53	54	119
40	186	10	64	55	133	238	16	65	59	142	296	25	67	63	148
50	208	12	73	61	152	266	20	75	66	162	331	31	77	70	169
60	228	15	79	67	167	291	24	83	72	178	363	37	85	76	186
70	246	17	85	72	179	314	28	88	77	191	392	43	91	81	200
80	263	20	89	76	189	336	32	92	81	203	419	49	95	85	213
90	279	22	92	80	197	356	36	96	85	214	444	55	99	90	225
100	295	25	96	83	205	376	40	99	89	224	468	62	101	93	236

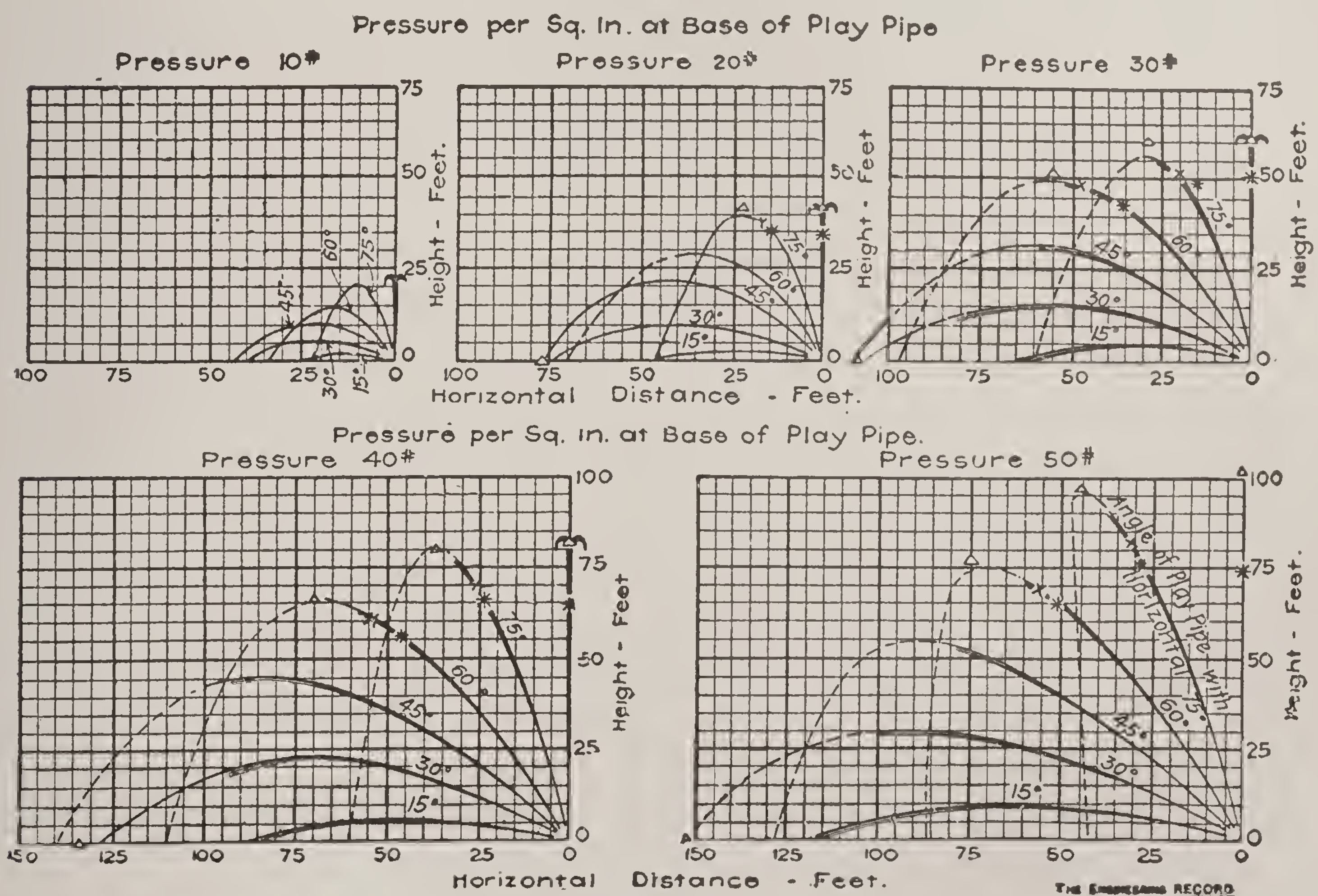
The range and quality of fire-streams has recently been studied by photography by Prof. Marston. His results for 1-inch and $1\frac{1}{8}$-inch smooth nozzles are shown in the diagrams on page 251. The paths

* *Eng. Record*, 1899, XL. p. 78.

† Trans. Am. Soc. C. E., 1889, XXI. p. 303.



1⅛ INCH SMOOTH NOZZLE.



1-INCH SMOOTH NOZZLE.

Freeman's Experiments Plotted thus: { *Extreme Drops* △ / *Fair Fire Stream.* × / *Good Fire Stream.* ✱

FIG. 36.—FIRE-STREAM DIAGRAMS.
(From *Engineering Record*, Feb. 18, 1899.)

of the streams are shaded for pressures of 30 pounds or above, wherever a solid stream was shown by the negative. Beyond the limits indicated, the slightest breeze would break up the stream badly. The results of Freeman's experiments are also given on the diagrams. The pressures in Marston's experiments were measured at the base of the play-pipe, and varied from the effective pressure at the orifice from -1.1 to $+1.7$ pounds per square inch. Experiments were also made on smooth nozzles of $\frac{3}{4}$- and $\frac{7}{8}$-inch, and on ring nozzles of 1-inch and $1\frac{1}{8}$-inch diameter. The ring nozzles gave, in general, streams of slightly less range than the smooth nozzles.

277. **Friction Loss in Fire-hydrants.**—Experiments by M. C. L. Newcomb at Holyoke, Mass., on twenty-one different kinds of hydrants showed that with a discharge of 500 gallons per minute the loss of head was in nearly all cases between 1 and 2 pounds per square inch, the maximum being 2.5 pounds, and the minimum .8 pound. In many cases the greater portion of the loss of head occurred in the nozzle and shows the necessity of making the passages in hydrants of large size and in curved lines.*

278. **Water-hammer.**—When a volume of water flowing in a pipe has its velocity rapidly checked by the closing of a valve or otherwise, a pressure is developed in excess of the static pressure. If the action is very sudden, the pressure will be very great, particularly if the velocity is high and the pipe of great length. This effect in general is called water-hammer.

The estimation of the amount of excess pressure due to water-hammer in a pipe system is a matter of difficulty, but all engineers admit that some allowance must be made. Where the conditions are definitely known, such as the size and length of pipe, and the rate and manner of closing a valve, it is quite possible to compute the pressure with a considerable degree of accuracy. The actual problem is, however, greatly complicated, due partially to the irregularity in form and arrangement of the pipes, but chiefly to a lack of exact knowledge with respect to the movement of the valves, pump-plungers, or whatever may be the cause of the trouble. It is, however, possible to gain from theoretical considerations, and from experiments, a knowledge of certain general laws with respect to water-hammer, and to indicate certain limits to the pressure which may be produced by it. The results of special experiments carried out under certain given conditions may also be studied with advantage.

279. *Theoretical Considerations.*—The greatest possible water-

* Trans. Am. Soc. M. E., 1899, xx. p. 494.

hammer will be caused in any particular case when a valve is closed so quickly as to be practically instantaneous. In this case the resulting pressure is a function involving the elasticity of the water and of the pipe, and is a case of impact of an elastic prism. If the elasticity of the pipe be neglected, which may be done for ordinary sizes, the pressure of impact has been shown to be

$$p = \frac{vE_w}{V}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (40)$$

in which v = initial velocity of water;
E_w = modulus of elasticity of water
= 300,000 pounds per square inch; and
V = about 4700 ft. per second, = velocity of sound in water.

Substituting the values of E_w and V, we have, in pounds per square inch, where v is in feet per second,

$$p = 64v. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (41)$$

The pressure developed is thus proportional to the velocity of the water and is independent of the length of the pipe.

Mr. Frizell * has derived the following expression for the pressure, in which account is taken of the elasticity of the pipe:

$$p = \frac{v}{V} \cdot \frac{E_w}{1 + \frac{2r}{t} \cdot \frac{E_w}{E_p}}, \quad . \quad . \quad . \quad . \quad . \quad . \quad (42)$$

in which E_p = modulus of elasticity of pipe;
$2r$ = diameter of pipe in feet; and
t = thickness of pipe in inches.

These formulas are valuable as indicating the maximum possible limit of the water-hammer. The question arises, however, as to what constitutes a sudden stoppage of the water. According to Mr. Frizell the closing of a valve is essentially instantaneous if the time of closing is less than the time necessary for the wave of pressure to be transmitted to the end of the pipe and back, at the rate of about 4700 feet per second. The length of pipe is thus seen to enter into the problem of water-hammer by affecting the definition of the word 'sudden.' In long pipes, therefore, the operation of the valves in a way similar to that customary for short pipes would be likely to cause a much greater water-hammer; and in very long pipes a severe hammer might be experienced even though the operation were relatively slow.

The other case to be considered, the one in which the stoppage of the flow is not sudden, is the more usual problem, but at the same time

* Trans. Am. Soc. C. E., 1898, XXXIX. p. I.

one more difficult of treatment. The pressure developed in this case is simply a function of retardation and of the static head; and if the manner of operating a valve is precisely known, the pressure can be computed. If a valve is closed at a uniform rate, the pressure will be a maximum at the end of the movement, and with similar laws of closing the pressure will be approximately proportional to the length of the pipe, to the speed of closing of the valve, and to the velocity of the flow. A lower maximum pressure will be experienced if valves are so arranged as to close rapidly during the first part of the movement and slowly at the last.*

280. *Experiments on Water-hammer.*—Experiments on water-ram have usually been made by determining the pressures developed in certain short lines of pipe by the sudden closing of a gate-valve, the pressure being measured by means of a gauge. Experiments by Mr. E. B. Weston at Providence, R. I.,† on small pipes, by the method described, gave results which approached well towards the theoretical maximum given by eq. (40). The ram was practically proportional to the velocity of the water.

Experiments by Prof. Carpenter on 2-inch pipes gave results which are shown in the diagram of Fig. 37.‡ The curve for the experiments without air-chamber shows values of pressure from one-half to two-thirds of those obtained by the use of Mr. Frizell's formula, eq. (42). The pressure here appears to increase somewhat more rapidly than the velocity. The effect of air-chambers is very marked.

Experiments at Dartmouth College in 1898 § indicated that the force of water-ram varies with the velocity, with the speed of the closing of the valve, and with the volume of water in the pipe. It is also greater when dead ends are located near the valve. Extensive experiments have been carried out still more recently in Russia, the results of which go to confirm the general laws expressed by the formulas of Art. 279. These experiments have also led to the general statement that the pressure caused by a sudden decrease in velocity is, for each foot per second of such decrease, approximately 4 atmospheres (60 pounds per square inch) for small pipes and 3 atmospheres (45 pounds per square inch) for large pipes.‖ These values are very nearly the same as would be obtained from eq. (42).

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* For a theoretical discussion of the pressures developed when valves are closed slowly, together with results of some experiments with slowly moving valves, see Trans. Assn. C. E. of Cornell University, 1898, p. 31. See also a paper by Prof. I. P. Church in the Journal of the Franklin Inst., April and May, 1890.

† Trans. Am. Soc. C. E., 1885, XIV. p. 238.

‡ Trans. Am. Soc. M. E., 1894, XV. p. 510. *Eng. Record*, 1894, XXX. p. 173.

§ *Eng. News*, 1898, XXXIX. p. 186.

‖ *Eng. News*, 1900, XLIV. p. 80.

281. *Practical Conclusions.*—From the foregoing discussion the extreme limits of water-hammer are approximately indicated, and it would appear that the general laws are to some extent quite definitely known, from both theoretical and experimental considerations. In simple cases of the operation of valves it is easy to determine from these considerations what are the necessary precautions to be taken. In many cases in practice the difficulty arises from causes not easily traced, and undoubtedly the effect of hammer is often greatly increased by the setting up of vibrations due to some synchronous action of the pumps or other machinery, accompanied by the collection of air in the pipes.

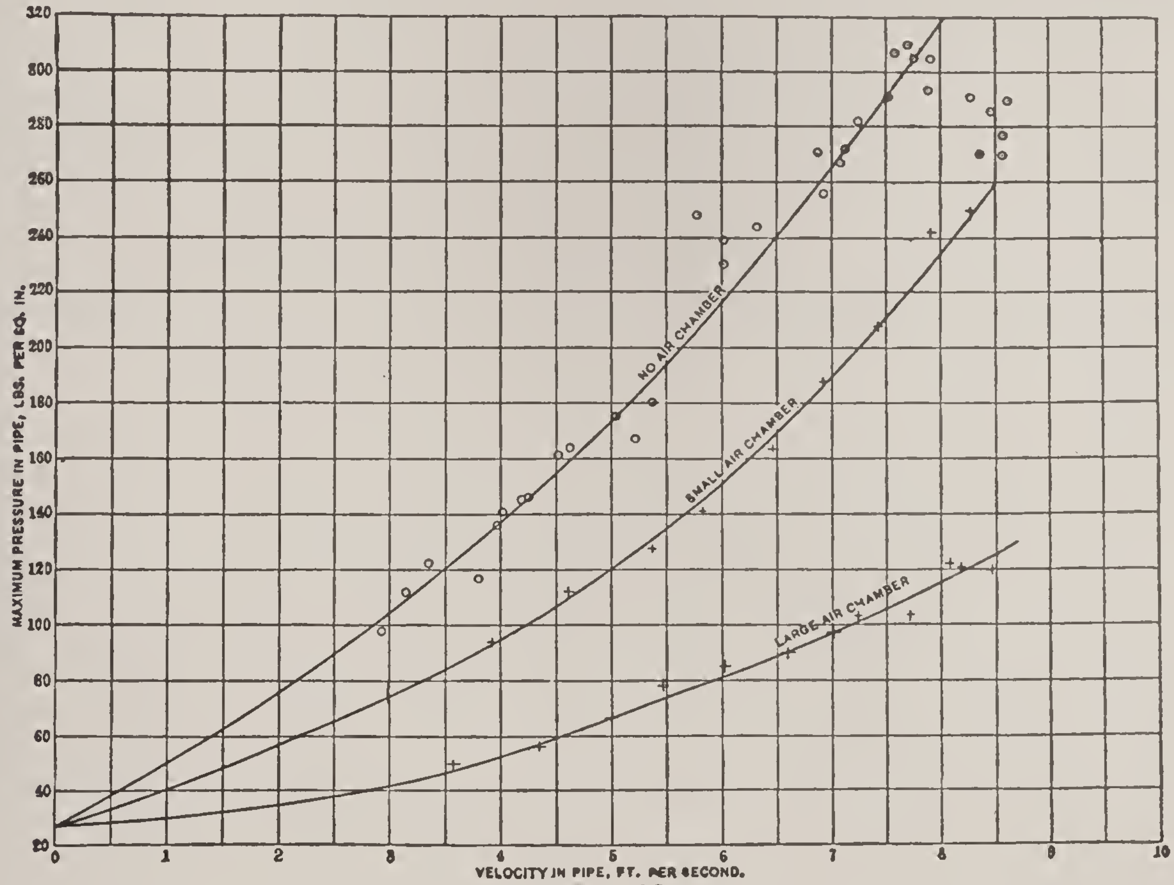

FIG. 37.—EXPERIMENTS ON WATER-RAM BY CARPENTER.

To prevent excessive water-hammer from the closing of valves it is only necessary so to design them that they cannot be closed very suddenly, or if they are closed suddenly they should be so arranged that the velocity in the adjacent main shall not exceed a moderate limit of 8 to 12 inches per second at the time when the valve is operated. If the velocity at this point were as great as 1 foot per second, the maximum possible hammer would be something less than 64 pounds per square inch, according to eq. (42). The operation of ordinary valves in a distributing system can scarcely give so great a ram as the above. Air-valves and pressure-relief valves should be so proportioned that any sudden reduction of velocity caused in filling or operating a pipe-line shall not exceed a moderate limit such as above specified. The closing of hydrant-valves would have an effect depending upon the amount by which the velocity of water in the adjacent main is influenced thereby. Hydrants attached to small mains will thus have a greater effect than when attached to large ones. In the operation of long pipe-lines at high velocities, such as are used in power plants on the Pacific coast, special precautions must be taken to insure a very slow movement of the valves; and frequent use made of air-chambers and relief-valves. It has been found that to check the pulsations which are caused by the waves of pressure set up, it is advantageous to use air-chambers which are single-acting, that is, those which permit water to enter readily but not to flow out rapidly. In a distributing system, water-ram is sometimes caused by the action of the pumps, due usually to a lack of capacity in the air-chambers, or to their becoming filled with water. The effect of such ram upon the neighboring pipes is frequently influenced by the presence of dead ends, and in some cases trouble of this sort has been removed by connecting up two or more such dead ends.

FLOW OF WATER IN OPEN CHANNELS.

282. Formulas Employed.—In calculating the flow of water in open channels the Chezy formula (page 228) is used. It is

$$v = c\sqrt{rs},$$

in which r = hydraulic mean radius;

s = sine of the slope of the water-surface;

c = a coefficient.

For channels similar in character to smooth pipe the value of c may be taken from page 231.

The most commonly used value of c is that given by Kutter's formula, which was derived from a study of a large number of experiments. It is,

$$c = \frac{\frac{1.81}{n} + 41.65 + \frac{0.0028}{s}}{1 + \frac{n\left(41.65 + \frac{0.0028}{s}\right)}{\sqrt{r}}}, \quad \ldots \ldots \quad (43)$$

in which n is a coefficient of roughness. The following are the values of n usually assumed for the various surfaces mentioned:

	n
Channels of well-planed timber	.009
" " neat cement or of very smooth pipe	.010
" " unplaned timber or ordinary pipe	.012
" " smooth ashlar masonry or brickwork	.013
" " ordinary brickwork	.015
" " rubble masonry	.017
" in earth free from obstructions	.020 to .025
" with detritus or aquatic plants	.030

In formula (43) it is seen that the value of c is made to vary with r and also with s, but the effect of a change in s for all but those cases in which the slope is very small is of little importance, and for all practical purposes in the design of sewers and water-conduits a constant value of s, such as .001, may be assumed. Table No. 51 gives values of c, corresponding to various values of r and of n, for a constant value of s equal to .001.

TABLE NO. 51.

VALUES OF c IN KUTTER'S FORMULA WHEN $s = 0.001$.

r in Feet.	Values of n.									
	.009	.010	.011	.012	.013	.015	.017	.020	.025	.030
.1	108	94	82	73	65	53	45	35	26	20
.2	129	113	100	89	80	66	56	45	34	26
.3	142	124	111	99	90	75	63	52	38	30
.4	150	132	118	106	96	80	69	56	42	34
.5	157	139	124	111	101	85	73	60	45	36
.6	162	143	128	116	105	89	76	63	48	38
.7	166	147	132	119	109	92	79	65	50	40
.8	170	151	135	122	112	95	82	68	52	42
.9	173	154	138	125	114	97	84	70	54	43
1.0	175	156	140	127	116	99	86	71	55	45
1.2	180	160	145	131	120	103	89	74	58	47
1.4	184	164	148	135	124	106	92	77	60	49
1.6	187	167	151	137	126	108	94	79	62	51
1.8	189	169	153	140	129	110	97	81	64	53
2.0	191	172	155	142	130	112	98	83	65	54
2.5	196	176	160	146	135	116	102	86	69	57
3.0	199	179	163	149	138	119	105	89	71	59
3.5	202	182	166	152	140	122	107	91	73	61
4.0	204	184	168	154	143	124	110	93	75	63
4.5	206	186	170	156	144	126	111	95	77	64
5.0	208	188	172	158	146	127	113	97	78	66

Values of c from gaugings of the New Croton Aqueduct and of the Sudbury Aqueduct are represented in Fig. 38.* The conduits are of horseshoe shape and are brick-lined. In the figure, Kutter's formula is also plotted for values of n equal to .013 and .014. It is to be noted that this formula gives values of c, as compared with the experiments, which increase too rapidly with increase in r.

The adopted curve for the Stony Brook conduit is also given. The equation of this curve is $c = 122.6r^{0.57}\sqrt{s}$. The flow in the new Croton Aqueduct is closely represented by the equation $v = 124r^{0.56}\sqrt{s}$.†

* *Eng. News*, 1898, XL. p. 11. See also article by Patch in *Eng. News*, 1902, XLVII. p. 488 for gaugings of Sudbury and Cochituate aqueducts and effect of vegetable growth on flow.

† *Eng. Record*, 1895, XXXII. p. 223.

283. Measurement of Water Flowing in Open Channels.—In the case of small channels the discharge can be measured by means of a weir specially constructed for the purpose, which should comply with the conditions already noted on page 228. The discharge of large streams may often be obtained by noting the head on some existing dam or weir. Where such a structure does not exist, then the discharge may be found by measuring the cross-section at a suitable place and determining the average velocity of the water by the use of floats or by a

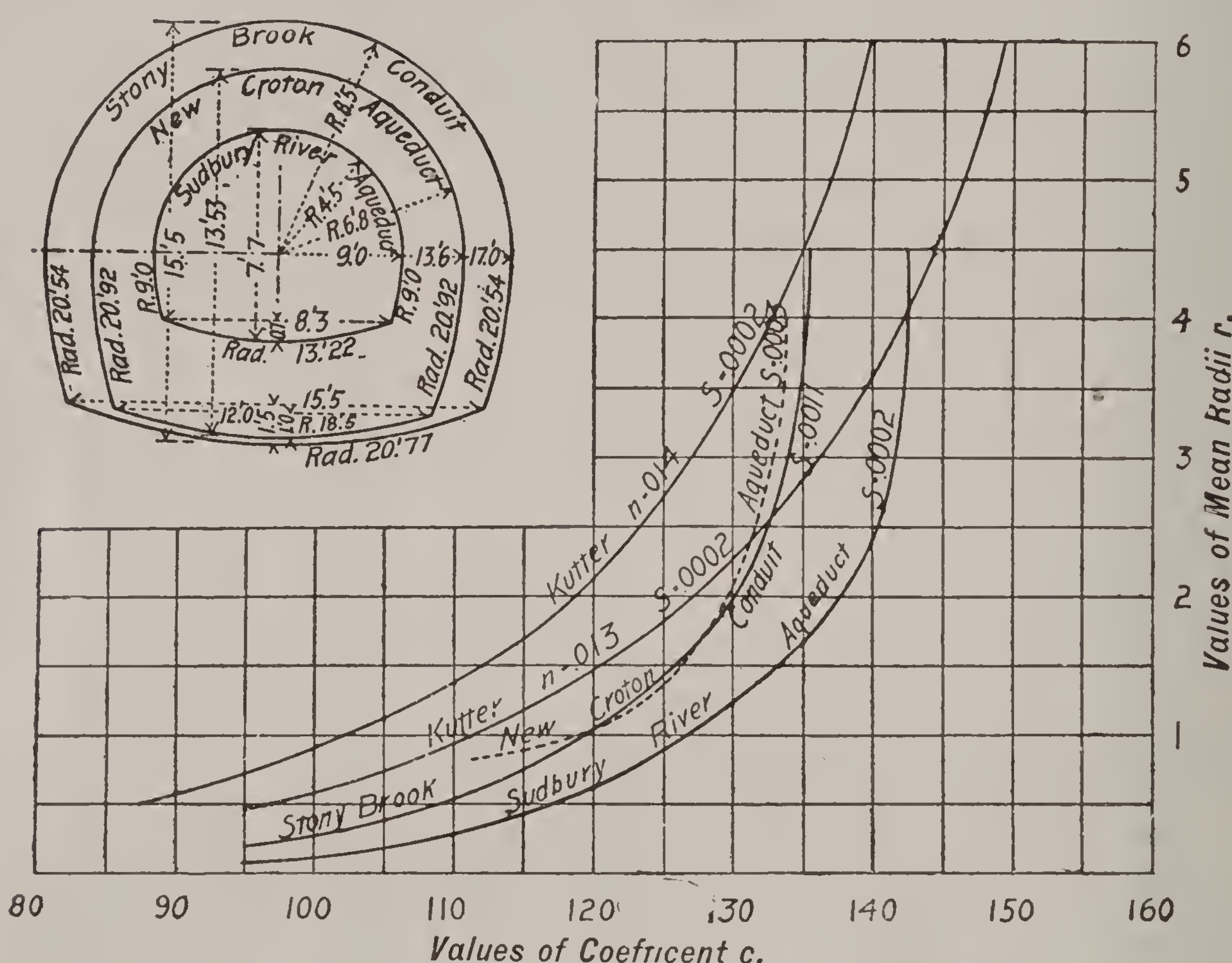

FIG. 38.—COEFFICIENTS FOR LARGE CONDUITS.

current-meter. The latter method is the most reliable. Determinations of discharge having been made at various stages of water, a discharge-curve can be drawn and subsequent values deduced from gauge records. Reliable work of this kind involves the consideration of many details which cannot be entered upon here and for which reference must be had to works on hydraulics and surveying.

NOTE.—The preceding chapter being but a very brief abstract of the more common formulas of hydraulics and of the results of experiments, no attempt is made to give a bibliography of the subject farther than is done by the footnotes throughout the chapter. These, however, will enable the student to refer to much of the most recent information on the subject. For further guide, reference should be made to such special works as Hamilton Smith, and Ganguillet and Kutter, and to the various general works on hydraulics.

A. WORKS FOR THE COLLECTION OF WATER.

CHAPTER XIII.

RIVER AND LAKE INTAKES.

284. General Conditions.—In drawing a water-supply from a natural body of water there are certain general requirements which the intake works are designed to meet. First in importance is reliability of operation, as a failure here often means the immediate shutting off of the entire supply. Another important requirement is that the point of intake should be so located as to obtain water of the best available quality. Provision should also be made for excluding fish, various floating objects, and the coarser sediment, such as sand and gravel. Finally, the construction should be an economical one.

Intake works consist of some form of conduit (pipe or tunnel) extending out to the selected point of intake, some protective works at the open end of same, and, usually, regulating-valves and screens placed at some point between the pumps and intake. If the intake-pipe is short, it may be merely an extension of the suction-pipe of the pumps; but where it is long, the usual practice is to interpose a wet-well or chamber as near the pumps as practicable and draw therefrom, long suction-pipes being disadvantageous.

Varying natural conditions give rise to important variations in arrangement and form of structures, and these will now be briefly discussed.

RIVER INTAKES.

285. Location.—The location of the intake must be selected with reference to (1) the quality of the water, and (2) the cost of construction and maintenance of the works connected therewith in so far as this is affected by the question of site. As regards quality the question of the effect of the pollution from other cities and towns higher up along the stream, and the question of the general suitability of the source, are here supposed to have been already considered and a conclusion reached in accordance with the principles discussed in preceding chapters. It is equally important that the precise point of location of the intake be decided upon with as careful consideration of these principles.

The point of intake should, first of all, be free from local sources of pollution and should therefore be located above all sewer outfalls of the town in question. In the case of tidal streams, sewage-polluted water may be carried long distances above the respective outfalls at flood tide, and before selecting the location careful study should be made of this question by means of floats and by examinations of the water at various seasons of the year. Again, it will often be found that the quality of the water is quite different along the two banks of a stream, owing to near-by sources of pollution and to the entrance of tributary waters. The location of the intake must also be determined with special reference to the lowest water-stage.

As regards the structural features, the points to be considered are: permanency of river-channel, nature of river-bed and velocity of current, suitability of adjacent ground for pumping-station and other works, and expense of conduit construction from intake to pumps and from pumps to distributing system. In the case of a stream of rapid fall the question of the head gained by going farther up-stream would be an important one.

286. Intakes in Large Streams Varying Little in Stage.—These are of the simplest character. The water may usually be taken from near the shore, the end of the intake-pipe being supported on a small foundation of concrete, or on a wooden crib, or by a masonry retaining-wall in the case of large works. In the last case some dredging may be required in front of the intake, and also wing walls built to retain the sloping bank. Gate- and screen-chambers may also be made a part of this structure, as in the intake at Philadelphia described below.

The intake-pipes, usually of cast iron, may lead directly to the pumps, thus acting as suction-pipes, or to a gate-chamber and wet-well. In the latter case the suction-pipes of the pumps lead from this wet-well. Gratings of cast iron or wood, with large openings, are usually placed at the entrance to the intake to prevent the admission of large objects, while fish-screens of copper of relatively fine mesh are inserted in the gate-house or placed over the ends of the suction-pipes.

287. *Examples.*—The Queen Lane intake of the Philadelphia water-works is illustrated in Fig. 39.* The intake here is divided into two equal parts, each half having three sluiceways 2.96 feet by 4 feet, provided with vertical

* Proc. Eng. Club Philadelphia, 1897, XIII. p. 245.

sliding-gates at the outer end. Iron screens are placed in front of the gates and held in place by masonry walls built out several feet from the face of the main wall. Two 48-inch cast-iron suction-mains lead from each division of the intake to the pumps, one for each of the four 20-million-gallon engines. It is to be noted that these suction-pipes are laid somewhat above water-level and therefore any leak would allow the entrance of air. Considerable trouble was in fact experienced from this cause, and it was not entirely overcome by recalking the joints and coating them with asphalt. The intake was constructed inside of a V-shaped coffer-dam. A channel 45 feet long extending to deep water was excavated in front of the intake.

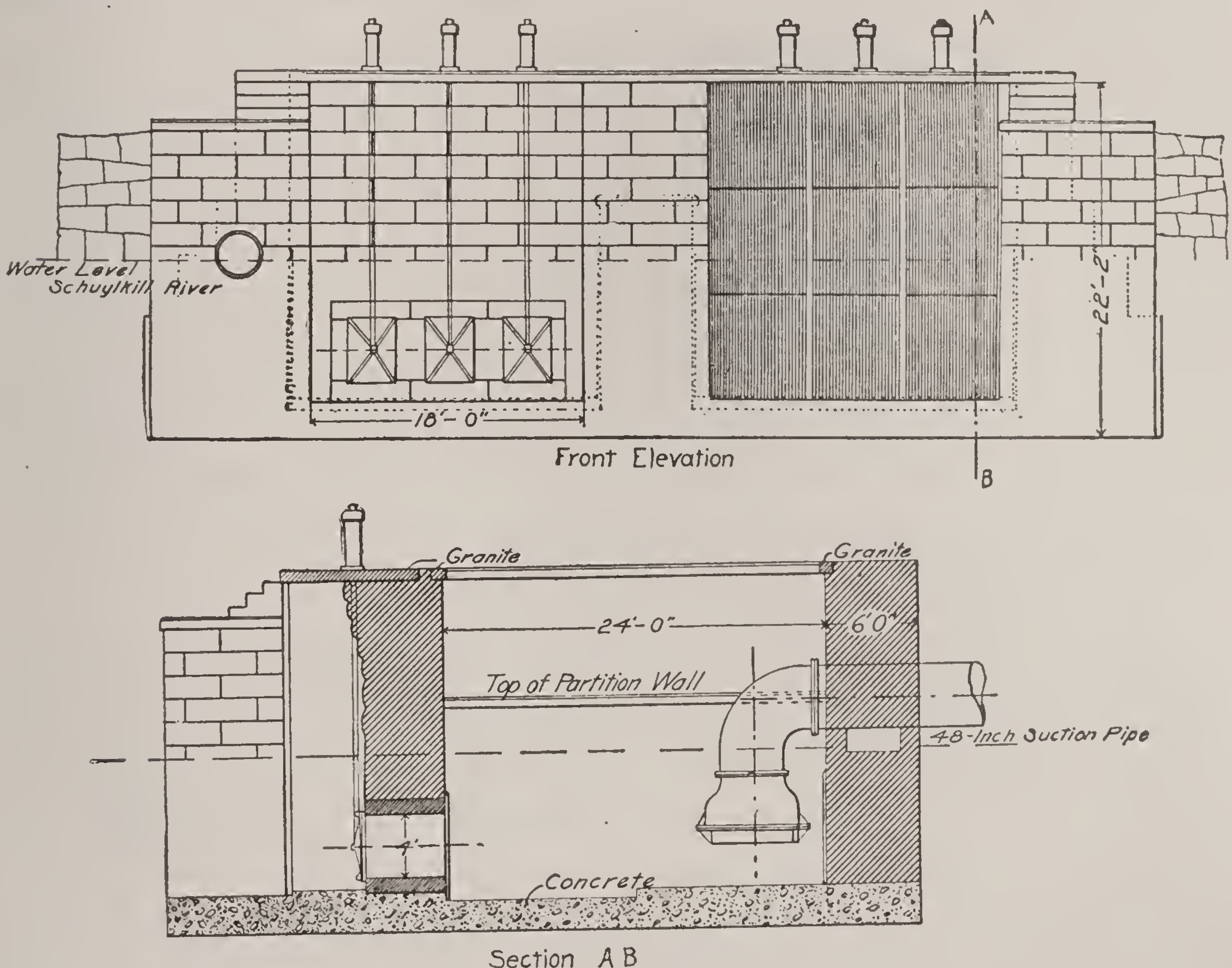

FIG. 39.—QUEEN LANE INTAKE, PHILADELPHIA.

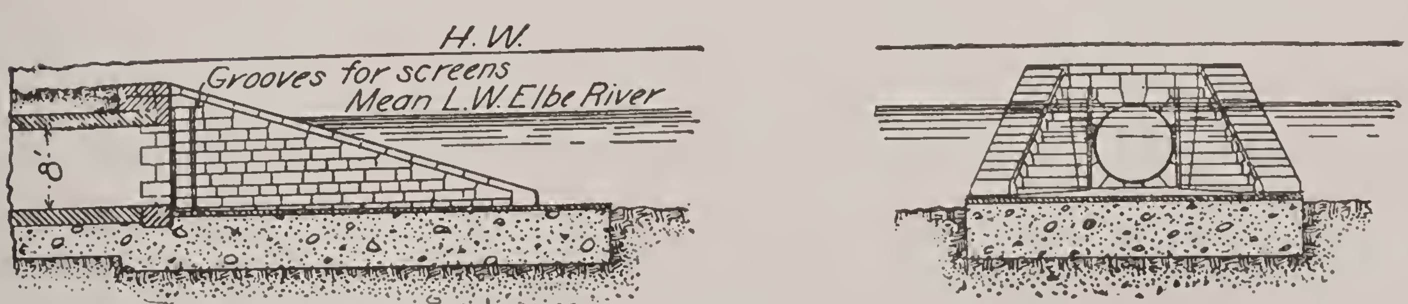

FIG. 40.—INTAKE AT HAMBURG, GERMANY.

In Fig. 40 is shown the intake of the Hamburg, Germany, Water-works.* This is also a case in which there is little variation in river stage, and the arrangement adopted is simple and substantial.

* M yter. Das Wasserwerk der freien und Hansestadt Hamburg, p. 14.

288. Intakes in Streams of Ordinary or Great Variation in Water-level.—In this case it usually becomes necessary to extend the intake-pipe a considerable distance from the banks of the stream in order to reach a suitable location at low water. Then in order to enable the pumps to reach the water at the lowest stage, which requires them to be not more than 15 or 20 feet above that level, it is often necessary to place them in a deep pump-pit much below high-water level. The construction of a water-tight pit for this purpose is then an important feature of the works. A method of avoiding this expensive feature for temporary works consists in mounting the pumps upon a car which may be moved up or down an inclined track built on the river-bank. This plan was in use for several years in the old St. Louis works.

The outer end of the intake-pipe is usually protected by a simple timber crib supporting the end of the pipe 2 or 3 feet above the river-bottom, and held in place and protected from scour by broken stone. A coarse screen or grating is ordinarily placed over that compartment of the crib containing the intake-pipe. It is desirable to have the total area of the openings of this grating 2 or 3 times that of the pipe itself in order to keep the entrance velocity low. Sometimes in order to strain out the sediment the crib is entirely filled with broken stone and sand to form a filter-crib as illustrated in Chapter XIV.

Another form of construction at the end of the intake is a masonry tower extending above high water and containing ports and sluice-gates similar in form to those used in reservoirs (Chapter XVI). To provide stability against ice and drift the tower is built similar to a bridge pier in form, the inlet ports being placed along the sides. The arrangement of interior compartments and gates is well illustrated by the St. Louis intake described in Art. 289. Heavy cast-iron gratings are bolted to the walls just outside the ports. The size of ports should be sufficient to keep the entrance velocity down to 2 or 3 feet per second.

The tower has the advantage over the crib construction in permanence and reliability. It also enables the water to be drawn from different levels, and by means of shut-off valves the intake-conduit can be emptied at any time and cleaned. For these reasons this form of construction is to be commended, but it is much more expensive than the crib construction and is therefore suited only for the larger and more important works.

From the crib or inlet-tower the intake-pipe usually runs to a wet-well, the end of the pipe being placed at least far enough below low-water level to give the head necessary for overcoming the pipe friction. It is also desirable that the pipe should be placed at all points below the hydraulic grade-line, as otherwise it will act as a siphon and require special apparatus for removing the air at intervals. From the wet-well the suction-pipes lead to the pumps. Provision for flushing the intake-pipe may be made by connecting it through a by-pass with the force-main of the distributing system.

Instead of a pipe, a tunnel may be used to conduct the water from tower to pumps, short vertical shafts connecting therewith at either end. This form of construction will be economical only in the largest works, but it is of the most permanent character.

289. *Examples.*—A typical arrangement for works situated along a stream of wide variation is that at Steubenville, Ohio, illustrated in Fig. 41.* The intake consists of two 24-inch cast-iron pipes running from a submerged crib in the Ohio River to a wet-well 15 feet in diameter and 30 feet deep. This well has a shell and top of $\frac{1}{4}$-inch boiler-steel, a Portland-cement bottom, and brick lining. From here two 16-inch suction-pipes extend through a tunnel to the pump-pit, the suction never being greater than 15 feet. Provision is also made for a third suction-pipe 20 inches in diameter. The pump-pit is far below high-water level and is thoroughly water-tight, as is the tunnel and well. It has a double wall with a filling of Portland-cement mortar, 1 to 1. The valves in the wet-well may be operated either from above or from the tunnel. Messrs. Wilkins and Davison, Pittsburgh, Pa., were the engineers.

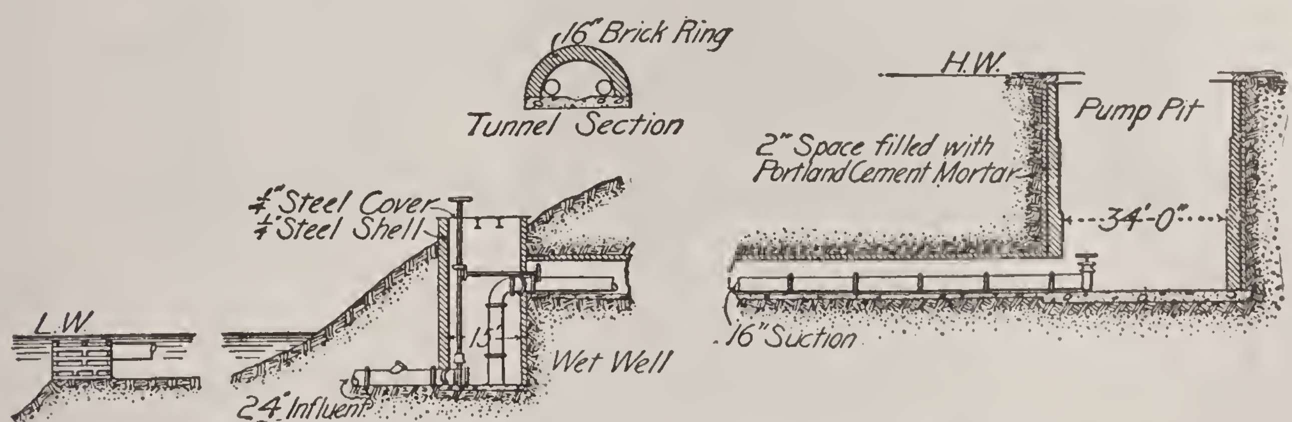

FIG. 41.—INTAKE AT STEUBENVILLE, OHIO.

The general arrangement of intake, and details of the inlet-tower, of the St. Louis Water-works are illustrated in Fig. 42. The intake here is located near the northern city limits far above all local pollution and 1500 feet from the shore. The exterior masonry of the tower is of granite. The portion subjected to the action of the floating ice is rough-pointed, and the remainder is quarry-faced. The interior is faced with limestone. Four inlets lead into the north chamber, and two into the chamber directly over the intake-shaft, but the latter are not ordinarily used. The gates are operated by hydraulic cylinders. The screen-chamber is placed on shore adjacent to the wet-well, two sets of screens being used of $\frac{1}{4}$-inch and $\frac{1}{2}$-inch mesh respectively. Unusual difficulties were encountered in sinking the crib for the inlet-tower on account of the rocky bottom and the very swift current of 6 to 8 miles per hour. In Fig. 43 are shown details of the gates of the inlet-tower.† These are again referred to in Chapter XVI.

* *Eng. Record*, 1898, XXXVIII, p. 360.

† *Eng. News*, 1891, XXVI. p. 4. *Eng. Record*, 1892, XXV. p. 319.

The new Cincinnati intake furnishes an instructive example of a modern and substantial engineering work. It is shown in Fig. 44. The inlet-tower, on account of the shape of the river-bed, is located close to the Kentucky

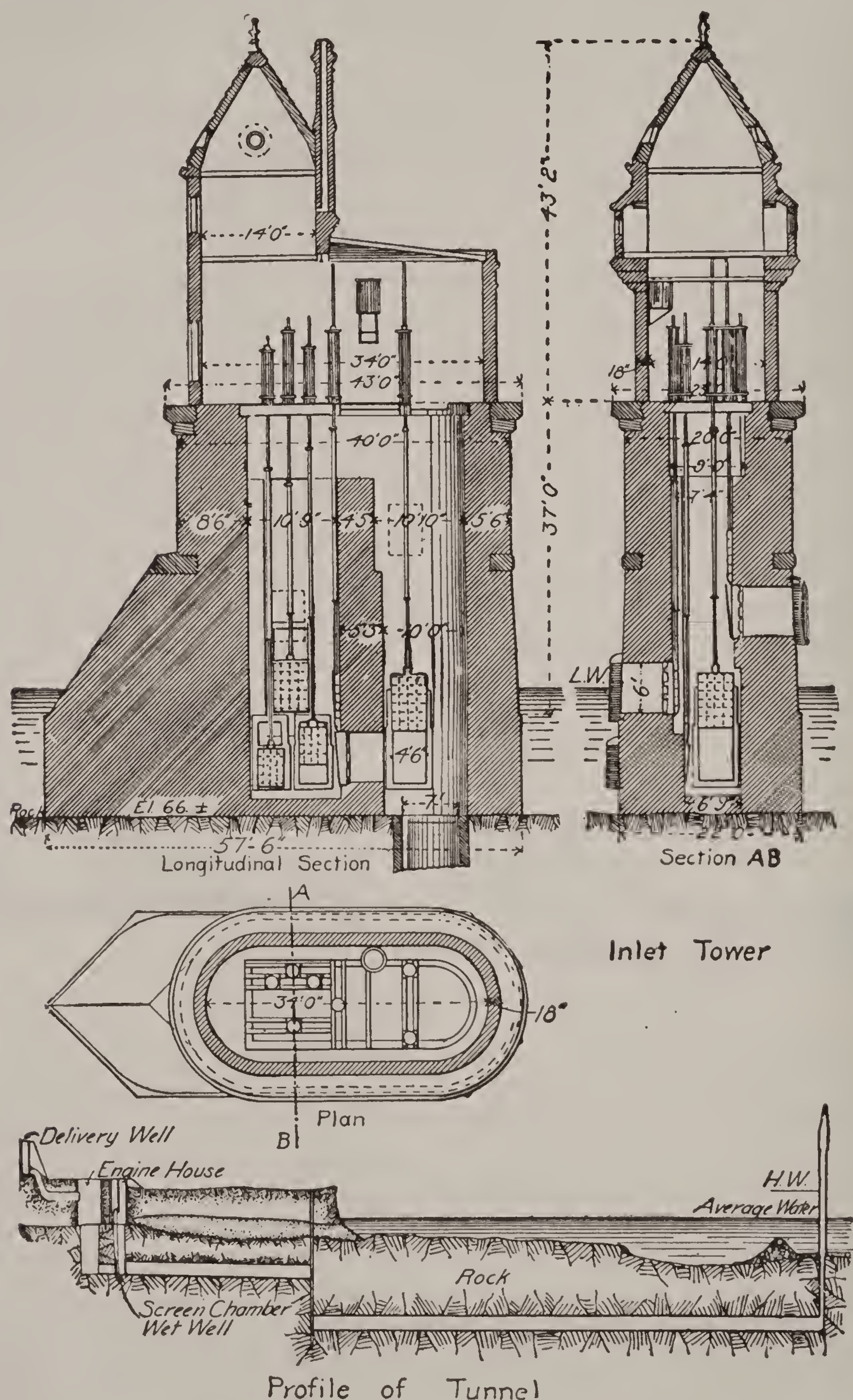

FIG. 42.—INTAKE AT ST. LOUIS.

shore. It is quite similar in general design to that at St. Louis. The tunnel is lined with two rings of brick with concrete backing; it is designed for a self-cleansing velocity of 3 feet per second. A peculiar feature is the very deep pump-pit, made necessary by the great variations in river stage (about 70

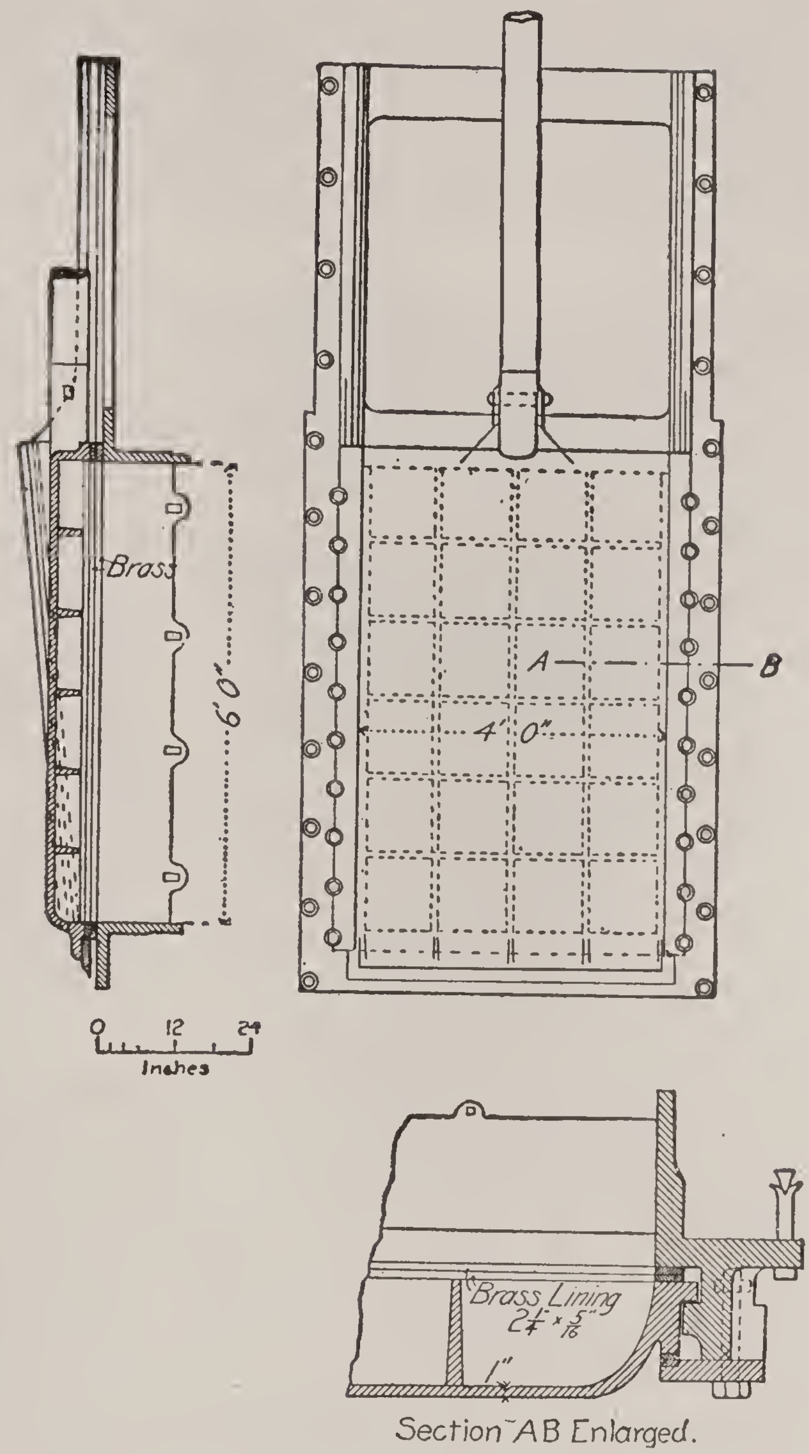

FIG. 43.—GATE DETAILS, ST. LOUIS INLET-TOWER.

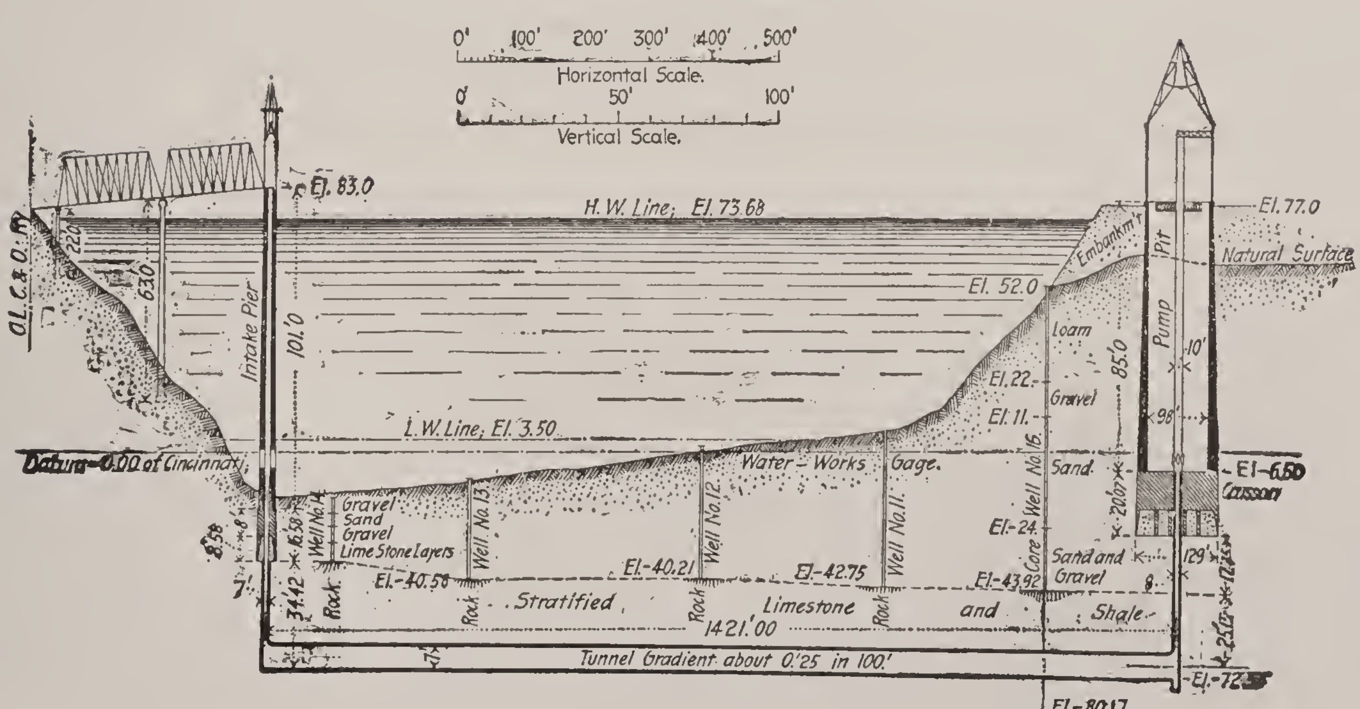

FIG. 44.—THE CINCINNATI WATER-WORKS INTAKE.
(From *Engineering News*, vol. XL.)

feet). The upper portion of the uptake-shaft is lined with $\frac{3}{4}$-inch steel plates, and this lining is carried up through the pump-pit as a steel pipe 10 feet in diameter. The suction-pipes of the pumps connect with this shaft near the floor of the pump-pit. The masonry walls are 4 feet thick at the top and 14.5 at the bottom, and to insure imperviousness a $\frac{1}{2}$-inch steel shell is built into the wall.

290. **Intake-works for Gravity Supplies.**—Where a stream has a rapid fall it may be practicable to conduct the water entirely by gravity through a canal or conduit to the place of consumption, or perhaps to filters or to pumping-stations. If the stream is small, it will usually be desirable to construct a low dam or diversion-weir impounding a small volume of water, from which reservoir the conduit may lead. A gate-house with screens and controlling gates or valves is placed at the entrance of the conduit. If coarse sediment is carried by the stream, small settling-basins should be provided near the head of the conduit, or the reservoir built large enough to act as such. (For descriptions of many works of this character see various works on irrigation.) Where the stream has a sandy or gravelly bottom it may be practicable to construct filter-galleries underneath and yet be able to convey the water entirely by gravity.

In Fig. 45 is illustrated the small diversion-weir of the Simla, India, Water-works. The stream is very small, and the weir is so arranged that only the dry-weather flow is caught, the muddy water of the floods, which flows at a relatively high velocity, leaping the opening and passing on. A somewhat similar arrangement is used at Altona, Pa. There the flood-water is conveyed in an artificial channel, in the bottom of which is a masonry gutter covered by a grating and connected to a pipe leading to the reservoir. The gutter and pipe are designed for a maximum capacity of 50 million gallons per day, and any flow in excess of this must pass on down the channel. Less can be admitted by partly closing a valve.*

LAKE INTAKES.

291. **Location.**—The location of a lake intake in such a position as to obtain at all times water of the best quality, and to fulfill the requirements of safety against interruption, is a question requiring very careful study. In a lake unpolluted by sewage some of the things to be investigated are: the location of the mouths of streams and the sediment carried by them; the character of the lake bottom; the direction of wind and currents and their effects in stirring up the mud on the

* *Jour. New Eng. W. W. Assn.*, 1899, XIV. p. 151.

lake bottom and in conveying sediment from point to point; and matters pertaining to the quality of the water, such as temperature, color, effect of stagnation, etc., as discussed in Chapter IX. (An investigation of this character carried out for the city of Syracuse on

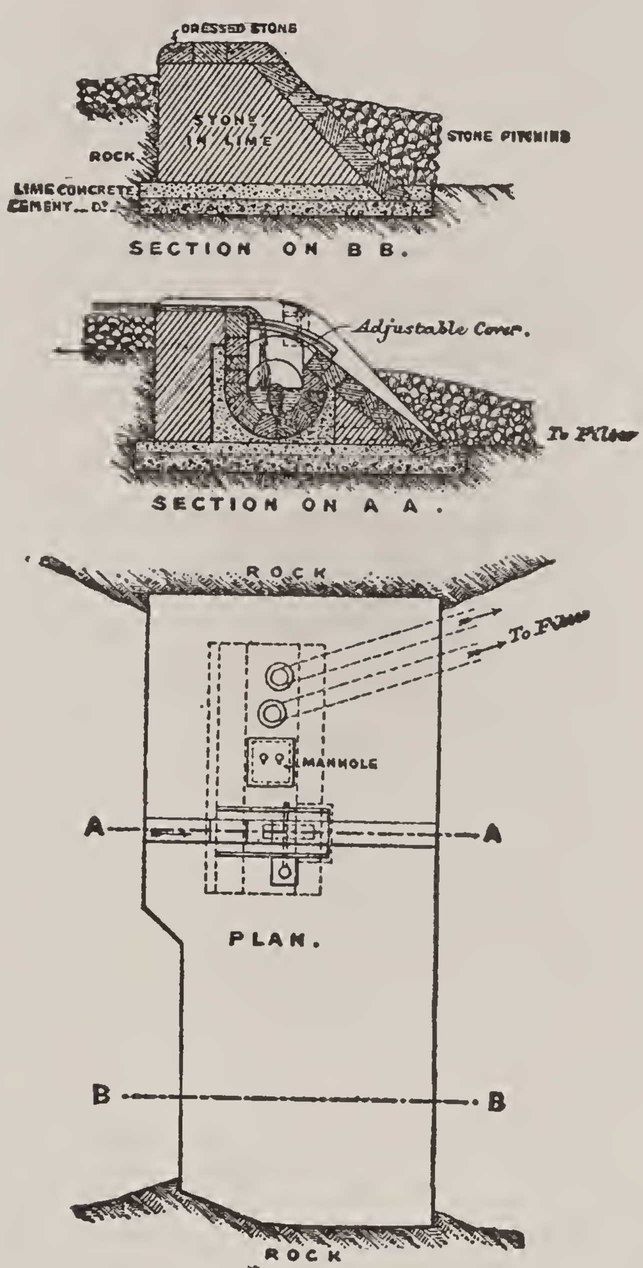

FIG. 45.—DIVERTING-WEIR OF THE SIMLA, INDIA, WATER-WORKS.
(From Proc. Inst. Civil Engineers, vol. CXXXII.)

Skaneateles Lake, a body of very pure water, involved the taking of over 3000 soundings.)

The intake should if practicable be located at a sufficient depth to be free from any considerable wave-action, both to secure greater stability and to avoid the effect of the disturbance of the sediment by the waves. It was shown in Chapter IX that even in small ponds the wind stirs up the water to a depth of 15 or 20 feet, so that this may be taken as about the minimum depth. A greater depth is desirable if bad effects of stagnation are not present, since the water becomes rapidly cooler below this point. In large lakes the wave-action extends to much greater depths and the intake should be extended accordingly to depths of 40 or 50 feet. In such large bodies of water, bad odors from stagnation are little to be feared. Where the water is shallow for a long distance from shore, as along Lake Michigan, and especially Lake Erie, the best length of intake-conduit becomes too great to be afforded by any but the largest cities.

Most of the cities along the Great Lakes dispose of their sewage by running it directly into the lake at the most convenient point; and for those places that draw their water-supply from the same body of water the most difficult part of the intake problem is to exclude their own sewage. As the cities grow, the intakes are pushed farther and farther out, but usually not until the necessity of the step is brought home by increased mortality from typhoid fever; and, however carefully this matter is followed up, the quality of the water taken from such sources must always be looked upon with suspicion. In Chicago the length of intake has gradually increased to 4 miles. In Milwaukee it is $1\frac{1}{2}$ miles, while the new intake at Cleveland is about 5 miles long.

292. **The Intake-conduit.**—Whether the conduit should be a pipe-line or a tunnel depends upon the cost of construction and the relative reliability of the two forms. In small works the cost of a tunnel would be prohibitory, while in the case of a very large intake a tunnel may be the cheaper. Again, a pipe-line, unless sunk very deep, is subject to disturbances near the shore end by ice action, wreckage, and scour from storms. The best solution may consist of a combination of the two, as at Milwaukee, where a pipe is used at the outer end and a tunnel at the shore end.

The size of the conduit will be largely controlled by the permissible loss of head from intake to pumps, and this in turn will depend upon the available depth of suction and upon the economy of construction of conduit, wet-well, and pump-pit. In very long intakes this will necessitate low velocities and large sizes.

The methods employed in executing tunnel work are similar to those in other cases where water is to be feared. Excavation is usually carried on from the intake-shaft, and often from one or more intermediate shafts sunk by the use of large wooden cribs having interior wells. Soft strata are penetrated by cast-iron or steel linings, with or without the use of compressed air as the case may require.

Submerged-pipe intakes are usually laid by the aid of divers, although other methods have been used. The pipe is preferably laid in a dredged trench, at least as far out as wave-action is to be feared, and should be covered generally to a depth of 3 or 4 feet. Near the shore end the covering should be considerably deeper than this. In some instances the pipe has not been covered, but held in place by piling or by special anchor-cribs. Various methods of laying submerged pipe are described in Chapter XXIV. Pipes have sometimes lifted on account of being emptied of water, but this is unusual and cannot happen if the shore end rises above the submerged portion by an amount equal to the diameter of the pipe.

Both cast-iron and riveted steel pipe have been used for intakes. Their relative advantages depend upon durability, convenience in handling, and cost. Steel is lighter and easier to handle, but at the same time more easily disturbed when laid.

293. **Protection-works.** — The greater number of lake intakes are protected by submerged cribs, but a few of the largest, notably those at Chicago and the new intake at Cleveland, have large exposed cribs. All these protect shafts at the ends of tunnels. Such cribs are much more expensive than submerged ones and require constant attendance after completion, but in the case of tunnel intakes an exposed crib is necessary in the construction of the end shaft, and to make it permanent is of great advantage in case of future extensions. It also enables water to be drawn at different levels. On the whole, however, the economy of this form may be doubted; and in the case of the Cleveland intake a submerged crib was recommended by a commission of engineers, consisting of Messrs. Rudolph Hering, G. H. Benzenberg, and Desmond FitzGerald, chiefly on the grounds of expense and of trouble with ice. Comparing a submerged crib with an exposed one they say: * "A submerged crib, on the other hand, say 10 feet in height, in 53 feet of water allows the free passage of ice on the surface, and uninterrupted access for the water. In a lake the size of Lake Erie stagnation effects would hardly occur in such a position, and the water will always be of excellent quality near the bottom. We, therefore, recommend a submerged crib for the intake." Also: "It is important that the velocity of the water, where it enters the crib, should be reduced to but 3 or 4 inches per second, and that the area of ingress be sufficient to produce this result. The evident consequence will be that less floating matter will be drawn into the crib." In a report on this subject to the city of Buffalo, Mr. E. B. Guthrie recommends the submerged crib on practically the same grounds.

To avoid the entrance of the coarser sediment the open end of the intake of the lower port-holes of a closed crib should be 6 or 8 feet above the bottom of the lake.

294. **Obstruction of Intakes by Anchor-ice.** — The greatest difficulty

* Abstract of report in *Eng. News*, 1896, XXXV. p. 117.

met with in operating lake intakes is due to the clogging of the ports by anchor ice or frazil ice. Frazil ice consists of needles of ice which form in open, moving water, and which on account of their small size are readily carried below the surface by comparatively weak currents. Anchor ice forms directly upon submerged objects in shallow, open water, due to excessive heat radiation such as occurs on cold, clear nights. Both anchor and frazil ice are apt to give much trouble at exposed cribs and shallow, submerged ones, by forming upon the bars of racks and port holes, and especially upon surfaces of metal. The trouble is met in various ways. The most effective method, where practicable, is the use of steam, as a very small rise of temperature of the exposed surfaces is sufficient to overcome the difficulty. Compressed air, chains drawn back and forth through the ports, axes and pike-poles are some of the other means used. Anchor and frazil ice do not form where a surface sheet has formed.

As tending to obviate the difficulty with anchor-ice, large port area and deep ports should be used; and in the later cribs this feature has been observed, the ratio of area of ports to tunnel being about four in the later Chicago cribs and eight in the new Cleveland crib. This is of equal or greater importance in submerged cribs. In this connection note the recommendation of a velocity of 3 or 4 inches per second mentioned in the preceding article.

Anchor-ice is often formed in Northern rivers at points of high velocity, but trouble with river intakes may usually be obviated by

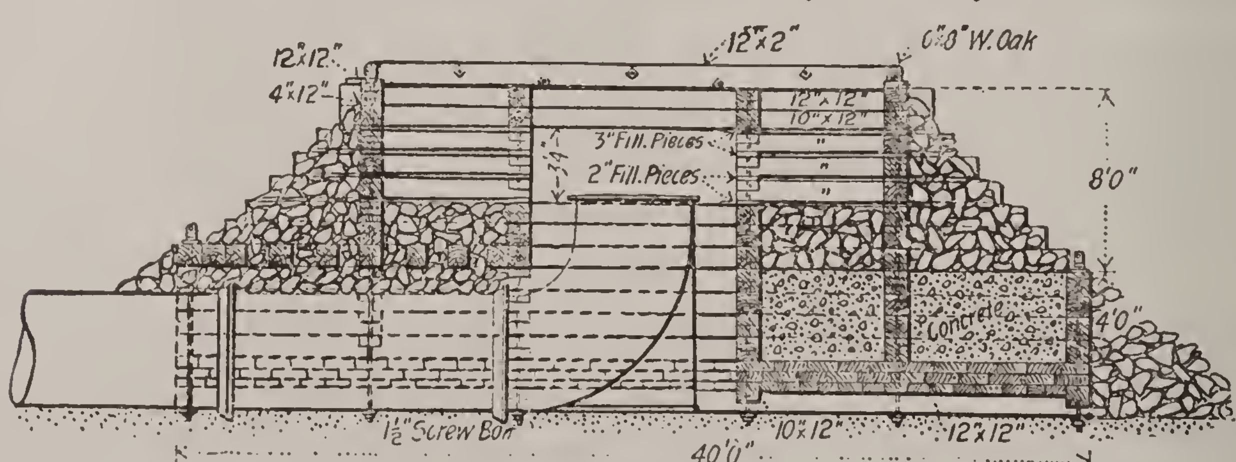

FIG. 46.—SUBMERGED CRIB, MILWAUKEE INTAKE.
(From *Engineering News*, vol. XXXIV.)

locating the intake at a point where the surface will readily freeze over. If this cannot be done, then measures similar to those employed on the lakes must be adopted. A method which has been used to advantage in dealing with anchor-ice, and one which is applicable to intakes near the shore of streams, small lakes, or reservoirs, is to create a quiet body of water for some distance around the inlet by means of a raft or boom of logs.

295. *Examples of Lake Intakes.*—It has already been mentioned that the Milwaukee intake consists partly of tunnel and partly of cast-iron pipe. At the junction of these portions is placed an exposed wooden crib with concrete filling, which is provided with emergency inlets. At the outer end of the pipe-line is a submerged crib: this is illustrated in Fig. 46. The compartment into which the pipe opens is covered with a wooden grating of 2 × 12-inch planks with 2-inch spaces between, giving 200 square feet of opening, or about ten times the pipe cross-section.

Fig. 47 illustrates the new 2½-mile crib of the Chicago Water-works. It is circular in plan and has a central well 60 feet in diameter with a timber floor 6 feet thick. The bottom 20 feet is of hemlock timber, and above this is a steel shell filled with concrete. After the crib was sunk in place, holes

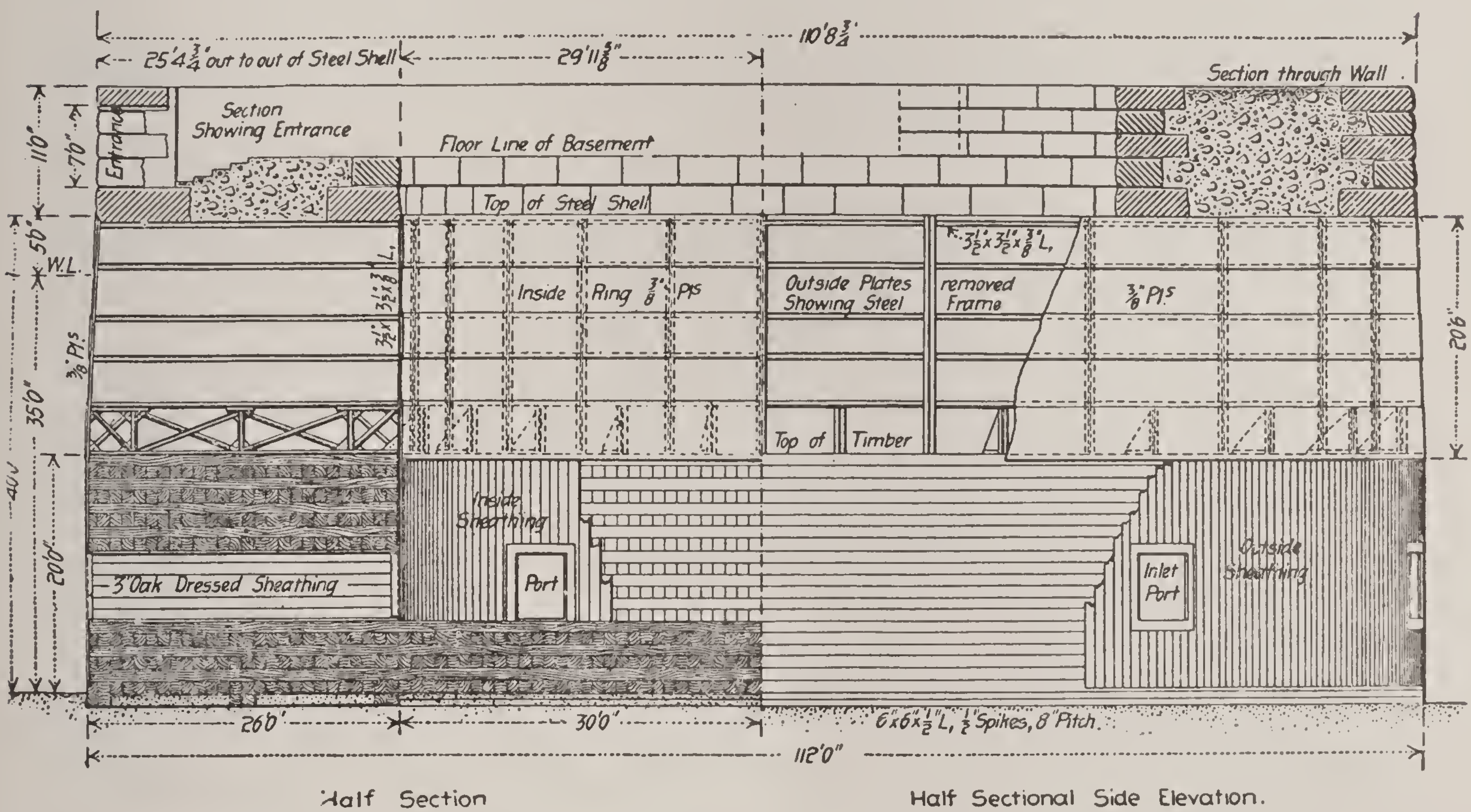

FIG. 47.—NEW 2½-MILE INTAKE-CRIB, CHICAGO.
(From *Engineering News*, vol. XLII.)

were cut through the timber bottom and two cast-iron shafts 12 feet in diameter were sunk to a depth of 61 feet, below which the lining of the shaft is of brick. Water is admitted to the interior well through eight ports 6 × 6 feet, located 6 feet above the lake bottom or about 30 feet below the water-surface, and at that depth it is thought that trouble from anchor-ice will be avoided. From the well the water passes into each shaft through three gates 4½ × 6 feet. The shafts connect with 10-foot tunnels. The superstructure of the crib includes quarters for the attendants, light-house, boiler- and engine-rooms, etc. The total cost was about $200,000.*

* *Eng. News*, 1899, XLII. p. 139.

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CHAPTER XIV.

WORKS FOR THE COLLECTION OF GROUND-WATER.

296. **Classification.**—The various forms of works built for the collection of ground-waters may be divided into the following classes:

(1) Works for utilizing the flow of springs;
(2) Shallow wells, including ordinary dug wells and tubular wells;
(3) Deep and artesian wells.
(4) Horizontal galleries and wells;

WORKS FOR UTILIZING THE FLOW FROM SPRINGS.

297. **Objects to be Attained.**—The chief objects to be accomplished in the construction of works of the kind here considered are, the protection of the water from pollution and the spring from injury through clogging or otherwise, the furnishing of a convenient chamber from which the conduit-pipes may lead, and, in some cases, the enlargement of the yield by suitable forms of construction. Besides these, other minor objects are sometimes provided for, according to the necessities of the case, such as gate-chambers, settling-basins, measuring-weirs, etc.

298. **Ordinary Forms of Collecting-basins.**—If a supply sufficient at all times for the demand can be obtained from one or more large springs, each one should have its separate basin from which the water may be conducted to a common main. The simplest form of works consists of a small masonry well or basin surrounding the spring and from which the conduit-pipe leads. To prevent a growth of vegetable organisms and consequent deterioration of the water, such basin should always be covered so as to exclude the light. For a small spring, a circular well covered with a stone cap cemented in place and provided with a manhole is a simple and effective arrangement. For larger springs a masonry vault covered with 2 or 3 feet of earth is preferable. If the spring is located on a steep hillside, the collecting-

chamber is conveniently constructed in the form of a horizontal gallery built into the hill, access to which is had through a door or manhole.

Overflow-pipes leading into drains or open channels should be provided for, and to facilitate cleaning and repairs a waste-pipe with valve may also be put in, through which the basin can be emptied. Gates or valves should also be provided in the conduit-pipe. Weir-chambers with suitable floats are an inexpensive but valuable feature, as they enable complete records of the yield to be easily obtained. If the water carries fine sand in suspension, the basin should be made large enough to permit this to settle.

Mineral and other springs occurring in public places usually have open basins, and opportunities are offered in the walls and parapets for ornamentation.

Examples.—In Fig. 48 is shown a simple covered basin. It is a type of those used in protecting the Vanne supply of Paris. The

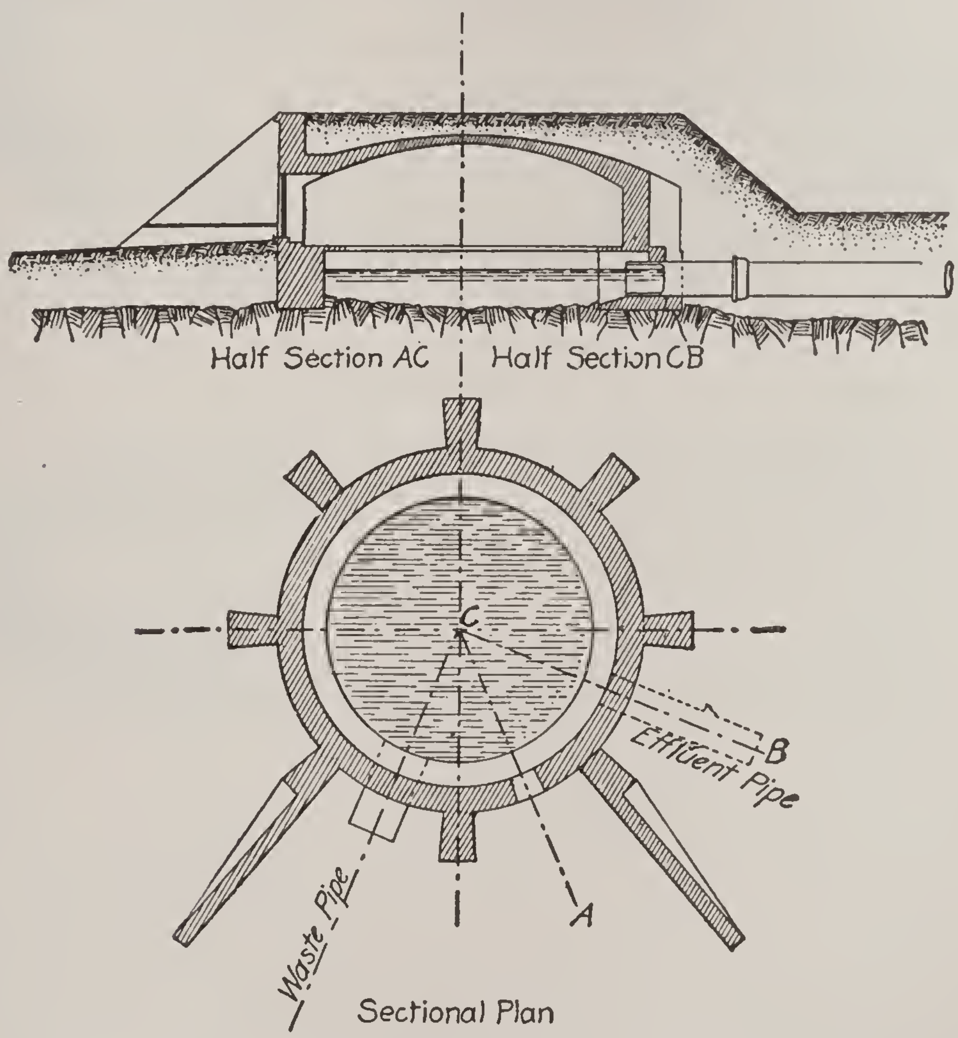

FIG. 48.—COLLECTING-BASIN, VANNE SUPPLY, PARIS.

ground here is quite level. Fig. 49 shows a collecting-chamber on a side hill for the water-supply of the city of Lahr. It contains weir, settling-chamber, conduit-pipe with strainer, overflow- and waste-pipes.*

* Lueger. Die Wasserversorgung der Städte, p. 397.

299. **Methods of Increasing the Flow.**—If the natural yield of a spring is insufficient, it will sometimes be possible to increase it. The proper form of collecting works to accomplish this depends upon the character of the spring. It will be here convenient to treat the springs under the same classification as in Chapter VII.

300. *Springs of the First Class.*—In this class the water appears at the upper surface of a stratum of impervious material overlaid by the water-bearing deposit, frequently in the form of several small springs. Instead of dealing with each one individually it will often be better to

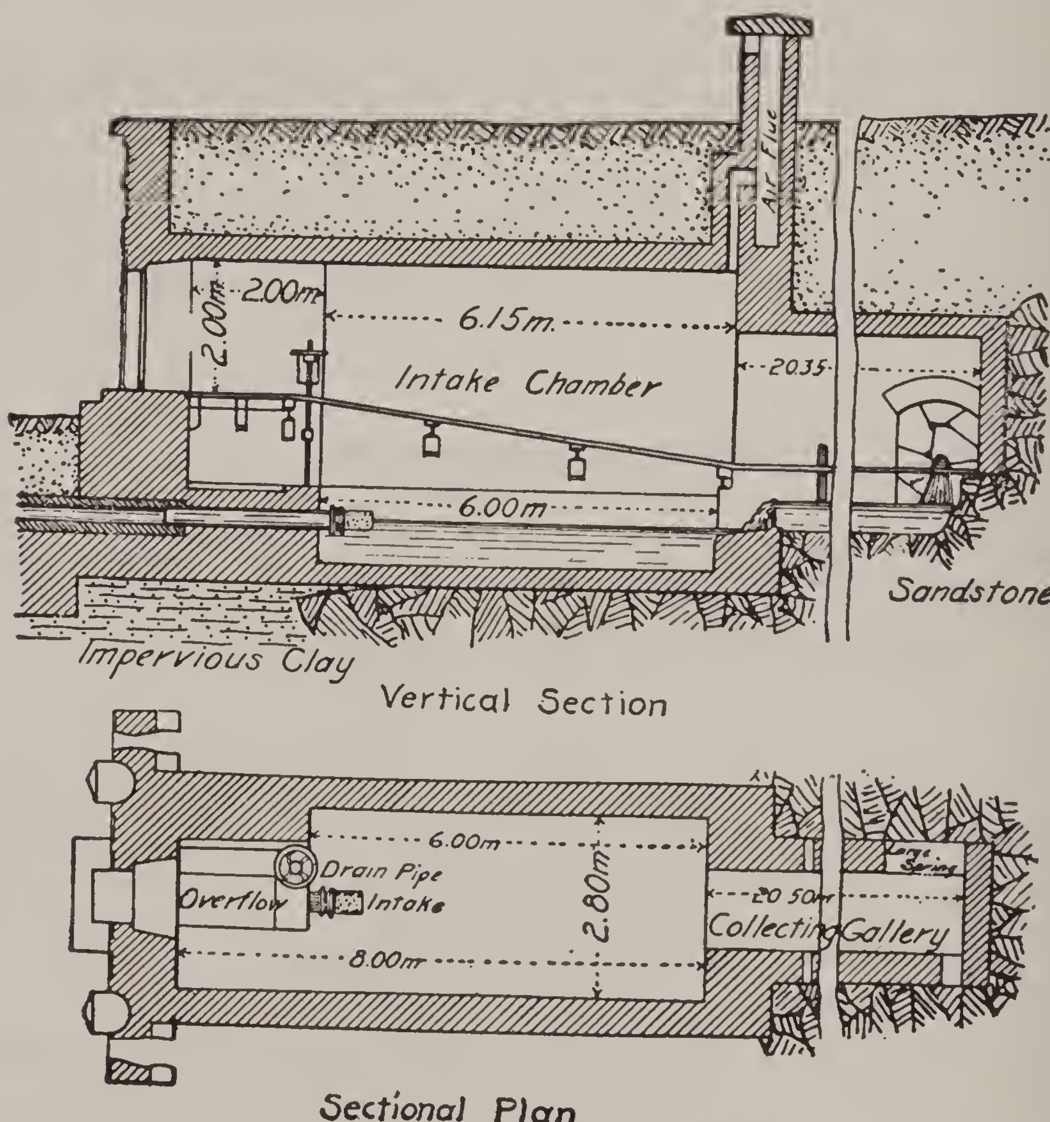

FIG. 49.—COLLECTING-BASIN, CITY OF LAHR. (Lueger.)

construct a long collecting-gallery running parallel to the outcrop and leading to a central collecting-chamber which can be made similar in form to that for a large spring. This gallery, which is made similar to those described in Art. 356, should be built deep enough to rest upon the impervious material, and thus to collect all the underground flowage as well as that appearing as springs. The total yield may be thus much increased, the increase being relatively greatest during dry weather.

In the case of a single large spring the flow can sometimes be increased by opening up the water-passages for some distance into the hill, thus decreasing the resistance to flow and possibly drawing from a larger area. Such a procedure is likely at the same time to make the flow more irregular by drawing more rapidly on the storage capacity of the ground, and this plan should hence not be adopted without careful consideration.

301. *Springs of the Second Class*, or those where an impervious layer covers to a greater or less extent the water-bearing stratum.—Such a spring may represent but a part of the ground-water flow, and if borings indicate a ground-water stream of considerable extent, the collecting-works may be arranged without much reference to the spring. Such works will ordinarily be some form of well or gallery like those described in subsequent articles.

In the case of one of the springs described in Art. 92, page 104, a well was sunk near by, and after sealing the spring the water rose in the well 3 feet higher than before, and 5 feet above the surface of the adjacent ground. Extended tests indicated considerably increased yield over that of the spring. A similar spring indicated by tests, after the construction of collecting-works, an average yield of about 440,000 gallons per day, as compared to an average previous flow of 275,000 gallons. In this case the "well is circular, 22 feet in average diameter, 24 feet deep, built of open-jointed rubble masonry with a lining of brick laid in cement mortar for the upper 18 feet, and is surmounted by a conical shingle roof. It was built at the largest and highest of the three springs, and the sites of the other two were sealed over by beds of concrete, while the water was kept down by pumping from the well. The outlet is 1.5 feet below the top of the well, which overflows between times of pumping." *

302. *The Third Class of Springs*, which are mere overflows of ground-water in a porous formation, are to be treated like those of the second class. The ground-water streams of which they are the indications may frequently be drawn upon to advantage by wells or galleries arranged with little reference to the springs themselves.

THE HYDRAULICS OF WELLS.

303. Before entering upon a discussion of the various forms of wells it will be desirable to consider the hydraulic principles governing the flow of water into them from the surrounding porous formations. There are two general cases to be considered: (1) Flow into ordinary wells, where the upper surface of the ground-water is exposed to at-

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* *Jour. New Eng. W. W. Assn.*, 1896, XI. p. 156.

mospheric pressure through the porous ground above. (2) Flow into artesian wells, where an impervious layer covers the porous one, thus enabling the water to flow under a pressure greater or less than the atmosphere. General formulas relating to these two cases will be discussed separately, after which matters common to both will be considered.

A. *Principles Governing the Flow into Ordinary Wells and Galleries.*

304. **General Form of Ground-water Surface.** — If a well, sunk into a body of ground-water, be drawn from, the level of the water in the well will be lowered, and the surface of the ground-water adjacent to the well will assume a form similar to that shown in Fig. 50. In this,

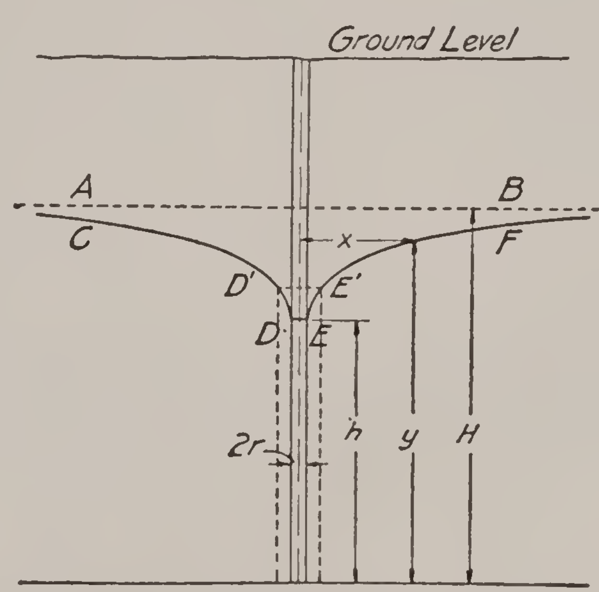

FIG. 50. — SECTION THROUGH WELL.

AB is the original surface and *CDEF* the new surface. The amount which the surface is lowered decreases rapidly as we get farther from the well, until at some point more or less remote there is no sensible effect. The area within which the level is appreciably lowered is called the *circle of influence.*

If the ground-water is present merely as a pond or reservoir and the pumpage exceeds the percolation on the area, the circle of influence will gradually enlarge until it includes the entire area of the pond, and the water will in time be exhausted. If there is, however, a general flow of the ground-water, a well will be lowered only until the circle of influence has broadened out far enough to cause to be tributary to the well an area into which the flow of water is equal to the pumpage.

305. **Derivation of Formula for Flow.**—In Fig. 50 let it be assumed that *AB*, the original surface of the ground-water, is horizontal and at a uniform distance *H* above an impervious stratum; that the porous material is uniform; and that the well is sunk to the impervious stratum.

Let r = radius of well, h = depth of water in the well when in operation, H = original depth of ground-water, x and y = co-ordinates of any point of the curve CF referred to the bottom of the well as origin, and Q = rate of flow into the well, or the yield.

The total available head, as represented by $H-h$, is consumed in four ways: first, and mainly, by the resistance to flow in the ground; second, by the entrance resistance into the well-tube or well; third, by friction in the well-tube in ascending to DE; and fourth, by the head necessary to give the rising water its velocity. For shallow wells all but the first are usually very small, and for the present they will be neglected. Their effects in exceptional cases are noted farther on.

The equation of the curve $CD - EF$ will now be derived. The flow being radial, the area of the cross-section through which the water passes at the rate Q at any distance x from the center is that of a cylindrical surface equal to $2\pi xy$. In Chapter VII, Arts. 85 and 88, it was shown that Q in cubic feet per day $= ksAp$, where k = a constant for the particular sand in question (see Table No. 20, Art. 85), s = slope, A = area of cross-section in square feet, and p = porosity.

In this case $A = 2\pi xy$ and $s = \frac{dy}{dx}$, whence

$$Q = 2\pi kpxy\frac{dy}{dx}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

Writing this in the form $Q\frac{dx}{x} = 2\pi kpydy$, and integrating, we have

$$Q \log_e x = \pi kpy^2 + C, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

in which $\log_e x$ is the natural or hyperbolic logarithm of x.

When $x = r$, $y = h$, whence we find $C = Q \log_e r - \pi kph^2$, and substituting and solving for y^2 we have

$$y^2 = \frac{Q}{\pi kp}\log_e\frac{x}{r} + h^2, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

which is the equation sought. The units are the foot and day.

This formula assumes the water to flow towards the well from an indefinite distance, and the curve therefore continues to rise indefinitely, but more and more slowly as we recede from the well. In the actual case the circle of influence is limited on account of the flow of the body of ground-water, this flow being maintained by percolation either near or remote. Furthermore, on account of the slope of the ground-water surface, the curve will be modified, being steeper on the up-stream and flatter on the down-stream side. It will also be more or less irregular on account of variations in the porosity of the ground. But the general form of the curve as determined by actual measurements agrees quite closely with the theoretical curve, and valuable general conclusions may be drawn from a theoretical consideration of the subject.

If in equation (3) R be that value of x for which the change in water-level is inappreciable, equal to the radius of the circle of influence, the corresponding value of y will be H, the original depth of water, and we have

$$H^2 = \frac{Q}{\pi kp} \log_e \frac{R}{r} + h^2,$$

and solving, we get for Q in cubic feet per day

$$Q = \pi kp \frac{H^2 - h^2}{\log_e \frac{R}{r}} = \frac{\pi kp}{2.30} \cdot \frac{H^2 - h^2}{\log_{10} \frac{R}{r}} \quad . \quad . \quad . \quad . \quad . \quad (4)$$

or in gallons per day,

$$Q = \frac{\pi kp \times 7.5}{2.30} \cdot \frac{H^2 - h^2}{\log \frac{R}{r}} = k' \frac{H^2 - h^2}{\log \frac{R}{r}} \quad . \quad . \quad (5)$$

in which
$$k' = \frac{\pi kp \times 7.5}{2.30},$$

a constant depending upon the fineness and the porosity of the material. All distances should be expressed in feet.

306. Calculation of Flow.—In Table No. 52 are given values of k' for various values of porosity and size of sand-grain. Table No. 53 contains values of the quantity $\dfrac{1}{\log \frac{R}{r}}$ of equation (5); they are also the value of Q for $k' = 1$ and for $H^2 - h^2 = 1$. To find Q for any given value of k' and of $H^2 - h^2$ multiply the quantities in the table by $k' \times (H^2 - h^2)$.

TABLE NO. 52.

VALUES OF k' IN THE FORMULA $Q = k' \dfrac{H^2 - h^2}{\log \frac{R}{r}}$.

Porosity Per cent.	(d) Effective Size of Sand in Millimeters.									Porosity Per cent.
	.10	.20	.30	.40	.50	.80	1.00	2.00	3.00	
25	71	286	643	1,140	1,785	4,560	7,140	28,600	64,250	25
30	132	525	1,180	2,090	3,270	8,380	13,200	52,500	117,900	30
35	218	870	1,955	3,470	5,430	13,900	21,840	87,000	195,500	35
40	336	1,345	3,025	5,380	8,400	21,525	33,600	134,500	302,600	40

TABLE NO. 53.

VALUES OF $\left(\frac{1}{\log\frac{R}{r}}\right)$ IN EQUATION (5)

R Feet.	Diameter of Well.									
	2 in.	4 in.	6 in.	8 in.	12 in.	2 ft.	4 ft.	10 ft.	20 ft.	40 ft.
100	.325	.360	.384	.403	.435	.500	.588	.770	1.000	1.430
200	.296	.325	.344	.360	.384	.435	.500	.625	.770	1.000
500	.265	.287	.303	.315	.333	.370	.416	.500	.588	.715
1000	.245	.265	.278	.287	.303	.333	.370	.435	.500	.588
2000	.228	.245	.256	.265	.278	.303	.333	.385	.435	.500
5000	.209	.223	.233	.239	.250	.270	.294	.333	.370	.417
10000	.197	.209	.217	.223	.233	.250	.270	.303	.333	.370

The formula or tables will enable approximate values of Q to be determined if all the other quantities are known. With shallow deposits no great difficulty arises in estimating rough values for k', p, and H; h is determined by the conditions under which the well is to be operated, and r is the known radius of the well.

307. *The Value of R.*—In none of the above quantities is there anything that involves the amount of water actually flowing in the ground, and it is obvious that without some knowledge of this no formula will enable one to predict the yield of a well. The effect of this element is all included in the value of R, the radius of the circle of influence, and it is in the determination of this that the chief difficulty arises; but it will be noted from Table No. 53 that large variations in R affect Q but little, so that a rough approximation will be sufficient. This can be obtained by properly conducted tests as explained in Art. 316, or it can be estimated as follows: Assuming that all the water in the circle of influence flows into the well, the width of the strip of the ground-water stream tributary to the well will be $2R$, and the original cross-section of this portion of the ground-water stream is $2RH$. Then, as on page 279, the quantity $Q = ks \times 2RH \times p$, whence

$$R = \frac{Q}{2ksHp}. \qquad (6)$$

By substituting the value of Q from equation (4) we have, after reduction,

$$R = \frac{\pi(H^2 - h^2)}{2sH\log_e\frac{R}{r}}, \qquad (7)$$

or, as $H - h$ is usually small compared to H, we have approximately

$$R = \frac{\pi(H-h)}{s \log_e \frac{R}{r}} = 1.36 \frac{H-h}{s \log_{10} \frac{R}{r}}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (8)$$

from which, knowing the slope s and the depression of the water-level ($H = h$), R can be estimated with sufficient accuracy by a few trials.

It is to be noted that R varies inversely with the slope, and from eq. (5) it is seen that Q increases as R decreases; hence for a given value of $(H^2 - h^2)$, Q will be greater the greater the slope, and with zero slope Q will be zero. This is an important point; it expresses mathematically what has been stated in Chapter VII, that there must be an actual flow of the ground-water as shown by an hydraulic slope in order that any definite quantity can be withdrawn for an indefinite length of time.

Table No. 54 gives values of R from eq. (8) for various values of r and of $\frac{H-h}{s}$.

TABLE NO. 54.

VALUES OF R IN FORMULA $R = 1.36 \frac{H-h}{s \log \frac{R}{r}}$ FOR VARIOUS VALUES OF $\frac{H-h}{s}$ AND OF r.

$\frac{H-h}{s}$.	Diameter of Well ($=2r$).					
	6 Inches.	1 Foot.	2 Feet.	4 Feet.	10 Feet.	20 Feet.
200	104	115	129	146	176	207
400	189	208	230	258	305	352
600	269	295	325	362	423	485
800	347	378	415	461	536	610
1,000	422	459	503	556	645	730
2,000	779	843	919	1010	1150	1290
4,000	1450	1560	1690	1840	2080	2300
6,000	2080	2240	2410	2620	2950	3250
8,000	2700	2890	3120	3370	3780	4150
10,000	3280	3530	3800	4110	4590	5030

Considering the values given in this table and the relatively slight effect of a large variation in R, shown by Table 53, it will be sufficiently accurate in most cases to take an arbitrary value of R such as 1000. In the nature of the case the results of such calculations must be looked upon as only crude approximations which serve, however, as a general guide and as a check upon unreasonable estimates.

308. *Example.* — To apply these tables to an example let it be required to estimate the yield of a 6-inch well sunk into a ground-water stream 30 feet thick, the water-bearing stratum consisting of a coarse sand of an effective size of 0.4 millimeters and a porosity of 35 per cent. Further, suppose the slope $s = 20$ feet per mile $= .0038$, and that the water is to be drawn down 5 feet below its original level. Then $H - h = 5$, $h = 25$, $\dfrac{H-h}{s} = 1320$, and $H^2 - h^2 = 275$. From Table No. 54 we find for $\dfrac{H-h}{s} = 1320$, $R =$ about 500 feet. From Table No. 52 we find for $d = 0.4$ and $p = 35$ per cent, $k' = 3500$.

Finally, by the aid of Table No. 53, we find a value of Q equal to $0.30 \times 3500 \times 275 = 290{,}000$ gallons per day. If R had been taken at 1000 the result would have been 270,000 gallons per day.

309. Effect on the Yield of a Change in the Various Elements. — The value of Q from eq. (5) is seen to vary directly with k', a constant which varies directly with the square of the diameter of sand-grains and with the porosity of the material. Furthermore, Q varies inversely as $\log \dfrac{R}{r}$, and the values given in Table No. 53 show that Q changes slowly with changes in the values of either R or r. Thus, other things being equal, a 2-foot well will yield but 15 to 30 per cent more than a 3-inch well.

Eq. (5) may be written in the form

$$Q = k' \frac{(H+h)\,(H-h)}{\log \dfrac{R}{r}}; \qquad (9)$$

or if H and h are nearly equal, as is usually the case, we may write approximately

$$Q = 2k' \frac{H(H-h)}{\log \dfrac{R}{r}}, \qquad (10)$$

from which it is seen that Q is directly proportional to H and also to $H - h$, that is, to the depth of ground-water and to the depression of the water-surface. Thus if in pumping at the rate of 100,000 gallons per day from a well the water-surface is depressed 2 feet, approximately 200,000 gallons may be obtained by lowering the surface 4 feet. If the lowering is too great, then eq. (10) becomes more in error and Q will increase less rapidly than the value of $H - h$. This general relation that Q varies with $H-h$ has been shown to be very nearly correct in many cases by actual tests and is an important principle to keep in mind.

The variation in yield with the depression of water level is graphically shown in Fig. 50a. For a considerable amount of lowering the

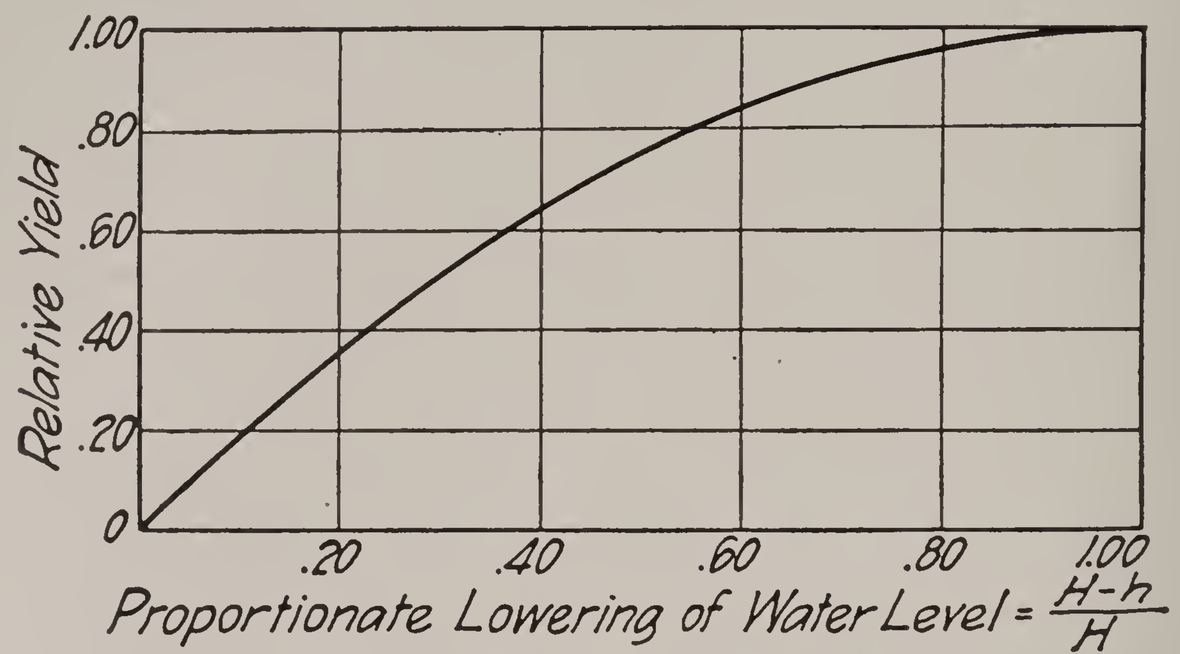

FIG. 50a.—RELATION OF YIELD TO LOWERING OF WATER LEVEL IN WELL.

curve is nearly straight, but as the level approaches the bottom of the stratum (100 per cent lowering) the rate of increase is small. A lowering of 50 per cent will thus give a yield equal to 75 per cent of the yield for a lowering of 100 per cent.

309a. Flow into a Gallery or a line of Wells Closely Spaced.—Fig. 50b is a section through a gallery and represents conditions similar to those shown in Fig. 50. The water is supposed to flow to the gallery from both directions under a head $(H-h)$, equal to the amount the water is lowered below the original level of the ground-water surface, which level is still maintained at a distance R from the gallery. Applying the same method of analysis as in Art. 305, the cross-section A of the ground-water stream at any distance x from the gallery is (considering both sides) equal to $2y$ per unit length of gallery. The slope is, as before, $s=\frac{dy}{dx}$, whence, as in eq. (1), the yield per unit length is given by the equation

$$Q=2ykp\frac{dy}{dx} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

Integrating as before we have $Qx=kpy^2+C$, and we find that $C=-h^2kp$, whence we have

$$y^2 = \frac{Qx}{kp} + h^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

as the equation of the curve CD. Substituting H for y and R for x we have

$$Q = kp\frac{H^2 - h^2}{R} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (13)$$

In this case the flow is seen to vary with H and h in the same manner as in the single well. The variation with R is, however, very different, being now inversely proportional to R.

In the case of galleries and rows of wells the calculations here given are of little value in estimating the total yield unless the area occupied

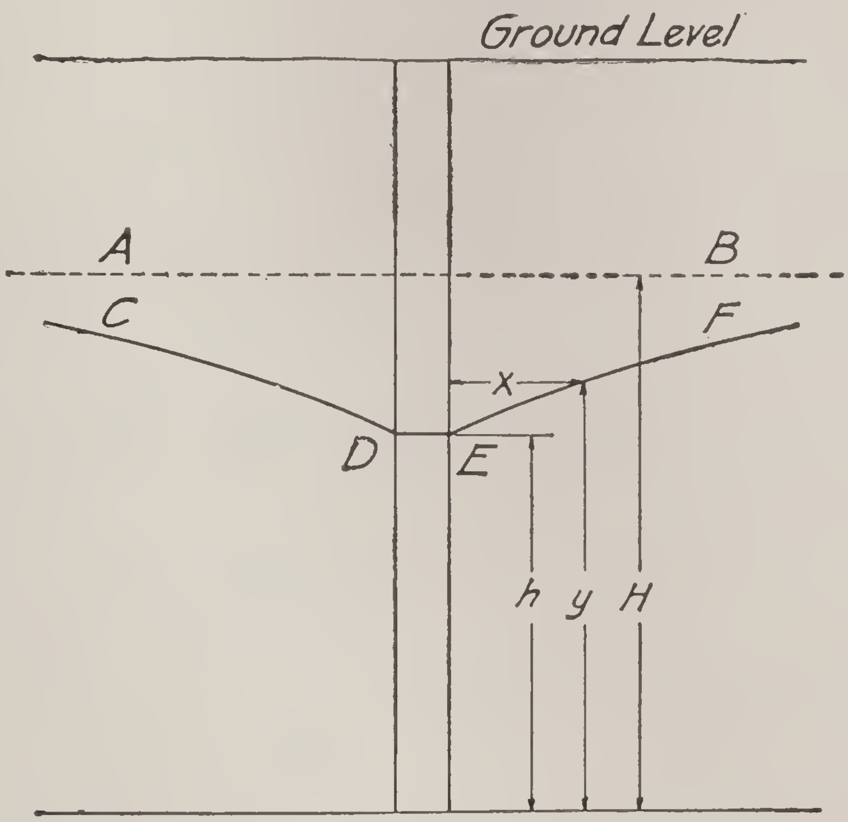

FIG. 50b. — SECTION THROUGH GALLERY.

by such wells is comparatively small so that the water enters from all sides, as in the case of supplies of great capacity as compared to the draught. Galleries, and to a less extent, wells, are usually arranged to intercept as much of the ground-water flow as possible so that most or all will enter from the up-stream side. The yield is then eventually a question of the amount of ground-water flowing through the area in question.

B. *Principles Governing the Flow into Artesian Wells.*

310. Where the water flows under pressure in a porous stratum overlaid by an impervious one, the flow into a well is not accompanied by a change of level in the surface of the water, but the curve of pressures is of a form similar to the water-surface in the case already treated.

In Fig. 51 the thickness of the porous stratum is t, the original

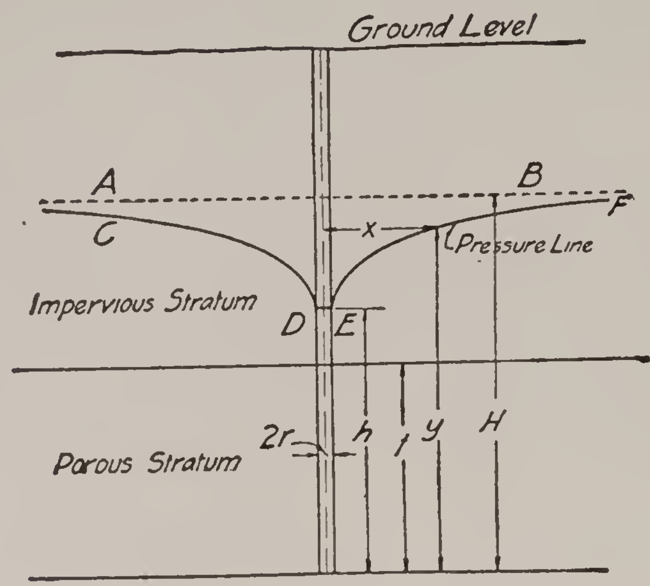

FIG. 51.—SECTION THROUGH ARTESIAN WELL.

pressure-line is AB (below or above the surface), and the pressure-line existing on pumping from the well is CD–EF. The derivation of the equation of the curve of pressures is similar to that in Art. 305, except that in this case the water passes through an area of constant depth t instead of a variable depth y. Making this change, eq. (1) of Art. 305 becomes

$$Q = 2\pi kpxt\frac{dy}{dx}, \qquad \qquad (11)$$

from which we readily get, as before,

$$Q = 2\,k'\,t\frac{H - h}{\log \frac{R}{r}}, \qquad \qquad (12)$$

in which k' = the same constant as before, given in Table No. 52, t = thickness of porous stratum, H = original pressure-head at the bottom of the stratum, h = head at bottom of well when flowing, R = radius of circle of influence, and r = radius of well.

This equation differs from (5) only in having the constant $2t$ in place of $(H + h)$, and hence the laws of flow are very nearly the same as in the previous case; that is, Q varies directly with k', also with $H - h$ (the lowering of the water in the well), and with t, and inversely with $\log R$.

Where an artesian stratum lies near the surface it can be investigated with respect to hydraulic slope, material, and depth as readily as the other class already discussed, and estimates of flow made in the same way. In using Table No. 53, $2t(H - h)$ should be used in place of $H^2 - h^2$.

In the case of deep artesian wells t is sometimes several hundreds of feet, and $H - h$ is also often very large. On the other hand k' is usually small, and likewise the slope, but on the whole the values of Q will usually be much larger than for shallow wells.

An important test showing the variations of Q with $H - h$ was carried out by Prof. Marston on a well 2215 feet deep at the Iowa Agricultural College in which the lowering of the water was very great.* The results were:

Lowering of Water-level.	Yield.
120 feet	10.2 cubic feet per minute
162 "	13.7 " " " "
184 "	15.1 " " " "
238 "	18.8 " " " "

Using the value of 10.2 as a basis, exact proportion would call for yields of 13.8, 15.7, and 20.3 cubic feet respectively.

C. *Considerations of General Application.*

311. Pipe Friction and Other Losses of Head.—The resistances to flow that have not been considered are the friction of entrance into the well-tube or well, the friction in the tube itself, and the velocity-head.

Inadequate area of openings into the well, and the effects of clogging and corrosion, may cause the loss of head at entrance to be a very considerable proportion of the total head. This question is further discussed in connection with the constructive features. The velocity-head is usually too small to be worth considering. It is easily figured in any case from the formula $h = \frac{v^2}{2g}$.

The friction-head in wells up to 50 or 100 feet in depth is usually small, but in deep wells of small diameter it is often a very large item and needs to be carefully considered. If the well is cased for a large portion of its length, the friction can be figured on the basis of the friction in wrought-iron pipes. Where not cased the friction would probably be greater, the amount depending on the roughness of the walls. It will be sufficiently accurate for present purposes to estimate it as 25 per cent greater than that for smooth pipes, and Table No. 55 has been computed on that basis. It gives the frictional head for wells 100 feet deep of various diameters and under various rates of flow. By the use

* *Eng. Record*, 1898, XXXVII. p. 387.

of this, together with the principle that the loss of head due to the resistance in the ground is closely proportional to the flow, we may compute the total head required to cause any given yield from a well, if we know the yield for any particular head; or, knowing the flow from a well, we can compute approximately the yield of wells of other sizes sunk to the same formation.

TABLE NO. 55.

LOSSES OF HEAD IN TUBULAR WELLS DUE TO FRICTION IN WELL-TUBE OR WELL.

Frictional Head. Feet per 100 feet.	Discharge in Gallons per Day.						
	2-inch.	3-inch.	4-inch.	6-inch.	8-inch.	10-inch.	12-inch.
0.5	13,000	39,000	84,000	250,000	550,000	1,000,000	1,600,000
1	19,000	56,000	120,000	350,000	830,000	1,500,000	2,400,000
2	28,000	84,000	180,000	550,000	1,200,000	2,200,000	3,600,000
3	35,000	110,000	230,000	700,000	1,500,000	2,700,000	4,600,000
4	42,000	120,000	270,000	830,000	1,800,000	3,200,000	5,300,000
5	47,000	140,000	310,000	940,000	2,000,000	3 700,000	6,200,000
6	53,000	160,000	350,000	1,000,000	2,300,000	4.200,000	6,900,000
8	62,000	190,000	400,000	1,200,000	2,600,000	4.700,000	7,900,000
10	69,000	210,000	470,000	1,400,000	3,000,000	5,500,000	9,000,000
15	90,000	270,000	590,000	1,700,000	3,800,000	7,000,000	12,000,000
20	100,000	310,000	690,000	2,000,000	4,600,000	8,300,000	
30	130,000	400,000	860,000	2,600,000	5.600,000	10,000,000	
40	150,000	460,000	1,000,000	3,000,000	7,000,000		

312. *Illustrative Calculations.*—1. Suppose a well 6 inches in diameter and 500 feet deep yields 500,000 gallons per day, with a total head of 15 feet. Let it be required to find the head necessary for a discharge of 1,000,000 gallons daily. In the first case the loss of head by friction is, from the table, about 9 feet, and, neglecting other losses, the head consumed in the strata is therefore 6 feet. To discharge 1,000,000 gallons requires about 12 feet head in the ground and 30 feet in pipe-friction = 42 feet total required head, or an added head of 27 feet over that required for a yield of 500,000 gallons.

2. Suppose a 6-inch well 1000 feet deep yields 1,000,000 gallons per day under a total head of 100 feet. What will be the yield of a 4-inch well under the same head?

From Table No. 55 the frictional head in the 6-inch well is about 60 feet, and, neglecting other losses of head, the head lost in ground friction is 100 — 60 = 40 feet. For other volumes it will be assumed that the head lost in ground-friction is proportional to the volume. The problem now is to determine an amount Q for the 4-inch well such that the total loss, pipe friction and ground friction, shall be 100 feet. It is readily solved by trial. Thus for various values of Q the losses of head are:

Q.	Pipe Friction.	Ground Friction.	Total.
470,000	100	19	119
400,000	80	16	96

Hence for a total head of 100 feet Q will be about 420,000 gallons per day.

3. As illustrating the effect of size of well where the pressures and depths are great, values of Q have been computed for various sizes of wells in accord-

ance with the data of example 2, so that the total loss of head is about 100 feet in each case. They are given in the adjoining table together with the losses of head.

Diameter of Well.	Q. Galls. per day.	Pipe Friction. Feet.	Ground Friction. Feet
2-inch	68,000	97	3
3 "	200,000	90	8
4½"	420,000	85	16
6 "	1,000,000	60	40
8 "	1,600,000	35	64
10 "	2,100,000	18	84
12 "	2,300,000	9	92

From this it is seen that for wells of small diameter and with high pressures the yield is principally dependent upon the pipe friction, but that with large diameters the yield depends rather upon the ground friction and is little affected by the diameter.

313. *Examples of Wells Flowing under High Heads*—In the Dakota artesian basin the pressures run up to 300 feet and over, thus giving rise to high velocities and large losses of head from pipe friction. The great differences in yields from different-sized wells there noted are largely due to this fact. Below are given data of several typical wells taken from the United States Geological Survey Report, 1895–6, Part II. A column of "computed yields" has been added, the computations having all been made on the basis of the flow of the 8-inch wells and on the assumption that all the water flows the entire length of the well. Only roughly approximate results could of course be expected, as the wells are distributed over a large area and the water-bearing stratum is more or less irregular; and, besides, the observed yield is doubtless in many cases from very rough measurements. The reported yields for some of the smaller wells would be impossible under the given head if all the water entered at or near the bottom.

TABLE NO. 56.

DATA OF ARTESIAN WELLS IN THE DAKOTA BASIN.

Diameter, Inches.	Depth, Feet.	Static Head, Feet.	Yield.	
			Observed Gals. per Min.	Computed, Gals. per Min.
1	480	142	30	16
2	715	303	200	100
2½	880	262	280	180
3	689	300	425	320
3	1315	287	350	210
4½	840	345	1000	900
4½	902	352	670	750
6	712	276	1500	1700
6	897	142	1200	1000
6	1350	138	500	850
8	530	198	3292	2600
8	1000	345	2000	3200
8–10	640	253	4350	4000

314. **Effect of Depth of Well.**—It has been assumed in the preceding discussion that the well penetrated to the impervious stratum. If it

reaches short of this, there will evidently be increased resistance near the well for like quantities of water, or for the same head the flow will be decreased. This added resistance due to decreased cross-section occurs only in the immediate vicinity of the well, and if the total loss of head or total depression is great, and if the well extends half or two-thirds through the porous stratum, the added resistance will be but a small proportion of the total and the consequent effect on Q will not be great. It often happens that the water-bearing formation is made up of layers of different degrees of porosity, so that the resistance to flow from one stratum to another would be very great. In this case the yield would be very largely influenced by the depth of the well.

315. **Mutual Interference of a Number of Wells.**—If two or more wells penetrating to the same stratum are placed near together and simultaneously operated, the total yield will be relatively much less than the yield of a single well pumped to the same level. This mutual interference of wells depends in amount upon the size and spacing of the wells, upon the radius of the circle of influence of the wells when operated singly, and upon the depth to which the water is lowered by pumping. Professor Slichter has investigated this subject theoretically,* and some examples given by him of the application of his formulas will be instructive.

Assuming the wells in question to be 6 inches in diameter, that the water is lowered 10 feet by pumping, and that $R = 600$ feet, the mutual interference of a group of two wells, a group of three wells, and of a large number of wells placed in one row are as given in the following table. The amount of the interference is expressed as the percentage of reduction in yield per well below that of a single well uninfluenced by others. According to the figures given in the table

TABLE NO. 57.

MUTUAL INTERFERENCE OF GROUPS OF SIX-INCH WELLS.

(Water lowered 10 feet by pumping. $R = 600$ feet.)

Two Wells.		Three Wells.		Large Number of Wells in a Row.	
Distance Apart of Wells. Feet.	Interference. Per cent.	Distance Apart of Wells. Feet.	Interference. Per cent.	Distance Apart of Wells. Feet.	Interference. Per cent.
5	38	5	55	100	66
10	35	10	51	200	45
100	20	100	31	400	24
200	16	200	22	600	14
400	11	400	12	1000	6
1000	6	1000	8		

* Report U. S. Geolog. Survey, 1897–98, p. 371.

two wells 200 feet apart will yield $84 \times 2 = 168$ per cent as much as a single well. If a third well be placed between these two, the yield will be $69 \times 3 = 207$ per cent as much as the single well. If a large number of wells are placed 100 feet apart, the yield of each is but 34 per cent as much as it would be if operated alone.

316. Determination of Yield by Tests.—Where it can be done, the best way to determine Q is by actual tests conducted for a sufficient length of time to bring about a condition of equilibrium in the flow, but unless this condition is approximately fulfilled such tests are apt to be very deceptive. With a flat slope to the ground-water a test may be carried on for weeks and even months, and the circle of influence will still continue to widen, resulting in a gradually decreasing yield. It may thus require years of operation to bring the conditions to a final state of equilibrium except as affected by variations in the percolation. The steeper the slope the quicker the conditions become constant; and to aid in judging of results obtained by pumping tests, the ground-water slope should be determined when possible.

In conducting tests on shallow wells it is desirable to observe the variations in ground-water level at different distances from the well. This will aid in determining when equilibrium has been reached, and will also enable R to be estimated and will give information as to the proper spacing of a series of wells.

The value of Q being found for the test well, the effect of variation in the size of the well and in the lowering of the water-level can be determined from the theoretical considerations already discussed. As a valuable check on results of tests, the yield should, where possible, be estimated by the method explained in the preceding articles.

As an example of long-continued lowering, the operation of the large well in Prospect Park, Brooklyn, may be cited. The water-level varied as follows:

Date.	Volume Pumped per Day. Gallons.	Elevation of Water-level above Tide. Feet.
1869	0	14.55
1870	300,000	14.15
1871	272,000	13.03
1872	437,000	10.56
1873	288,000	11.29
1874	333,000	10.70
1875	294,000	9.83
1876	235,000	9.83
1877	252,000	9.21
1878	249,000	8.80
1879	260,000	8.85

The actual capacity of the well appears thus to be about 250,000 gallons per day.

317. Wells Sunk into Strata in which the Flow takes Place through Fissures.—The preceding analysis has been based upon the assumption that the water flows through the interstices of the porous material. In some rock formations, however, much of the flow undoubtedly takes place through fissures. This is apt to be the case with limestone strata, the passageways in this material sometimes assuming large dimensions, due to the solvent action of the water.

The effect of these fissures is greatly to increase the capacity of the material and at the same time to modify the law of flow. The resistance to flow through large fissures will vary approximately as the square of the velocity, instead of as the first power; and as one result the yield of a well supplied largely in this way will not increase at the same rate as the lowering of the water in the well, but much more slowly. As a matter of fact most wells in sandstones do follow approximately the law of the proportionality of yield and head, but it has been observed in the case of some limestone wells that there is a large departure from it in the direction above indicated.

CONSTRUCTION OF WELLS.

318. Forms of Construction.—The various forms and sizes of wells used to collect ground-water may be divided into the following classes: (1) Large open wells; (2) shallow tubular wells; and (3) deep and artesian wells.

No sharp line of division can be said to exist between shallow wells and deep wells, and in many matters that which applies to one class applies equally well to the other. It will, however, be convenient to divide them into the above classes, the methods of construction, of investigation, and of operation being in many respects different in wells of 25 feet to 100 feet deep than in wells of greater depth. One hundred feet may be roughly taken as the limit of shallow wells.

319. Location of Wells.—To procure water economically in the large quantities required for public supplies, it was made evident in the discussion on the flow of ground-water that there must be present a water-bearing formation of considerable extent and porosity. The location of such a deposit is here supposed to have been determined upon through borings and tests, and as a general requirement the works for collection should be so placed as to intercept for a given expense as large a quantity of water as possible.

It has been shown that the more the water in a well is lowered the greater is the yield. A favorable location for a well-plant will therefore usually be at a point where the ground-water is reached with the least lift of the pumps. This will ordinarily be on low ground and often in the vicinity of surface streams. If wells in such a situation are pumped too low, they will draw water from the stream as well as from the ground-water, a result sometimes undesirable. In some cases it may be allowable to obtain filtered surface-water in this way, but this use of wells will be discussed subsequently. For the present it will be assumed that all the flow is strictly ground-water.

320. Relative Advantages of Large and Small Wells.—The yield of any form of well is a question rather of the flow of the ground-water and of the area made tributary by the depression of the water-level in a well, than a question of the size or form of construction. The discussion in Art. 309 shows that the effect of size alone is very small, and that therefore we need not expect an increase in the yield of large wells commensurate with increase in size. The increase in flow is, however, something; and in the case where the circle of influence is small, or where the water is present in large quantities, the increase may be very considerable.

The large well possesses a great advantage over the small well in its storage capacity. If the pumping is carried on at a variable rate, it thus acts to increase greatly the real capacity of the large well over that of a series of small tube-wells. Furthermore, in the operation of the pumps there are many advantages in being able to get the entire supply from a single well, or from two or three large wells close together, chief among which is the avoidance of long suction-pipes. The large well is also of great advantage where it becomes necessary to lower the pumps, as it permits the use of a more economical form of pumping machinery.

Trouble is often experienced in the small wells through clogging and the entrance of fine sand. This is largely avoided in the large well, as the entrance velocity of the water is very small. Opportunity is also given for the settling of fine material.

The chief disadvantage of the large well is in its great cost compared to the tube-well for like yields. This disadvantage increases rapidly as the depth increases, and where it may be economy to construct a large well to a certain depth to serve as a pump-pit it will usually be cheaper to develop the yield by sinking tube-wells from the bottom, or by driving galleries therefrom, than by further sinking.

Except where used as pump-pits it will seldom be economical to adopt the large well for depths exceeding 30 or 40 feet.

It may be said, therefore, that large wells are suited for places where the water can be reached at moderate depths, where the excavation is not difficult, where a single large well will furnish the desired amount, and where the pumps are to be operated but a few hours of the day. The tubular well is particularly suited for developing a supply from a wide area and from strata of irregular character, and for penetrating deep strata.

Large Open Wells.

321. **Size and Depth of Wells.**—Large wells for water-works are constructed of diameters of 10 feet or less to as great as 100 feet, 30 to 50 feet being the most common size. The best size must be determined from a consideration of the various factors mentioned in Art. 320; but as the cost of a well increases with increase in diameter more rapidly than does the yield, a very large diameter should be adopted only after most careful consideration.

322. The minimum depth of a well is determined by the depth necessary to reach and penetrate for a short distance the water-bearing stratum, allowing a margin for dry seasons. Beyond this it should be extended to allow for storage, and to permit of such a lowering of the water-level as is estimated will be the most economical with the form of pump employed, or such as will be necessary to secure the desired amount of water.

323. **Construction.**—In the first place, it is to be noted that large quantities of water will be met with, and if the excavation is to be made in the open, adequate means of handling it must be provided. As the water-level must be kept at the lowest level of the excavation, the maximum pumpage will be considerably more than the future capacity of the well. For moderate depths the excavation can be carried on with no other aid than sheet-piling. If the well is of large diameter, an annular trench is usually first excavated and the curb or lining built therein, after which the interior core is removed. This method enables the sheet-piling to be readily braced. A method adapted to smaller wells is to drive the sheet-piling outside of a series of wooden frames or ribs, and to excavate the entire well at once. The ribs are built in place as the excavation proceeds. This method is illustrated in Fig. 53, page 296.

For wells of considerable depth sunk in soft material, the curb may be started on a shoe of iron or wood, and the excavation and the construction of the curb carried on simultaneously, the curb sinking from its own weight. The material may be either excavated in the ordinary way, or by the use of compressed air, or dredged out without attempting to keep out the water, the method used depending upon depth of well, quantity of water, and character of the material. Where the friction becomes too great to sink the first curb the desired distance, a second curb with shoe may be sunk inside the former. In Fig. 52 are illustrated two forms of shoes used in sinking wells. These are both constructed mainly of wood. To strengthen such curbs iron rods should

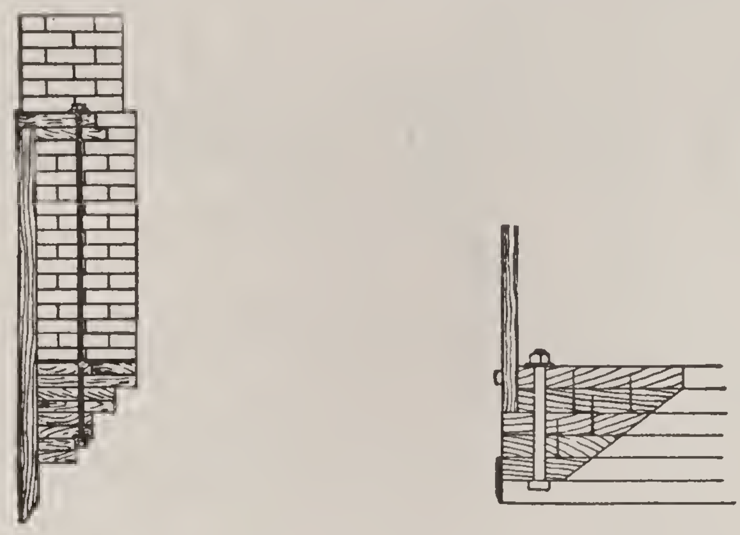

FIG. 52.—SHOES FOR SINKING WELL-CURBS.

extend from the shoe well up into the masonry. For large wells, pump-pits, etc., heavy iron shoes are often employed, and occasionally a pneumatic caisson is found necessary.

The lining or curb usually consists of a circular wall constructed of concrete or masonry of a thickness varying with diameter and depth of the well, and the material employed. If concrete is used, slightly reinforced, a thickness of 12 to 18 inches will usually be ample. The upper portion of the lining should be impervious in order to prevent the entrance of imperfectly filtered surface water. If the well is to be fed from strata that are partly or wholly cut off by the curbing, entrance for the water should be provided for by laying the wall dry or by means of special openings. The entrance of fine sand through such openings can be prevented by a back filling of broken stone and gravel suitably graded in fineness. A lining of cast-iron segments bolted together has been frequently employed in sinking deep wells, especially in Europe, where wells of 5 to 10 feet in diameter and 75 to 100 feet deep are quite common.

All wells should be covered to exclude the light and to prevent pollution of the water. The cover is usually made of wood, which for large wells may be conveniently made of a conical form and supported by a light wooden truss, or by rafters resting against the wall.

324. Yield.—The actual yield of large wells which are considered successful varies from 100,000 to 4 or 5 million gallons per day, the higher values being very exceptional. A computation of the carrying capacity of ordinary porous material by the methods explained in a preceding chapter will show that for a single well to furnish one million gallons per day requires a very extensive tributary area and a considerable lowering of the water-level in the well.

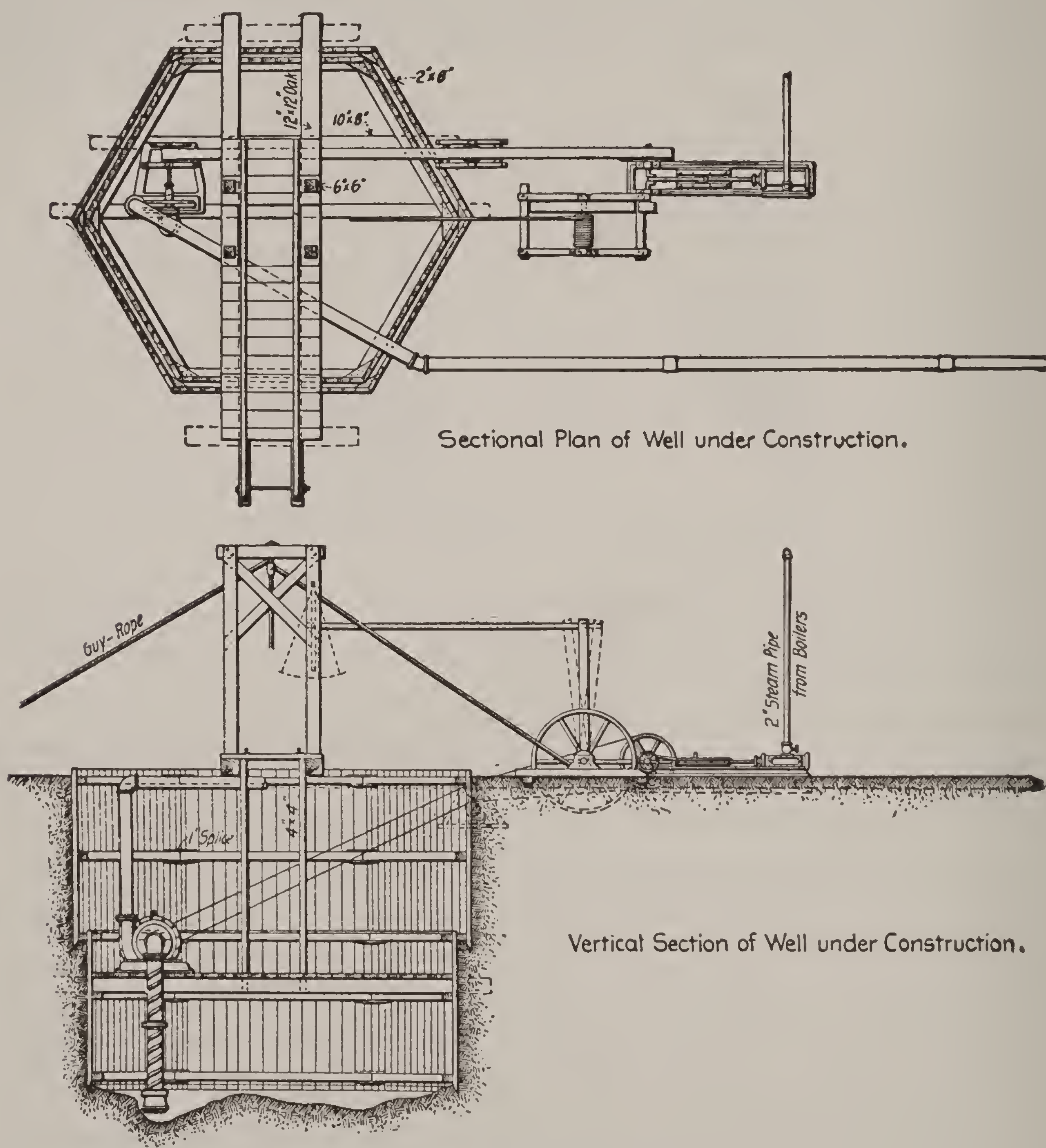

FIG. 53.—LARGE WELL AT ADDISON, N. Y.
(From *Engineering News*, vol. XXXIII.)

325. Examples.—At Peoria, Ill., a well 36 feet in diameter was sunk on a wooden shoe with cast-iron cutting edge to a depth of 44 feet through clay to a water-bearing gravel. During sinking, from 11 to 13 million gallons per day were pumped. The curb is a 30-inch brick wall.*

* *Eng. News*, 1892, XXVIII. p. 26. See also Reference No. 16 under "Driven Wells" at end of chapter.

At Webster, Mass., a well 25 feet in diameter was sunk in gravel to a depth of 30 feet, sheet-piling being driven on the outside of circular ribs made of 3-inch plank bolted together. Two sets of piling were used. The curb was built as a dry rubble wall 5 feet thick at base and 2 feet thick at top, with a 12-inch brick lining laid in cement. Two 6-inch centrifugal pumps were used during construction, the maximum pumpage being 1 million gallons per day. The cost complete was $13,190.*

At Addison, N. Y., an auxiliary supply was obtained from a well 12.5 feet in diameter and 23 feet deep. Fig. 53 illustrates clearly the method there used in sinking. On account of very soft material the well was stopped short of the necessary depth and the water-bearing stratum was reached by twenty-three $1\frac{1}{2}$-inch tubes driven from 7 to 20 feet below the bottom of the well. The yield was 165,000 gallons per day. The curb is a 20-inch wall in cement, and the cover is of flagging laid on I beams. The cost was, for the excavation $2.18 per cubic yard, and for the masonry $5.82. The total cost was $851.

Shallow Tubular Wells.

326. Shallow tubular wells or wells of small diameter, also called driven wells,† are sunk in various ways, depending upon the size and depth of well and nature of the material encountered. As wells for public supplies would rarely be sunk in rock except to a considerable depth, the methods of construction here considered will refer only to shallow wells sunk in soft material. To furnish large quantities of water it usually requires a number of wells, and in addition to the question of sinking, questions of arrangement, spacing, connecting, and operation are important. While the strata penetrated by shallow wells are often artesian in character, yet this fact is of little consequence in this case, as the pressures would be small and the method of construction and operation the same as for wells tapping the ordinary ground-water.

327. Methods of Sinking.—As regards methods of sinking there are two principal kinds of wells: the closed-end well or driven well proper, and the open-end well.

328. *The Closed-end or Driven Well.*—In this form the well-tube consists of a wrought-iron tube from 1 to 4 inches in diameter, closed and pointed at one end, and perforated for some distance therefrom. The tube thus prepared is driven into the ground by a wooden maul or block until it penetrates the water-bearing stratum. The upper end is then connected to a pump and the well is complete. Where the

* *Jour. New Eng. W. W. Assn.*, 1895, IX. p. 240.

† The term "driven well" is somewhat loosely applied to small tubular wells of all kinds where the tube is sunk largely by driving. It is also used in a more restricted sense to denote a closed-end well sunk wholly by driving and without the removal of any material.

material penetrated is sand the perforated portion is covered with wire gauze of a fineness depending upon the fineness of the sand. To prevent injuring the gauze and clogging the perforations, the pointed end is usually made larger than the tube, or the gauze may be covered by a perforated jacket.

Fig. 54 shows a common form of well-point and a method of driving wells by means of a weight operated by two men. The tube may

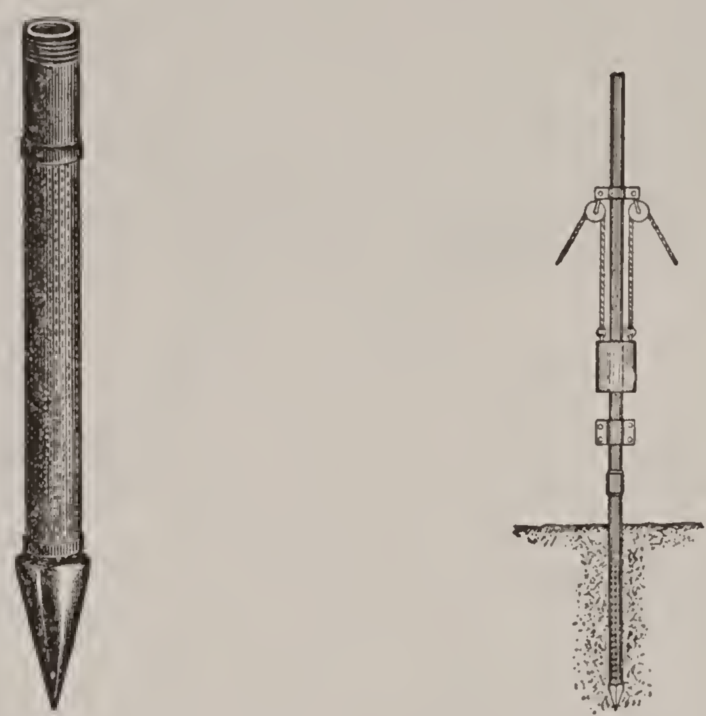

FIG. 54.—WELL-POINT AND DRIVING-RIG.

also be driven by a wooden block operated by a pile-driver or other convenient means.

The well above described is adapted for use in soft ground or sand up to a depth of about 75 feet, and in places where the water is thinly distributed. On account of the ease with which it can be driven, pulled up, and redriven, it is useful in prospecting at shallow depths, and in fact groups of wells are often finally located by driving and testing until a good result is obtained.

329. *Open-end Wells.*—For use in hard ground and for the larger sizes the open-end tube is better adapted. This is sunk by removing the material from the interior, and at the same time driving the tube as in the other case. A very common method of sinking is by means of the water-jet. In this process a strong stream of water is forced through a small pipe inserted in the well-tube, the water escaping in one or more jets near the end of the pipe. At the same time the pipe, which is provided with a chisel edge, is churned up and down to loosen the material, which is then carried to the surface by the water in the annular space between the pipe and tube. If the material is hard or the well deep, a steel cutting-edge may be screwed on to the end of the well-tube.

Fig. 55 shows an outfit used by Mr. L. L. Tribus, Mem. Am. Soc.

C. E., in jetting down 6-inch wells at Pensacola, Fla to a depth of 90 to 130 feet.*

The driving was done by a hammer weighing 1000 pounds and operated by a pile-driver. The jet-pipe was worked under a water-pressure of 75 pounds and churned up and down by a rope led over the head of the pile-driver and wound on another spool of the pile-driver engine. One engineman, one driller, and two laborers operated the machine, and with this force 6-inch pipes were driven 140 feet in ten hours. Similar methods have been used in several recent investigations of ground-water supplies and in the construction of permanent plants. In the extensive investigation made on Long Island by the New York Water Supply Commission a 200-pound hammer was used, operated by ropes running through blocks attached to a pipe derrick. The average cost per foot of two-inch test wells was about $1.00.†

A process similar to the water-jet has been used in which steam is employed instead of water. Another method is to remove the material by means of a sand-bucket.‡

Extra-strong pipe, called drive-pipe, is ordinarily used for well

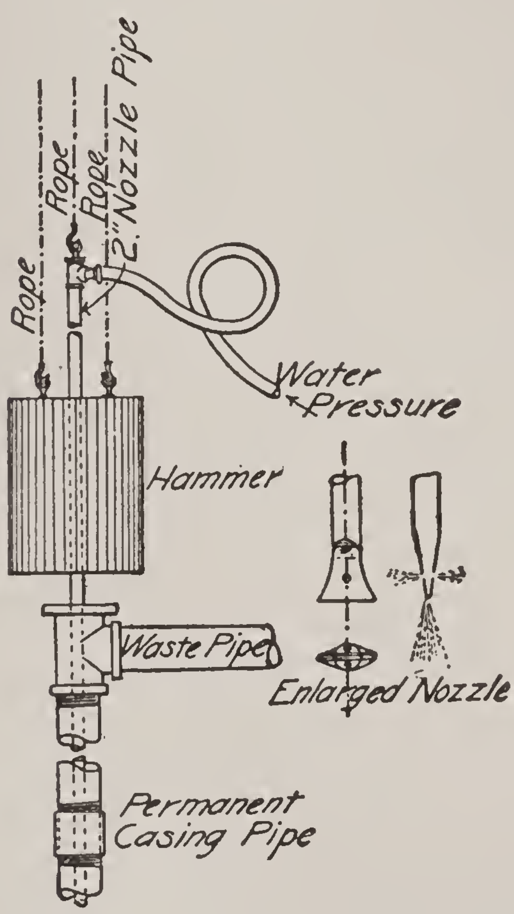

FIG. 55.—JETTING APPARATUS.

FIG. 56.—COOK WELL-STRAINER.

tubes, care being taken that the joints are screwed up so that the ends of the pipe are in contact.

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* *Eng. Record*, 1898, XXXVII. p. 428.

† Report of Commission, 1903, p. 629. Various driving rigs are illustrated.

‡ See also stove-pipe method in Art. 345.

330. Strainers. — With the open-end well the lower portion may be merely perforated with small holes in case the material is coarse or gravelly, or if sand is met with the holes may be covered with brass gauze. Instead, however, of using a gauze it is common with this style of well to sink a solid tube, insert a special strainer of suitable length, and then withdraw the tube nearly to the top of the strainer. If necessary a tight joint can then be made between the tube and strainer by means of a short piece of tubing or lead packer cut to a bevel.

Fig. 56 illustrates a commonly used form of strainer known as the Cook strainer. It is made of brass tubing and provided with very narrow, slotted holes, which are much wider on the interior than on the exterior, an arrangement intended to prevent clogging. Fig. 56a illustrates the Johnson strainer, a very ingenious and satisfactory form. It is made up of a strip of brass of special section, spirally wound upon a temporary core. Successive turns of the strip interlock, thus forming a continuous cylinder of any desired length and diameter. Between successive strips a narrow slit of any desired width is formed on the outside, through which the water may pass into an interior annular space and thence through large circular holes into the well. The width of outer slot is made from .004 to .02 inch.*

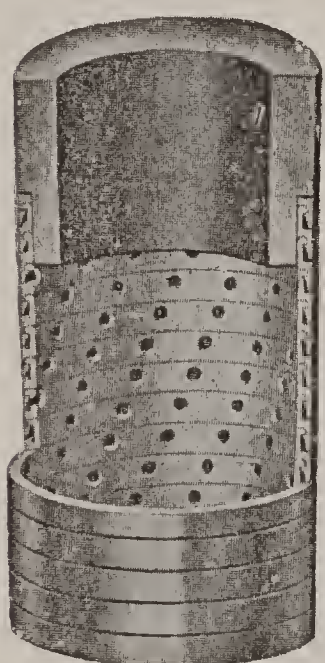

FIG. 56a.—THE JOHNSON STRAINER. (From *Engineering News*, VOL. LV.)

Strainers of the type just described are intended to prevent the inflow of sand and are especially useful where the water bearing material contains little or no coarse material. The resistance to entrance immediately adjacent to the slots will be relatively great as the water is forced to pass through a very small sectional area. This results in a considerable loss of head unless the size and length of strainers are carefully proportioned to the requirements. Where coarse material is present with the fine, a coarse strainer or perforated pipe may be used to advantage. This will permit the inflow of some fine material, but as this escapes the coarser particles will form a natural strainer outside the pipe of much greater effective cross-section. Before the wells are placed into service the fine sand should be removed by rapid pumping, or by the sand buckets, or it may be loosened and washed out by a jetting bit. This general result may also be accomplished in the case of

* *Eng. News*, 1906, LV. p. 260.

fine sand by inserting a coarse strainer of smaller diameter than that of the well, filling between strainer and well tube with coarse sand or gravel and then drawing up the tube. A very large gravel strainer has been successfully used by Mr. D. H. Maury at Peoria.* This general scheme is used in the form of well illustrated in Fig. 57. In this the strainer is made of perforated vitrified pipe.† This type of well has been used successfully in the Brooklyn plant, the strainers being 4 or 8 inches in diameter and the outer casings 12 or 18 inches respectively. In very fine material two or three layers of sand of graded size can be used so as to effectually prevent clogging and filling of the well. A removable basket-strainer has also been used with success.

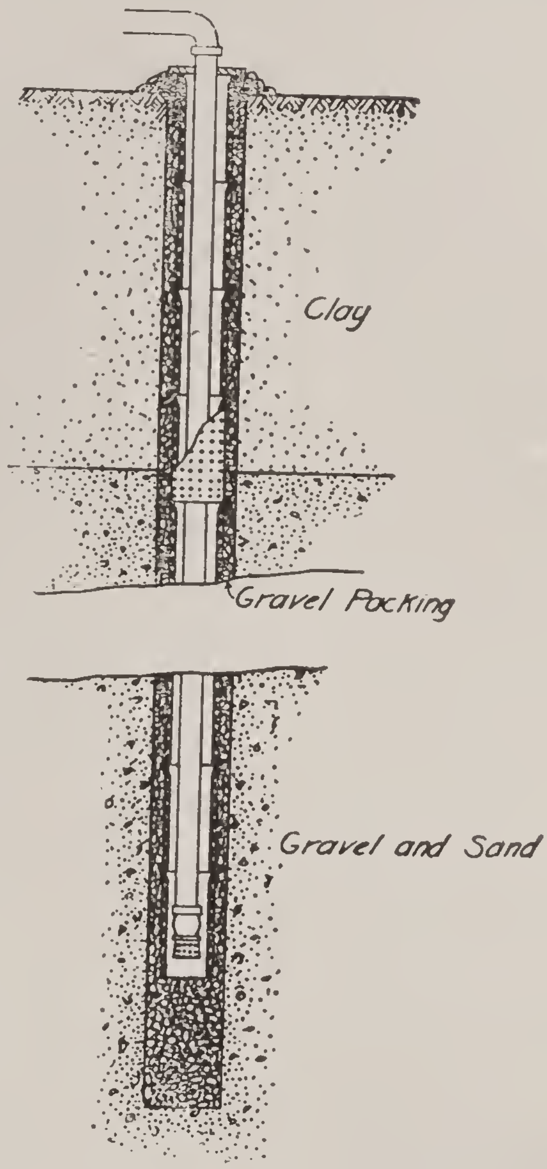

Fig. 57.—The Dollard Well.

In some waters strainers have given trouble by corroding, thus necessitating removal and cleaning. Small perforations in ordinary pipe are also apt to give trouble by rusting. This has been avoided in some cases by bushing the holes with brass and in other waters galvanized pipe is more successful, the holes being drilled and reamed before galvanizing.

The length of the strainer or perforated portion, in order to reduce the friction in the ground to a minimum, should be equal to the thickness of the porous stratum passed through, but the resistance to flow will be but slightly increased if it is made materially shorter, even half or one-third the thickness. The total area of the perforations should be sufficiently large to keep the velocity of entrance down to 2 or 3 inches per second, both to keep the friction loss low and to prevent the entrance of sand. Sometimes open-ended tubes without perforations are employed, and in the case of thin strata the yield may be nearly as great as with perforated tubes. With thick strata, however, the resistance to entrance would be greatly increased if all the water is forced to enter at the bottom.

* *Eng. News*, 1904, LII. p. 138.
† *Ibid.* 1896, XXV. p. 114.

332. General Method of Operating a Well System.—Small tubular wells are usually arranged in one or two rows alongside a suction-pipe and connected thereto by short branches. The smaller sizes are connected directly to the branch, the well-tube acting also as a suction-pipe, but with the larger sizes a separate suction-pipe is ordinarily employed. In the former case, to avoid the entrance of air, it is necessary that the perforated portion of the pipe be always under water, and to insure this being the case it should be kept below the limit of suction. With the latter arrangement there are no such limitations to the position of the perforated well-casing.

Since the amount of water that can be pumped from a given system of wells increases with the amount that the water-level is lowered (up to the point where the water-level is reduced to the bottom of the water-bearing stratum), and as this is limited by the limit of suction, it is always desirable to make the connection between wells and pumps at least as low as the pumps. In many cases, to increase the yield the suction-pipe is laid at a considerable depth in the ground, and the pumps are lowered accordingly. This of course increases the expense of construction, and as the height to which the water is pumped is increased, it also adds to the cost of operation. The most economical design can be arrived at only by a due consideration of all these elements.

The effect of an extreme amount of lowering is apt to be a somewhat serious matter in causing the drying up of wells and springs and even of streams, and recent court decisions indicate that a city cannot draw too greatly upon the ground-water without rendering itself liable.

333. Arrangement and Spacing of Wells.— The most favorable arrangement for a system of small wells is in a line at right angles to the direction of flow of the ground-water, as in this way the largest possible area will be drawn upon. By placing the wells across the line of flow or along a ground-water contour, the advantage of equal heads in the several wells is also secured. Where but a small area or width needs to be drawn upon, the arrangement is not so material, as the water will flow towards the wells from all directions; but with a long line of wells and a large draft it becomes a question of much importance. The amount of water which can be obtained from a system of wells depends upon the average amount which the water-level can be lowered along the line of wells. The ground-water surface through a line of wells when in operation will have some such form as shown in Fig. 58, *A*, *B*, *C*, and *D* being the wells, *L M* the original level of ground-water, and *N O P Q R*, etc., the new surface. The yield will be some function of the average lowering h. If intermediate wells, E, F, and G, are inserted and pumped to the same level as the others, the surface will be $N\,O'\,P$, etc., the average lowering now being h',

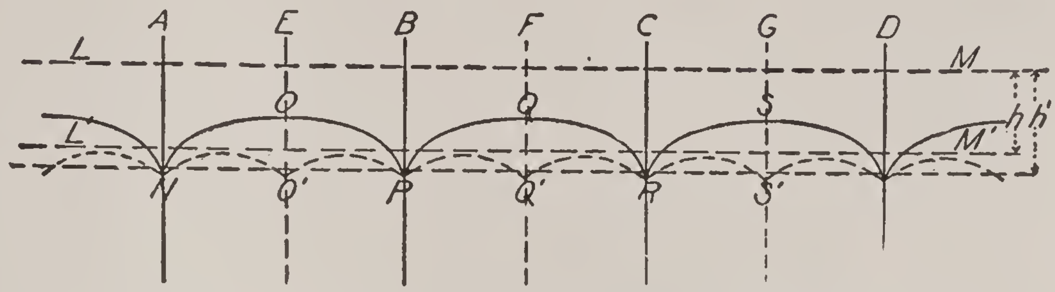

FIG. 58.—SECTION THROUGH LINE OF WELLS.

and more water will be extracted with the same amount of suction; but if the circles of influence of the first wells already intersect, the additional amount drawn from the intermediate wells will be much less than the yield from the others.

The maximum amount of water obtainable from a given number of wells would be when they are spaced far enough apart so that their circles of influence will not overlap, but on account of cost of piping, and loss of head by friction, this would not be the most economical spacing. If wells are deep and therefore expensive, they should be spaced to interfere comparatively little; if shallow, then closer. As indicating what the mutual interference of wells may be, the examples given on page 290 are of some value. In practice the extent of this interference can best be judged by pumping tests of trial wells or of those first sunk, the wells being operated at different rates and in various combinations. The information thus obtained, together with a knowledge of items of cost, will enable the best spacing of subsequent wells to be determined.

While it is impossible to give figures which would be of general application, it may be stated that from 25 to 100 feet is about the range for economical spacing of shallow wells. With very deep or artesian wells the spacing becomes still greater. Spacing less than 25 feet has quite often been used, but with doubtful economy.

The principles here discussed relating to arrangement and spacing have frequently been overlooked in the location of small wells, and many instances exist where they have been placed in such a way that a small part of the actual number would furnish as much as the entire group. In one case seventeen 3-inch and 6-inch wells were placed in an area within a circle 125 feet in diameter, and a pump test showed that seven would furnish as much water as the entire seventeen. In another case six 4-inch wells were placed in a row 10 feet apart, and a test showed one well to furnish half as much as the six. In still another case twenty-four 2-inch wells were placed in an area 20 feet by 95 feet. Wells so placed that they do not extend the general circle of influence do not add to the flow.

334. **Size of Well.**—It has been shown (Art. 309) that the effect upon the yield of a considerable change in size of well is very small provided that the head lost by friction in the well-tube is small. A well should therefore be large enough to keep the friction loss within low limits, but beyond this little advantage is gained by further increase. The proper size thus depends upon the quantity obtainable per well, and this in turn upon the spacing. The size and spacing should therefore be considered together. With the shallow wells under consideration the slight additional cost of the larger well will make it economical to keep the friction-head down to a few inches or at most 1 or 2 feet, corresponding to velocities not exceeding 2 or 3 feet per second. A low friction-head, besides making the pumping more economical, also increases the suction limit of the pump and hence the capacity of a given number of wells. For estimating frictional losses for different sizes of wells, use may be made of Table No. 55, page 288.

In many cases the best size and spacing are largely influenced by the means available for sinking the wells.

335. As illustrating the relation between size and spacing, reference may be made to the various plants of the Brooklyn Water-works.* Some of the driven-well plants of these works yield 20,000 to 40,000 gallons per day per well from 2-inch wells about 50 feet deep and spaced about 13 feet apart in two rows. If a much wider spacing were adopted and a larger quantity per well expected, it would manifestly be necessary to use at least a 3-inch well, or else a great loss of head would result. Another plant yields about 200,000 gallons per well from 6-inch wells ($4\frac{1}{2}$-inch suction-pipes) spaced 40 feet apart, a yield which would be impossible from 2-inch wells. To procure the same yield from 2-inch wells with the same friction-head would require about seven times as many wells, but with a closer spacing a less lowering of water in the well would be required, so that for the same *total* loss of head perhaps five times as many wells spaced 8 feet apart would be equivalent. This would probably be a more expensive arrangement than that with the larger wells. A still more economical arrangement than the one used might be to space the wells say 100 feet apart, using 6-inch suctions and 8-inch wells.

336. **Details of Connections.**—Each well should be connected to the suction-main by means of a short branch in which should be placed a gate-valve, so that any well can be shut off at any time. Where the well-tube itself is connected to the main it has been found convenient to insert a short piece of lead pipe to allow of easy adjustment, as the

* Brooklyn Water-supply, Department of City Works, 1896.

well is likely to be slightly out of plumb. Connection should be made at the well by means of a curved T, from the vertical branch of which the well-tube should extend to the surface and there be capped. This arrangement makes the well readily accessible for inspection and cleaning purposes. To reduce friction as much as possible, the connection of branch to main may be made with a Y branch instead of a T. The main suction-pipe is usually made of flanged pipe, as this enables air-tight joints to be more readily made, although ordinary bell-and-spigot pipe with lead joints has been successfully used.

The greatest care must be taken in every part to make the work air-tight, and to secure this it should be thoroughly tested in sections by means of compressed air. All valves should be carefully tested for air-tightness, and all screw connections thoroughly fitted. Mr. Freeman C. Coffin in certain specifications* prescribes that the suction-main, and branches up to the valves, shall be tested by air at a pressure of 50 pounds per square inch, which pressure once secured shall be maintained without pumping. The valves are tested under 100 pounds pressure.

If settling of main is feared, special foundations must be provided. Main and branches are usually laid underground, both for protection and to increase the range of pump-suction. The pipe system should be made of sufficient size, and all connections so designed as to reduce the friction to the lowest limits consistent with economy. This will require the use of velocities not exceeding 1 or 2 feet per second. The suction-main should be laid on a slightly ascending grade toward the pump to prevent lodgment of air at any point. All perforations in suction-pipe, or in well-tube used as such, should be below the limit of suction.

337. *Air-separator.*—In spite of the most careful construction, air will usually accumulate to some extent, and to eliminate it many plants are provided with air-separators placed on the suction-main near the pump. The simplest form consists of a large drum of wrought iron through which the water passes at a slow velocity and in a thin sheet, either over broad horizontal surfaces or over several weirs, in order to promote the escape of air. To this drum a vacuum-pump is attached, which in some cases is arranged to work automatically. Some plants are successfully operated without a separator.

338. *Sand-box.*—Where sand is drawn up with the water it may be got rid of by passing the water at a slow velocity through a large

* Quoted in Goodell's Water-works for Small Cities and Towns, p. 119.

drum or box inserted in the suction-pipe and provided with suitable hand-holes for cleaning.

339. **The Clogging of Wells** by filling with sand or by corrosion of the screen is a frequent occurrence and may reduce the yield very greatly. Wells may be readily cleaned of sand by means of the sand-pump or bucket, but if the strainers are corroded they must be pulled up, cleaned or renewed, and replaced. If the clogging is due to fine sand collecting about the outside of tube, it may be removed to some extent by forcing water into the wells under high pressure,* or by the use of a hose, or by means of a steam-jet. Sometimes instead of the yield of a well becoming less through continued operation it is actually increased, owing probably to the gradual removal of the finer material immediately surrounding the well.

340. **Tests.**—Besides the preliminary tests already mentioned for determining the character of the strata, slope, and flow of ground-water, spacing of wells, etc., a tube-well system should always on completion be subjected to a thorough test as to capacity. Such tests should be continued until the ground-water level has reached a state of equilibrium as determined by careful observations at the wells and at various distances therefrom. If possible the tests should extend over the dry months of the year, and where the system is built by contract to supply a certain amount of water, the successful operation of the works for a year under a definite head should be a prerequisite to final acceptance. If the quantity pumped is large in proportion to the capacity of the ground, a long time will elapse before the ground-water will cease to fall. The case mentioned on page 291 is instructive in this connection.

341. **Yield.**—The maximum possible yield of a ground-water source would be when the entire flow is utilized. With a system of wells this can be accomplished only in case the water can be drawn to the bottom of the porous stratum, or can be drawn so low that there is no head to cause flow away from the wells on the lower side. If the wells are located near a body of water and the level of the water in the wells is kept as low as that of the surface-water, the entire flow will then be utilized. In special cases an artificial dam can be constructed and a line of wells or a gallery placed above, as at Daggett, Cal. (Art. 357). Ordinarily, however, only a part of the flow is intercepted, the proportion depending upon the actual lowering compared to the maximum as above explained.

The actual yield in any case will depend, of course, upon the conditions relative to the ground-water; but where these conditions have

* This method is regularly employed at Memphis. See *Eng. Record*, 1902, XLVI. p. 513.

been favorable, as at Brooklyn, yields of 300,000 to 500,000 gallons per day per 100 feet of suction-main are common.* At Plainfield the yield is about 250,000 gallons per 100 feet. Conditions are often less favorable than at these places, and yields are likely to be much less; but if the ground-water is distributed so thinly that the yield would be but 20,000 to 30,000 gallons per day per 100 feet, the cost of suction-main, wells, land, etc., would render a ground-water project very expensive. It would, however, be rarely possible in such a case to find water-bearing strata sufficient in extent to furnish any except very small supplies.

342. Examples.—The tubular-well system at Plainfield, N. J., is an example of a very successful plant (Fig. 59). It consists of twenty 6-inch

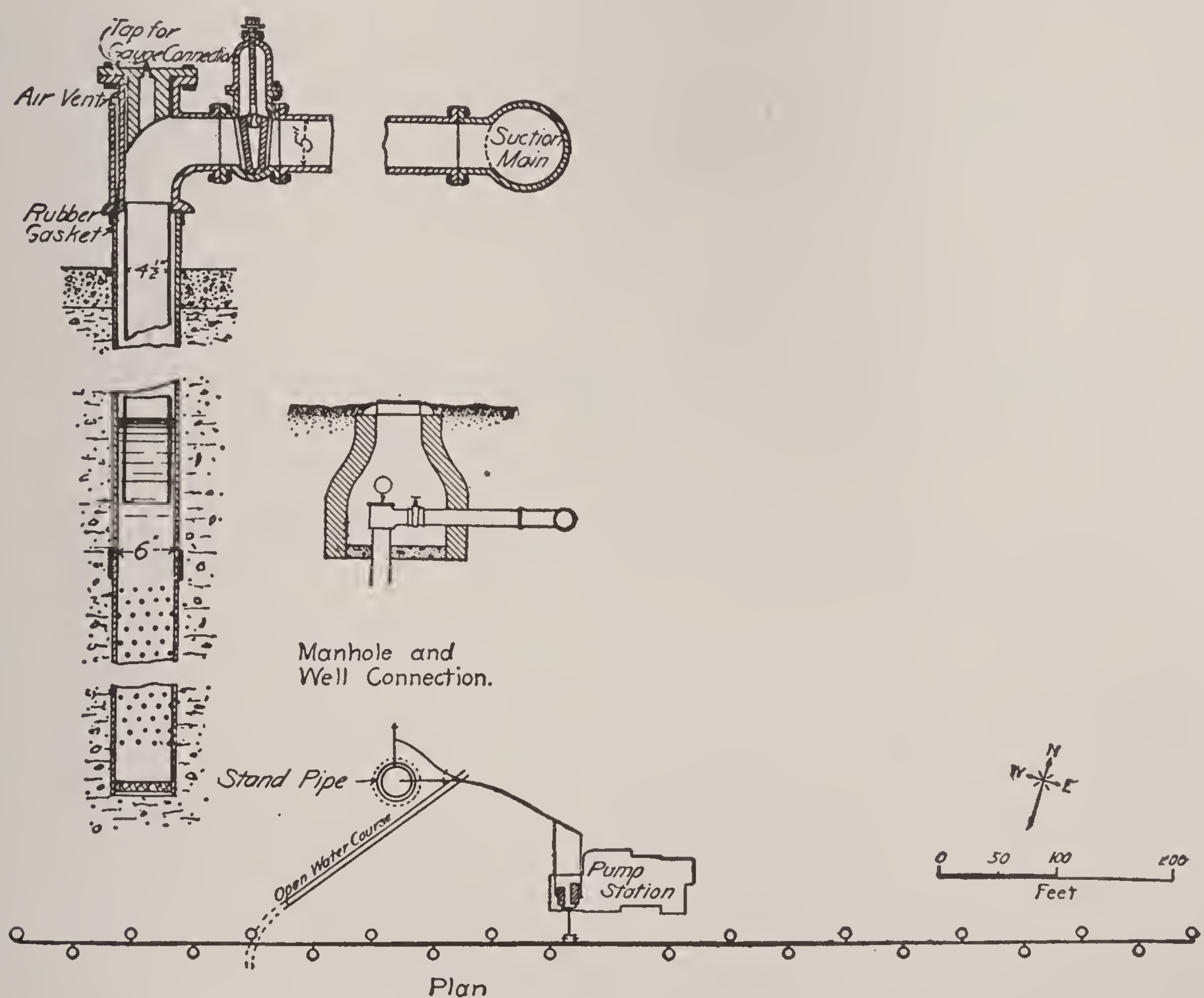

FIG. 59.—TUBULAR WELLS AT PLAINFIELD, N. J.

wells of perforated pipe, open at the lower end and provided with separate 4½-inch suction-pipes. The wells are from 35 to 50 feet deep, and are sunk into a coarse water-bearing gravel overlaid by clay. They are spaced about 50 feet apart and are connected to the suction-main by 5-inch branches. The suction-main varies in size from 8 to 12 inches. The ground-water at this place has a slope of about 3 feet in 1900, indicating a copious flow. A 24-hour pumping-test indicated a yield of about 150,000 gallons per day per well with ten wells connected, and a depression of water-level of only

* The estimated underflow of Long Island is at least 10 in. per year or 475,000 gals. per day per sq. mi. of watershed.

2 feet. At 700 feet distant the lowering in a test well was 0.6 foot. With more wells operated the yield per well was less. From Table No. 55, page 288, it is seen that the size of the well is none too great for the yield. Fig. 59 shows the arrangement of wells and details of well and manhole. The cap of the well is tapped for a vacuum-gauge connection.*

A very economically constructed system is that at Brookline, Mass. The wells, of which there are 160, are 2½ inches in diameter and from 35 to 95 feet deep. They are open at the bottom and perforated for the lower 2 feet, the holes being bushed with ⅜ inch brass pipe. They are arranged along a suction-main about 6000 feet long. Each well is connected to this main by means of two short pieces of lead pipe between which is placed a gate-valve. The cost of driving and connecting up 118 good wells averaging 50 feet deep, including work done in driving and pulling up 41 unsuccessful wells, was \$47.90 per well. The average rate of driving with four men was 50 feet per day, at a cost of 21 cents per foot for labor.†

The city of Brooklyn, N. Y., obtains a large proportion of its water-supply from small wells. In 1895, of the total supply of about 75 million gallons, about 35 million was from wells, and an additional well-supply of 25 million gallons was contracted for. The old wells are nearly all of the small closed type and are grouped at six stations. Fig. 60 shows a plan of one of

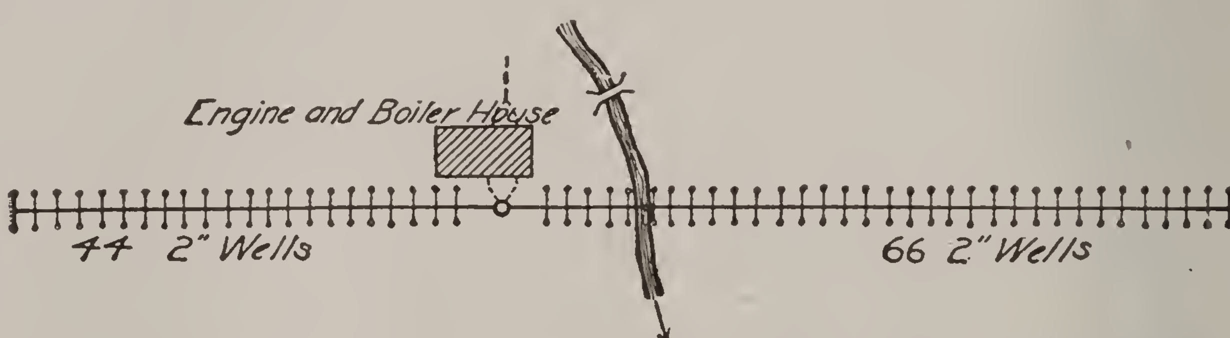

FIG. 60.—FOREST-STREAM DRIVEN-WELL STATION, BROOKLYN, N. Y.

these older stations.‡ The arrangement is quite similar in all, the wells being placed in two rows, one on each side of the suction-main. The later plants are to consist of open wells perforated and covered with screens, with suction-pipes placed inside. A description of one of the new plants is as follows: The main suctions are about 2340 feet long with a fall of 12 inches from centre to each end. The 62 wells are staggered along the main suction-pipe, 12 feet from it and 75 feet apart on each side. Their average depth is 45 feet, a stratum of fine sharp sand being met with at that depth. The outside casing is 4½ inches, with 6-foot strainer, 2-foot sand-pocket, and 6-inch point. Suctions are 3 inches in diameter and 28 feet long. Lateral branches are 3½ inches, and each is provided with a gate. It is expected to get 6 million gallons from this station. The contract price for the last 25 millions was \$167,250 for sinking and connecting wells, the yield to be determined by a test lasting one year and taken as the lowest average for five consecutive days.§

* Trans. Am. Soc. C. E., 1894, XXXI. p. 371.

† Jour. New Eng. W. W. Assn., 1897, XI. p. 196.

‡ Brooklyn Water-supply, Plate 25.

§ See Report of Commission on Additional Water-supply of New York, 1903, for further data.

Deep and Artesian Wells.

343. **Comparison with Shallow Wells.**—Where the depth exceeds 75 to 100 feet the small driven well is no longer practicable. The expense of construction per well now becomes much greater, preliminary investigation much more difficult, and the problem altogether requires more careful consideration. Fortunately the deeper strata are usually more uniform and of greater extent than strata near the surface, so that in regions already explored deep wells can be sunk with far more certainty of success than is usually the case with shallow wells. Methods of sinking deep wells are in many respects different from those already described, and matters of spacing, pipe-friction, arrangement of connections, etc., are much more important than in the shallow-well plant.

344. **Boring Deep Wells.**—Well-boring is an art by itself, and the execution of any deep-well project should usually be put into the hands of some reliable well-drilling concern. The variety of ingenious tools and appliances in use for overcoming all kinds of difficulties and for penetrating all sorts of strata is very great, and it is possible to give here but a very general description of some of the methods of sinking in use. The methods used for soft and for hard materials are very different, and the subject will be divided accordingly.

345. *Sinking of Wells in Soft Materials.*—In soft material it is of course necessary to case the well the entire depth, and on account of the difficulty of getting the casing down to great depths this operation becomes the chief feature of the construction.

For depths up to 200 or 300 feet the ordinary well-drilling outfit can be used, and the casing driven close after the drill. By the use of an expansive drill the hole can be made slightly larger than the casing, thus making it possible to drive the casing much farther, and even enabling strata of soft rock to be passed. When the casing can be driven no farther a smaller size is inserted and the sinking continued with a smaller drill, and so on until the well is sunk as far as desirable or possible. The material excavated is brought to the surface by means of a sand-bucket, or by the water-jet as previously described in Art. 329, the water being conducted to the end of the drill through hollow drill-rods. By the latter method the hole is kept clean and a more rapid progress made.

The friction against the casing is greatly lessened, and the depth attainable much increased by the use of the revolving process. In this the lower end of the casing is provided with a toothed cutting-shoe of hard steel of slightly greater diameter than the pipe, and the upper end is connected by means of a swivel to a water-pipe through which water is forced by suitable pumps. The well is bored by turning the pipe, and the loosened material is carried to the surface by the water which passes down inside the casing and up on the outside between casing and soil. So long as the water-pressure is maintained there is very little friction between the earth and the pipe, and the tube is readily rotated and sunk at the same time. This process is very common in sinking artesian wells in the alluvial basins of California. It is very rapid, a rate of sinking as high as 20 or 30 feet per hour for depths of 1000 feet having been attained.

It is essential to have a good length of strainer in the porous stratum. This is usually inserted after the desired depth has been reached, and the casing is then pulled up to the top of the strainer. By special devices it can, however, be attached to the end of the well-casing and sunk with it.

A very efficient method of well-sinking in deposits of sand and gravel is that so largely employed in Southern California and known as the "stove-pipe" method of construction. Wells of this type are put down in gravel and boulder deposits or other unconsolidated material to depths as great as 1300 feet. The usual size ranges from 8 to 14 inches. After the first length the casing consists of short sections, two feet long, of No. 12 riveted sheet steel. It is of double thickness, and is made up as the well is sunk by telescoping the sections together using alternately an "inside" and an "outside" section, breaking joints at the center of each section. The pipe is thus smooth both inside and outside.

The material from the inside is usually removed by a large sand bucket operated with jars, and the casing is forced down by heavy hydraulic jacks. After the well is sunk the casing is slotted by special perforating knives which operate very effectively. The cost of such wells is remarkably low, a 500-foot well costing in 1903 about $700, not including the casing.

The chief advantages of this type of well consist in its strength of joint, smoothness, convenience of construction, and low cost. The large size is also very advantageous where boulders are encountered, and where large yields are met with.*

346. *Examples.*—At Memphis, 8-inch wells were sunk by the jetting process. A 10-inch well was first sunk 70 to 90 feet to clay, to cut off undesirable

* *Eng. News*, 1903, L. p. 428.

water, and the 8-inch pipe was then forced down by means of an hydraulic jack anchored to the 10-inch pipe. In this way a depth of 600 feet could be reached. In one case a 6-inch pipe was continued inside the 8-inch to a total depth of 1165 feet. Cook strainers 50 feet long were used, ½ inch smaller than the 8-inch casing, and closed at the bottom. After being lowered the casing was pulled up nearly to the top of the strainer, and a ring packing of rubber and brass driven between strainer and tube.*

A very deep well was sunk by the revolving process at Galveston, Texas. The first casing was 22 inches in diameter and was sunk 57 feet. This was followed by 15-inch casing to a depth of 870 feet; then, following this, pipes of 12-inch, 9 inch, 8-inch, 7-inch, 6-inch, and 5-inch diameter were used, with which a depth of 3067 feet was reached. The contract price was $75,000 for a depth of 3000 feet. The cost of the plant ready for work was $12,000. A water-pressure of 250 pounds per square inch was used in sinking.†

347. *Boring Wells in Rock.*—A drilling outfit for deep wells is very similar to the ordinary familiar outfit for shallow wells worked by horse-power. A string of tools consists essentially of a steel bit, an auger-stem into which the bit is screwed, a pair of links or "jars" connecting the auger-stem with another bar, called a sinker-bar, and finally the rope cable which supports the apparatus and which passes over a pulley at the top of a derrick and then down to a winding drum. Just above the drum the cable is attached, by means of an adjusting or "temper" screw, to a large walking-beam operated by a steam-engine. As the work progresses the drill is lowered by the temper-screw. By means of the jars an upward blow may be struck to dislodge a jammed drill. Many ingenious tools are employed for recovering lost tools, cutting up and removing pipe, and carrying on the various operations involved.

Wooden poles are sometimes employed to support the drill, but this system is not much used in the United States. Its advantage lies in the greater command the driller has over the drill, both in turning it and in maintaining a straight hole. Its chief disadvantage is in the length of time required to remove the drill from the hole.

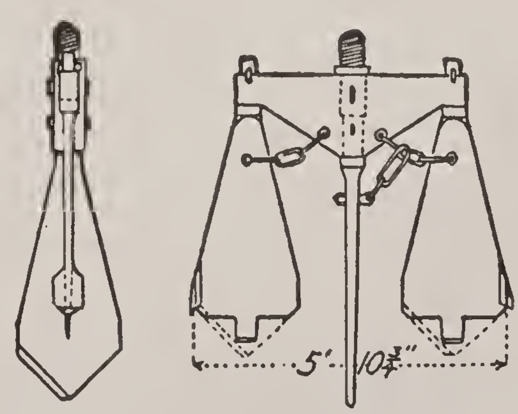

FIG. 61.—LARGE WELL-BORING APPARATUS.

In this country deep wells are almost invariably of small diameter. In Europe, however, deep-bore wells are frequently constructed of a diameter of 2 to 4 feet and even as large as 6 feet. One such well is the Place Herbert well of Paris, 3½ feet in diameter and

* *Eng. Record*, 1891, XXIV. p. 234.
† *Eng. News*, 1892, XXVIII. p. 122.

2536 feet deep.* At Southampton, England, two 6-foot wells spaced $11\frac{1}{2}$ feet apart were bored in chalk to a depth of 100 feet. They were intended to serve also as pump-pits. The boring apparatus used at this place is illustrated in Fig. 61.†

As already shown in Art. 309 the effect of size of well upon the yield is not great, so that in this respect large, deep wells are of doubtful economy. It may, however, often be economical to build a well large enough to serve as a pump-pit, thus permitting the use of a more economical type of pump than would otherwise be possible. (See also Art. 352.)

348. Casing of Wells. — Wells in soft material must be cased throughout. When bored in rock it is necessary to case the well at least through the soft upper strata to prevent caving. Casing is also desirable for the purpose of excluding surface-water, to which end it should extend well into the solid stratum below. Where artesian conditions exist and the water will eventually stand higher in the well than the adjacent ground-water, the casing must extend into and make a tight joint with the impervious stratum, otherwise water will escape into the ground above.

A reliable joint can best be made by sinking a smaller tube inside the outer one, and filling the space between it and the well or outer tube by some form of packing. Rubber packing-rings are used for this purpose, which if well placed are very effective. They are hollow cylinders of soft rubber inserted in an adjustable length of tubing of a diameter smaller than the well, and when lowered to the proper place are expanded to fill the annular space by screwing up the tubing. Linseed bags wrapped around the tubing have been much used for packing. The seed expands as it becomes water-soaked and so fills the space. Lead packing has also been used with satisfactory results. It is first cast around the end of the pipe, the pipe lowered in place, and the joint made by driving the pipe firmly into the hole, which has previously been reamed out smooth.

If two or more water-bearing strata are encountered, the water-pressures in the different strata are likely to be different, that from the lower usually being the greater. If it is desired to utilize only the water from the lower stratum at its maximum head, it will be necessary to place the packing in the impervious stratum immediately overlying the one in question. Otherwise the head attained will be more nearly

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* *Eng. News*, 1888, xix. p. 488.

† Proc. Inst. C. E., xc. p. 33.

that of the upper strata. Where different pressures thus exist it is only possible to determine their amount by separately testing each stratum as reached, the others being cased off. This operation is an essential part of the boring and should be carefully performed. Important differences in quality are also often discovered in this way.

In placing permanent packing it does not necessarily follow that certain strata should be excluded because of a less static head than others. None need be excluded from which the head is greater than the head existing at the level of the stratum when the works are in operation. The question, therefore, depends upon the amount it is proposed to lower the water by pumping.

Ordinary artesian well-casing is made of light-weight wrought-iron lap-welded pipe. For pipe which is to be driven the standard wrought-iron pipe is ordinarily used, but for heavy driving extra strong pipe is necessary. Joints of drive pipe should be made so that the ends of the tubing are in contact when screwed up. The life of a good heavy pipe is ordinarily very great, but cases have occurred where the pipe has been rapidly corroded, due to the presence of excessive amounts of carbonic acid.

349. Cost.—The cost of sinking wells will of course vary greatly according to locality, nature of strata, and depth and size of well. For wells 6 to 8 inches in diameter and sunk in ordinary rock the cost per foot, not including casing, will usually range from $2.00 to $3.00 for depths of 500 feet, up to $3.00 to $5.00 for depths of 2000 feet. For smaller sizes the cost will be somewhat less, especially for the shallow depths. In the Dakotas 2-inch wells have been sunk 285 feet for 42 cents per foot, and forty-two such wells with an average depth of 322 feet had an average cost of 78 cents per foot. Six-inch wells cost from $5.00 to $6.00 per foot complete for depths of 500 to 1000 feet.*

In soft material the cost for small depths will be somewhat lower than in rock, but for great depths much higher, on account of the difficulty of sinking.

For sizes much exceeding 12 inches the cost will rapidly increase. The cost of the large 6-foot wells at Southampton already referred to was about $25.00 per foot for boring, and $20.00 for casing.

350. Arrangement.—The best arrangement of deep wells is in a straight line at right angles to the line of flow, but the latter point is of much less importance than with shallow wells in a limited water-

* Eleventh Census, Report on Agriculture by Irrigation.

bearing stratum; as, owing to the lateral extent of the strata and slight inclination of the hydraulic grade-line, water will flow towards a small group of wells nearly equally from all directions.

351. **Size and Spacing.**—The high cost of deep wells renders a thorough preliminary investigation relative to proper size and spacing impracticable, but for the same reason, a correct determination of these points is of great importance. The desired knowledge must be got by a study of similar plants, or gained as the sinking of the wells progresses.

To be able to draw correct inferences from tests of artesian wells it is very essential that water from the upper strata be carefully excluded. Static heads can then be measured by allowing the water to rise in an open pipe, or by means of a gauge. The difference between this static head and the head measured when the well is flowing or being pumped from is then partly consumed in overcoming friction in the well and partly in forcing water into the well through the porous strata. The effect of a change in size of well and of head can then be readily estimated, in accordance with the principles of Art. 311. As a valuable aid in such calculations the flow at different heads should be determined and a discharge curve drawn. This gives directly the effect of variation of head in the particular well tested, and also enables the matter of friction to be more accurately determined.

The economical spacing for deep wells will be much greater than for shallow wells. It will likewise pay to spend much more money in lowering the flow-line by making deep connections, thus decreasing the number of wells and increasing the spacing. The questions of size, spacing, and connections are interdependent, and also depend upon the economy of different types of pumps; and a correct solution requires a careful study of these questions, together with many others depending upon local conditions. It will often be necessary to estimate on several different arrangements before the best one is arrived at.

Spacings in some carefully constructed works which have apparently given satisfactory results are: At Galveston, Texas, thirty 7-in. wells, about 800 feet deep and 560 feet apart, yield about 250,000 gallons per well. At Memphis, forty-two 6- and 8-inch wells about 400 feet deep were spaced at first 75 feet apart and afterwards 250 feet. Maximum flow = about 250,000 gallons per well. At Savannah, Ga., 12-inch wells 500 feet deep were spaced 300 feet apart. Yield =

* Eleventh Census, Report on Agriculture by Irrigation.

about 800,000 gallons per day per well. At Galveston, the connections were made at a slight depth below the surface; at Memphis, in a tunnel 80 to 90 feet deep; and at Savannah, in a conduit 20 feet deep. At Fort Worth, Texas, thirteen 8-inch wells 1000 feet deep were placed 800 feet apart. The total yield was about 1,000,000 gallons, although the first two or three wells indicated a total of 3,000,000 gallons. At Madison, Wisconsin, a spacing of 600 to 800 feet is found desirable for wells in the Potsdam sandstone when operated under about 15 feet of head. The yield under these conditions is about 300,000 gallons per day each.

352. **Methods of Operation.** — On account of the relatively great cost of deep wells it will often be found economical to so arrange the pumps and connections that a considerable lowering of the water-level below the ground-surface may be obtained. This is generally accomplished by connecting all the wells to a single pump or set of pumps, placed at a greater or less depth below the surface. Where the connections are very deep tunneling may have to be resorted to. Another common method of drawing water from deep wells in the case of small plants is by the use of a separate deep-well pump for each well. The usual type of deep-well reciprocating pump used in such cases is generally of very low efficiency and small capacity and not adapted to large supplies. The air-lift is also of comparatively low efficiency, but is a very flexible system, and in many cases can be used to advantage in relatively large works. Where the yield per well is large the most economical method of deep pumping is probably the use of the small multiple-stage centrifugal pump. These pumps are made to fit casings of 15 to 20-inch diameter and may conveniently be direct-connected to vertical electric motors operated from a central station. The well need be made of the large size only to the depth desired for the pump. (For further discussion of the question of pumping machinery see Chapter XXVI.)

353. **Examples of Artesian-well Plants.** — In the majority of cases an artesian-well plant, where consisting of several wells, is operated in the same way as a system of shallow wells, a good illustration of which is the Plainfield works described on page 307. Two noteworthy exceptions to this arrangement are the large works at Memphis, Tenn., and at Rockford, Ill., — plants which represent the most modern practice in this branch of water-works engineering.

The supply at Memphis is obtained from a series of forty-two wells sunk about 400 feet deep to a stratum of water-bearing sand. Little natural flow was obtained, and, to increase the yield, the pumps were located in a pump-pit about 50 feet deep and connections made with the wells by means of tunnels. Vertical compound engines were adopted having a duty of about

115,000,000 foot-pounds per 1000 pounds of steam.* Mr. T. T. Johnston was the engineer.

A later example and one involving several interesting features is the plant at Rockford, Ill., Mr. D. W. Mead, Mem. Am. Soc. C. E., engineer.

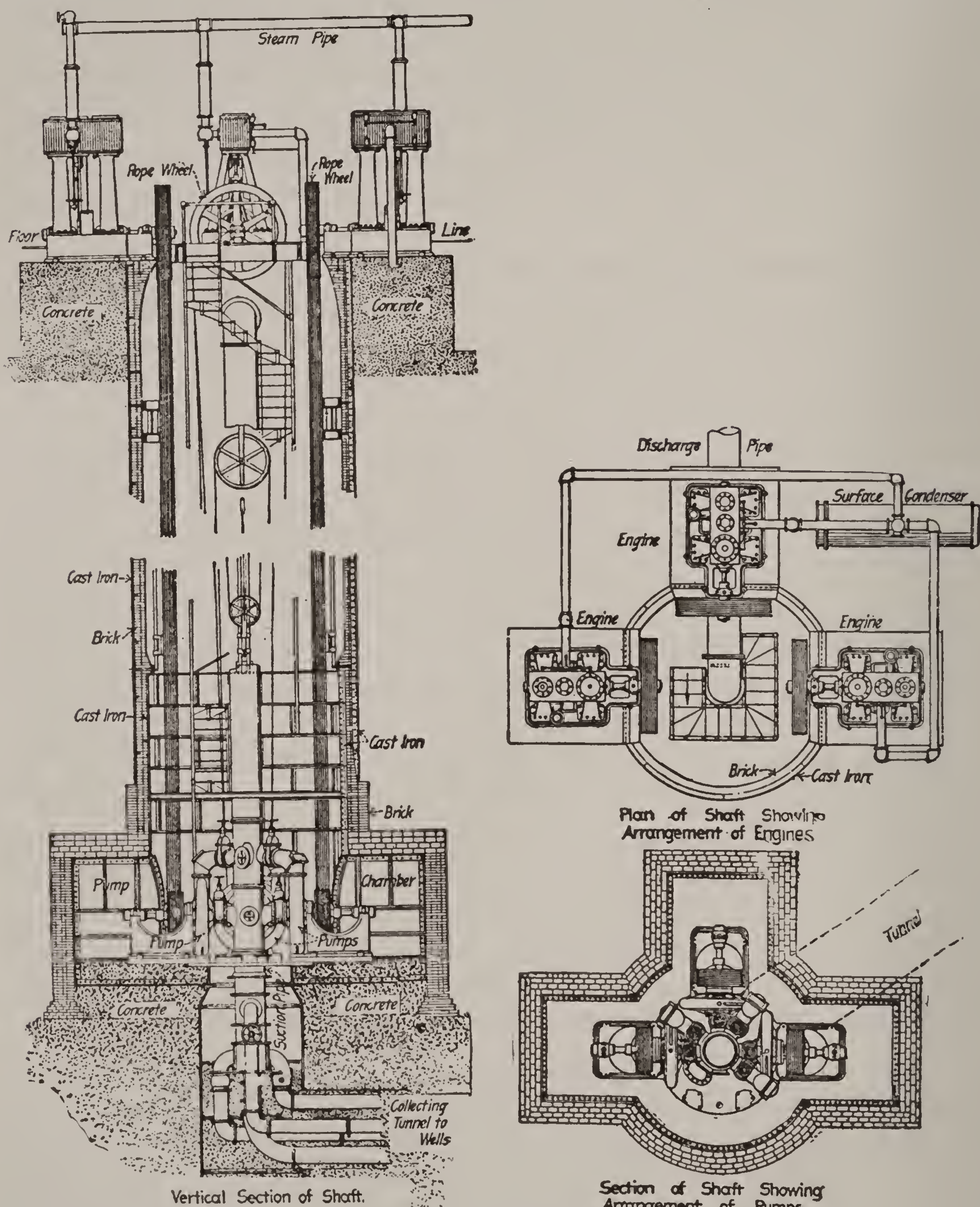

FIG. 62.—PUMP-SHAFT AND PUMPS, ROCKFORD, ILL.
(From *Engineering News*, vol. XLII.)

Previous to 1898 the city had been supplied for some years by artesian wells, part sunk to the Potsdam sandstone and part into the St. Peter (see Fig. 42, page 112). Increased demand necessitated increased lowering of the

* *Eng. Record*, 1891, XXIV. p. 234.

water-level, a result temporarily accomplished by means of deep-well pumps and by the air-lift. The arrangement adopted for the new plant was to sink a shaft 95 feet deep, place pumps therein, and connect them to the various wells through tunnels constructed from the bottom of the shaft. The arrangement is clearly shown in Fig. 62. The shaft is water-tight, with a floor of concrete, and the lateral tunnels are filled with concrete, put in after the suction-pipes were placed. The two pumps, each of 3 million gallons ordinary capacity, are of the centrifugal type and are operated through rope transmission by compound engines placed at the surface. Under a head of over 100 feet the pumps alone developed an efficiency of from 70 to 75 per cent, and the whole plant a duty of 58,000,000 foot-pounds per 1000 pounds of dry steam. For further data regarding these pumps see Chapter XXVI.

354. **Yields.**—In making estimates regarding flow it is important to bear in mind that it requires a considerable length of time to determine with certainty the adequacy of the supply, and furthermore that the sinking of wells by other interests, even though at considerable distances, may very seriously affect the yield. The amount of water flowing through a porous rock, per square foot of cross-section, is likely to be very much less than that which is often found in coarse sand and gravel strata. The slope of the hydraulic grade-line in the former case is usually very much less and the friction much greater, large volumes depending on great thickness and breadth of strata. Detailed figures relating to yield for several supplies have been given on page 315. Where conditions are sufficiently favorable for works of some magnitude the yield per well under a moderate head ranges from about 150,000 gallons per day to 800,000 gallons, or even more. With yields of less than 100,000 gallons per day, works for developing large quantities become very expensive, relatively more expensive than for small quantities, since with a large number of wells there is much greater interference.

355. **Failure of Wells.**—The chief cause of a decrease in yield of a well is the influence of other wells sunk in the vicinity. This effect is likely to be felt much farther than when similar quantities are drawn from gravel strata, on account of the great depth to which the head, or flow-line, is often reduced in deep wells by pumping. The case of the failure of wells in the vicinity of Chicago, and its effect for several miles around, has already been mentioned. At Savannah it is estimated that the withdrawal of 10 million gallons per day affects the pressure for 8 miles distant. At Dubuque, Iowa, a large well bored on low ground caused the flow from several wells higher up to cease entirely. At Denver the flow has also greatly decreased and varies with the season.

The yield of a well may also decrease on account of causes inherent in the well itself. One such cause is leakage due to defective packing, this being a common fault of wells in the Dakota basin. Another cause of partial failure is by clogging through the inflowing of fine sand. This can be removed by the methods described on page 306.

The gradual lowering of head due to long-continued operation is well illustrated by Fig. 63 which represents the conditions at Memphis.* In a report of a Commission of Engineers in 1902 it is stated that in 1898 the pumpage was about 19,000,000 gallons per day, and in 1902 about

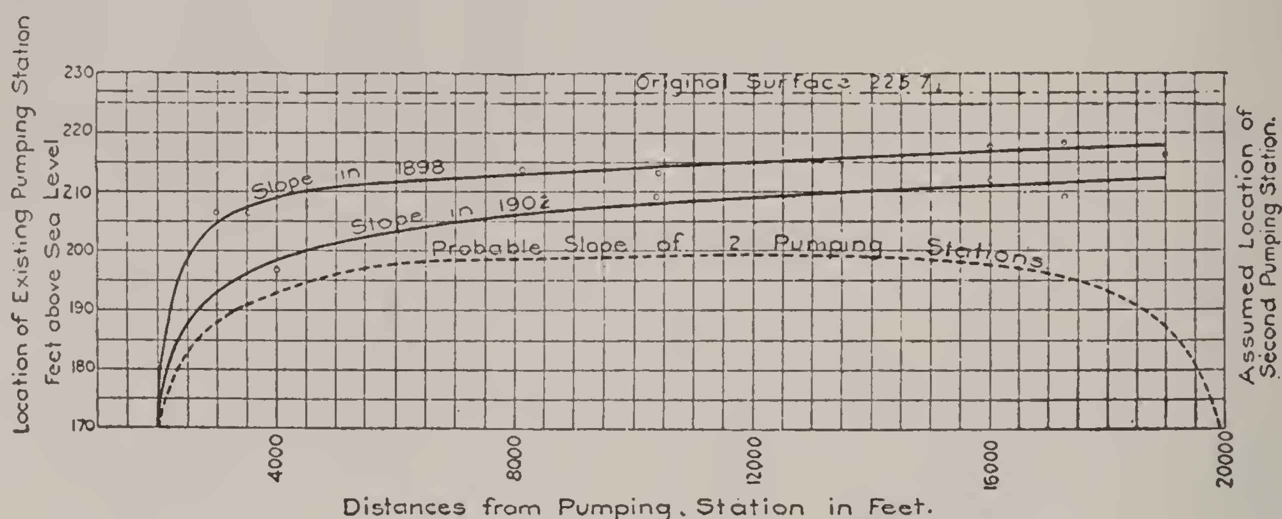

FIG. 63.—GROUND-WATER CURVES AT MEMPHIS, TENN.
(From *Engineering Record*, vol. XLVI.)

12,000,000 gallons. It is estimated that a second station and group of wells might be established about four miles distant with a maximum capacity of each group of about 15,000,000 gallons per day under a head of about 60 feet.

HORIZONTAL GALLERIES AND WELLS.

356. Filter-galleries.—Where ground-water can be reached at moderate depths it is sometimes intercepted by galleries constructed across the line of flow. If these are placed at a sufficient depth they will evidently enable the entire flow of the ground-water to be intercepted. In form a gallery may consist merely of an open ditch which leads the water away, or it may be a closed conduit of masonry, wood, iron, or vitrified clay pipe, provided with numerous small openings to allow the entrance of the water. Unless constantly submerged, wood should not be used. Masonry and vitrified pipe are preferable to iron, as these materials are uninjured by exposure to water. If galleries are not covered, excessive vegetable growth is apt to occur which may injure the quality of the water.

Galleries are usually constructed in open trench. To prevent the entrance of fine material the back-filling near the openings should be

* *Eng. Record*, 1902, XLVI. p. 514.

of gravel of graded size, and as an additional precaution the openings may be made in the bottom only. Manholes should be provided to permit of inspection and cleaning. Galleries need be of a size only large enough to carry the estimated quantity, or, in case trouble with sand is feared, large enough to permit of inspection. They are arranged to lead the water to the pump-well, and may be provided with gates so that the water may be shut off from various sections.

The cost of galleries is about the same as that of sewers in similar ground. It increases rapidly with the depth, but up to a depth of 20 or 25 feet it is sufficiently low so that the construction of galleries can often be advantageously undertaken. A gallery not only intercepts the water more completely than wells, but it replaces the suction-pipe, it is more durable than either pipe or wells, and all trouble from the pumping of air is avoided.

Where conditions are favorable surface-water may sometimes be used to augment a ground-water supply, thus using the natural soil as a filter instead of constructing an artificial filter. This system would be suitable only for removing suspended matter as it would be too unreliable to deal with sewage polluted water.

357. **Examples.**—There are comparatively few cases in this country where galleries have been used to intercept ground-water proper, they being mainly used to collect filtered stream-water as explained in Art. 359. As an example of their use for ground-water collectors, mention may be made of the gallery at Naples, Italy. The supply there is taken from a gravel stratum 10 to 13 feet thick, overlaid by 30 feet of clay. The gallery is 2000 feet long. It was constructed in open trench and back-filled with gravel and clay. The yield is 38 million gallons per day.*

The city of Munich, Germany, gets its water-supply from springs and collecting-galleries in the foothills of the Alps. The water occurs in a deposit of gravel resting upon a bed of sandstone, and is intercepted by collecting-tunnels driven nearly horizontally, and located partly in the sandstone and partly in the overlying gravel. From these galleries the water is conveyed to an aqueduct leading to the city. Fig. 64 shows the general arrangement and an enlarged cross-section of a collecting-tunnel.†

Karlsruhe obtains its water-supply from an old river-bed filled with gravel, the water is impounded by a clay wall or dam and collected by a gallery which yields 1,600,000 gallons per day.‡

Many cities in Holland secure their supplies from open ditches and galleries in the sand-dunes. Amsterdam collects in this way about 10 inches out of a total rainfall of 30 inches per year. Other yields have been obtained as high as 20 to 25 inches.

Galleries have been constructed at several places in the West for collecting

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* Proc. Inst. C. E., LXXXIV. p. 468.

† *Eng. Record*, 1898, XXXVIII. p. 78.

‡ *Jour. f. Gasbel. u. Wasservers.*, 1894, p. 269.

water from the large gravel deposits beneath and adjacent to the streams. Such are the older works of Denver* and of Golden,† Colorado, and Eureka,‡ Cal. These galleries usually consist of wooden boxes or cribs constructed at a considerable depth in the water-bearing gravel, and are often arranged to

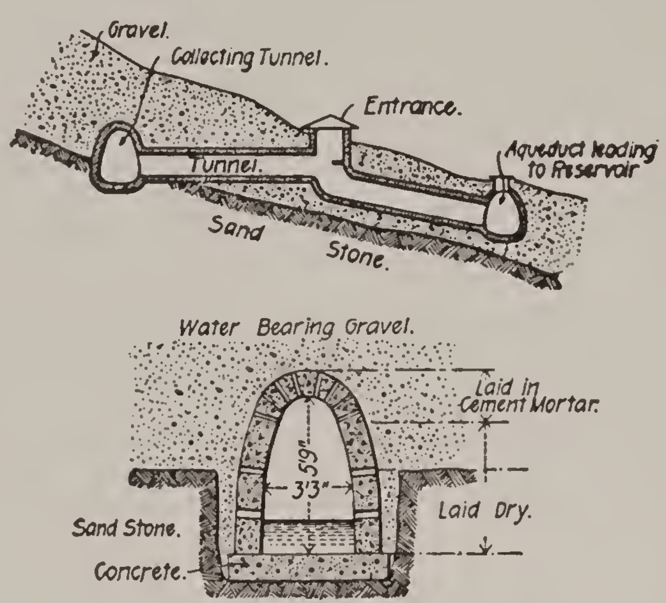

FIG. 64.—COLLECTING-GALLERIES, MUNICH WATER-WORKS.
(From *Engineering Record*, vol. XXXVIII.)

collect not only ground-water but filtered surface-water from the streams. At Eureka provision is made for back-flushing.

Los Angeles obtains its supply of about 26,000,000 gallons per day from galleries in the underflow. Vitrified pipe and concrete galleries are used.§

At Daggett, Cal., water for irrigation is obtained from underground streams by means of a flume, the water being dammed up by sheet-piling driven across the line of flow. Surface-water flows here also after heavy storms.‖

Another notable case of a subsurface dam is that at Pacoima Creek, Col. Here the dam is from 25 to 50 feet deep and consists of a two-foot concrete well. The water is collected by means of horizontal rows of concrete pipes laid about 10 feet apart and leading to two collecting wells.¶

358. Tunnels in Rock.—Galleries for collecting ground-water are occasionally tunneled in solid rock. This may happen along a side hill where an outcropping porous stratum overlies an impervious one and it is desired to develop the flow by running a tunnel along the hill near the bottom of the porous stratum.

Tunnels or galleries are also sometimes run from the bottom of large wells for the purpose of increasing the yield. This method of

* Trans. Am. Soc. C. E. 1894, XXXI. p. 135.
† *Eng. News*, 1891, XXV. p. 610.
‡ *Ibid.*, p. 339.
§ *Ibid.*, 1906, LV. p. 595.
‖ *Ibid.*, 1896, XXXVI. p. 157.
¶ W. S. Paper, No. 69, U. S. G. S., 1902; also Eighteenth Annual Report, U. S. G. S., Part IV, p. 693.

increasing the flow is advantageous where it is necessary to lower the pumps and to concentrate the flow in a single well.

359. Wells and Galleries Near Streams.—Wells and galleries are often constructed near streams for the purpose of getting all or a portion of the supply therefrom; and, on the other hand, it often happens that wells sunk near streams obtain much of their water from them when they are supposed to get it all from the opposite direction. In general the natural ground-water surface will slope towards a stream as shown in Fig. 17, page 90; and until a well is pumped from, the water-level therein will stand higher than the surface of the water in the stream. The amount the water in the well can now be lowered without drawing from the river depends upon the distance of the well from the stream and upon the ground-water slope; but no water can enter from the river so long as there is a summit in the ground-water surface between the stream and well. A few test-borings placed between the well and stream will determine this point. The source of water can also often be determined by chemical analysis.

Where wells or galleries are placed near streams for the purpose of obtaining surface-water filtered through the ground, the success of such works depends much upon the character of the river-bottom. Even when the lower strata are porous, the river, if a silt-bearing one, may have a nearly impervious bottom and the natural filter will only become more clogged by use, necessitating perhaps the abandonment of the collecting-works. Such failures have occurred in some instances. With a sandy river-bottom kept clean by the scouring action of the floods, and with a porous substratum, works of this kind will give good results. To secure good filtration the works should be located at least 50 feet and preferably a greater distance from the stream. Galleries of the kind here considered have sometimes been built of great width, but, as most of the filtration must be lateral, very little is gained by increasing the width over that required for convenience in construction and inspection.

The city of Painsville, O., has adopted this method of securing water from Lake Erie. Wooden galleries 500 feet long constructed close to the lake receive about 1,000,000 gallons per day.* Galleries are also successfully used at Laredo, Texas, under a sandy island in the Rio Grande river.†

At Nancy, France, a system of double filtration has been devised to

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* *Eng. Record*, 1901, XLIII. p. 518.

† *Ibid.*, Jan. 9, 1904.

overcome difficulties due to the silting up of the original collecting galleries receiving water from the river. The river-water now passes through a rough filter of coarse gravel and thence into a long distributing chamber whence it flows through the natural sand and gravel to the filter gallery.*

The yield of a series of wells or of a gallery collecting filtered surface-water will be, as in the case previously discussed, proportional to the lowering of the water-level, or to the head on the filter, and will be nearly proportional to the length of the line of works. In gallons per day per 100 feet of gallery, the yield from various successful works varies from 30,000 to 1,000,000 or more, which is about the same as is obtained from lines of wells.

360. **Horizontal or Push Wells** are tubular wells pushed approximately horizontally into a water-bearing stratum, or under the bed of a lake or stream. They are forced into the ground from a trench by means of jacks braced against the opposite side. These wells have been most successful when extended out under a body of water. At South Haven, Mich., three 6-inch Cook wells with 30-foot strainers were pushed out 150 feet under Lake Michigan. The wells were 25 feet apart and yielded 1,250,000 gallons per day. Another similar plant at Crystal City, Mo., consisted of two 8-inch wells with 65 feet of strainer, extending 200 feet under the Mississippi River in a sand stratum 20 feet thick. The yield was 1,300,000 gallons per day.†

361. **Filter-cribs.**—Another method of utilizing a river-bottom as a natural filter is to construct a wooden crib in an excavation in the bed of the stream, fill it with graded gravel and then cover the structure with 3 or 4 feet of sand up even with the river-bottom. The suction-pipe then leads from the crib to the pumps. This form of construction is well adapted to sandy-bottom streams with swift currents and has proved a very efficient way of clarifying muddy river-waters. The rate of filtration through such a filter may be quite closely estimated according to the principles explained in Chapter XXI.

In Fig. 65 is illustrated a crib at Kensington, Pa., similar to several which have been constructed in the Allegheny River. This crib is 200 feet long, 32 feet wide, and 4 feet high, and is covered with 4 feet of gravel and sand dredged from the river-bottom. It is designed for a capacity of 3 million gallons per day, equal to a rate of

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* *Eng. Record*, 1905, LI. p. 148.
† *Eng. News*, 1893, XXIX. p. 452.

filtration of about 16 million gallons per acre per day. The estimated cost is $2400 per million gallons capacity. Provision is made for

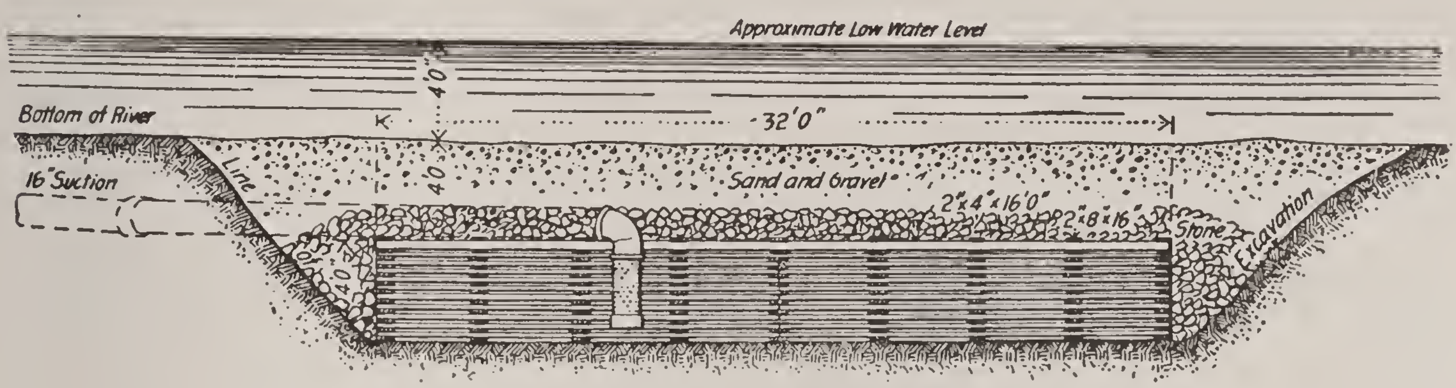

FIG. 65.—FILTER-CRIB, KENSINGTON, PA.
(From *Engineering News*, vol. XXXI.)

cleaning by back-flushing. The system was designed by Mr. James H. Harlow, Mem. Am. Soc. C. E. The cribs are usually placed where the velocity of the current is from 4 to 8 feet per second and little trouble has been experienced through clogging.

Filters of this sort are found to clarify the water at most times quite satisfactorily, but the bacterial content of the water is but little changed. The hardness is likely to be increased.

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CHAPTER XV.

IMPOUNDING-RESERVOIRS.

CAPACITY.

362. Use and Value of Storage.—Whenever the minimum rate of yield of a source of water-supply is less than the demand, the excess of demand over supply may often be furnished by storing up the surplus waters during periods of greater yields. In the case of surface-water supplies the yield is very variable, and a large amount of storage is needed to make available even 50 per cent of the total flow. On the other hand the minimum flow is relatively so small that a comparatively small storage is sufficient to increase many times the capacity of the stream to deliver uniform quantities.

The artificial storage of surface-water in large volumes is usually accomplished by constructing a dam across the valley in question, thus forming an impounding-reservoir. Frequently it will be impossible or uneconomical to store sufficient water in a single reservoir, in which case several may be constructed on the same or different watersheds, thus forming a system of reservoirs. Considerable quantities of water are also often stored in excavated reservoirs of such capacity that they may be called storage-reservoirs. They are similar in construction to the smaller service or distributing reservoirs and will be discussed in Chapter XXVII. They are seldom used for impounding the flow of small streams, but rather for purposes such as sedimentation, storage of river-water to avoid the necessity of pumping during the floods, storage of ground-water to allow of more uniform operation of wells, etc.*

Natural lakes or ponds can frequently be utilized as reservoirs. Their value for storage will depend upon the amount their surfaces can be varied in elevation, and not upon their total capacity.

* It has been proposed to construct storage-reservoirs of this nature on the Thames watershed for the London water-supply, there being few or no good natural reservoir-sites.

363. Factors to be Considered.—In all questions of storage there are three general factors to be considered: (1) the yield of the source for successive intervals of time; (2) the demand for all purposes for like intervals of time; and (3) the storage necessary or available. The problem may be to determine the storage necessary with the first two factors given, or it may be to determine the maximum possible demand from a given watershed when the amount of storage is limited, or it may be to determine the supplying capacity of the watershed for various volumes of storage when comparing the cost of different sources of water-supply.

The yield of the source of supply has been discussed in Chapter VI. The demand to be considered includes not only the consumption for the city in question, but also the loss of water by evaporation from water-surfaces not included in the estimate of the flow of the stream, such as that from the area of the reservoir itself, also loss from leakage and percolation, and often the necessary withdrawals to satisfy the demands of riparian owners below. The reservoir surface may be taken at from 3 to 5 per cent of the total area, but if the assumed area is found later to be materially in error, the computations may be revised. The amount of leakage through the dam will usually be very small, but with certain forms of construction may be large. This question is further discussed in succeeding chapters.

The ultimate loss by percolation will not be large unless the dam is underlain by a porous stratum which will lead the water away underneath. A careful geological examination of the impounding area and of the cross-section at the site of the dam will determine this point. If porous earth overlies impervious strata, the ground-water flow may be made use of to increase the capacity of the stream; and, furthermore, as a reservoir fills, such porous material will become fully saturated and will act to increase the capacity of the given reservoir beyond its apparent capacity. As this increase due to ground-water is difficult to estimate, it should be considered as an additional safeguard and not relied upon except under very favorable circumstances. Cases have been cited where the capacity has thus been increased 20 to 30 per cent.

A fourth factor to be considered is that of the effect of storage upon the quality of the water; and it may be desirable for the sake of the improvement resulting from storage to make the capacity greater than that determined from considerations of quantity alone.

364. Appropriation of Surface-waters.—The quantity of water necessary to satisfy the demands of the riparian owners below the reservoir is often an exceedingly difficult matter to determine, and usually becomes a question for the courts to settle. Practice differs greatly in different States, and in many of the Western States the water belongs to the State to dispose of as it sees fit. It is often expedient to buy up all rights and to utilize whenever necessary the entire flow of a stream, or to fix by contract the amount which will be allowed to flow. When full water compensation is given the amount is determined on the general principle that only that portion of the flow can be abstracted that is not ordinarily used by the riparian owners for legitimate purposes, such as for water-power, domestic uses, etc. This would usually mean that during the dry months all of the natural flow must be allowed to pass, and during the remainder of the year a uniform quantity equal in value for the uses in question to the former flow of the stream. The amount in any case would thus depend much on local circumstances. Many court decisions in this country have awarded damages for diverting flood-waters that were entirely useless to the riparian owners. In England a volume equal to one-third the total flow has often been adopted as compensation. In any case the most of the flood-flow would be available, and this usually amounts to considerably more than half the total flow.

365. Computation of Storage.—The method of computation described below is essentially that of Rippl.* It consists in graphically representing the net yield of the source in question during dry periods, and in obtaining from the resulting curve the solutions to the various forms of the problem which may be presented.

The method is as follows: From the measured or estimated flow of the stream for each month during the period to be treated, subtract the monthly loss by evaporation from water-surfaces not included in the estimate, the monthly loss by leakage, and the monthly compensation as previously determined. The result will be the net yield for each month. Add together the yields from the beginning to each month in succession; then from these figures construct a curve OA, Fig. 66, in which the abscissa of any point is the total time from the beginning of the selected period, and the ordinate is the total net flow during the time represented by the abscissa. The inclination of the curve at any point is thus equal to the rate of the net flow, a minus inclination, as at B, representing a negative flow. Now in like manner plot a curve of consumption, OC, which may ordinarily be assumed as a straight line, as the variation month by month is a refinement hardly warranted by the accuracy of the other data.

* Proc. Inst. C. E., LXXI. p. 270.

The ordinates between the lines OA and OC will now represent the total surplus from the beginning, and where the two lines converge, as at B and D, the yield is less than the demand, and conversely. The greatest deficiency occurring during any dry period B is found by drawing EF parallel to OC and tangent to the curve; and the amount of it is given by the maximum ordinate IG drawn from EF to

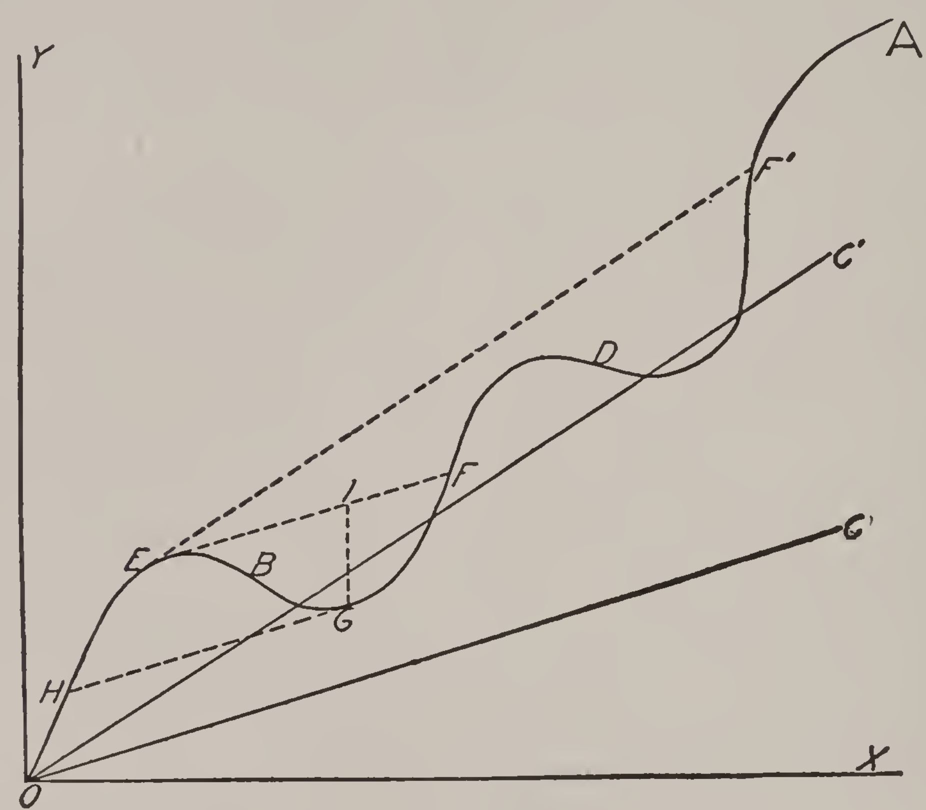

Fig. 66.

the curve. The deficiency for any other dry period is likewise found, and the maximum so found is the storage volume required to supply the demand OC. The time during which the reservoir would be drawn down below high-water line would be represented by the horizontal distance between E and the point of intersection F. In like manner the storage capacity for any other rate, OC', may be determined by measuring from the tangent EF'.

If the tangent from any summit does not intersect the curve, it indicates that during the period investigated the supply is not equal to the demand; and to insure a full reservoir at the point E, for example, it is necessary for the parallel tangent drawn backwards from G to intersect the curve at some point H. In investigating various dry periods it is therefore necessary to begin the curve a year or two back of the dry years to insure the accumulation of surplus water. When actual stream measurements are to be had covering a series of years, it is best to consider the entire period.

If the yield is to be limited by the time during which the reservoir is to be drawn below high-water line, the rate of supply and corresponding storage can be determined by finding by trial the line of lowest slope which shall be tangent at a summit, and whose horizontal projection equals the time specified. If the storage is fixed and it is desirable to know what amount of water the area will yield at a constant rate per month, the rate is found by drawing the lines *EF* from various summits, which shall have their maximum ordinates, *GI*, equal to the given storage. The rate is given by the line of least slope.

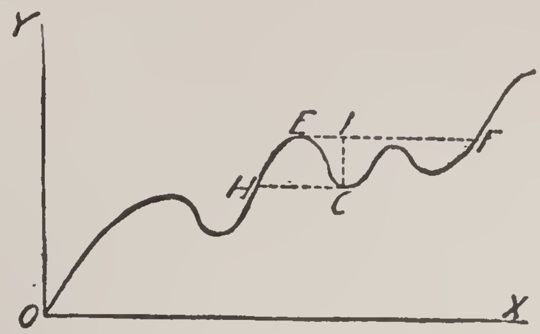

FIG. 67.

If the case is one where the consumption cannot be assumed as uniform, the line *OC* will be a curve, and the desired information can be more easily got by replotting the ordinates between the demand and supply curves—the accumulated surplus—from a horizontal axis, as in Fig. 67. Storage volumes, etc., are then found by drawing the tangent lines *EF*, etc., horizontally.

366. Storage Calculation from the Sudbury River Records.—The results of calculations of storage volumes based on the records of the flow of the Sudbury River watershed are given in Table No. 58. The data are from a more extended table by Mr. FitzGerald.* The Sudbury watershed has $3\frac{1}{3}$ per cent of water-surface, and the observed flow is reduced to monthly flow from one square mile of land-surface by correcting for evaporation from the water-surface. Then from these figures, and the yield from one square mile of water-surface as given by the difference between rainfall and evaporation, calculations are made of the yield of one square mile having various percentages of water-surface. These results are then plotted and the storage volumes for various rates of consumption determined in a way similar to that explained in the preceding article. Mr. FitzGerald estimated the evaporation for each month throughout the entire period, but nearly the same results would be obtained by using the mean monthly evaporations as given on page 56.

Table No. 17, page 84, gives the data of stream-flow covering the most important part of the record, and from these figures and the mean monthly evaporations the student should be able to obtain storage volumes closely agreeing with those of the table up to a daily draught of 600,000 to 800,000 gallons, depending upon the percentage of water-surface. Beyond this the draught is greater than the average flow for the five years given, and a longer period would need to be considered.

* Trans. Am. Soc. C. E., 1892, XXVII. pp. 267, 268.

TABLE NO. 58.

STORAGE CAPACITY REQUIRED FOR VARIOUS DAILY DRAFTS FROM ONE SQUARE MILE OF WATERSHED CONTAINING VARIOUS PERCENTAGES OF WATER-SURFACE BASED ON SUDBURY RIVER RECORDS.

Constant Daily Draft.	0 per cent.		10 per cent.		25 per cent.	
	Storage Volume per Sq. Mile.	Length of Time Reservoir is Below High-water.	Storage Volume per Sq. Mile.	Length of Time Reservoir is Below High-water.	Storage Volume per Sq. Mile.	Length of Time Reservoir is Below High-water.
Gallons.	Gallons.	Months.	Gallons.	Months.	Gallons.	Months.
100,000	314,000	1½	15,012,000	6½	53,565,000	7½
150,000	3,006,000	3½	19,642,000	7½	59,665,000	8½
200,000	8,797,000	7½	25,742,000	7½	65,765,000	8½
250,000	17,997,000	7½	33,338,000	8½	71,865,000	8½
300,000	28,473,000	8½	43,437,000	8½	78,807,000	8½
350,000	39,173,000	9½	54,137,000	9½	87,957,000	9½
400,000	51,303,000	9½	66,050,000	10½	99,089,000	10½
450,000	63,553,000	9½	78,300,000	10½	127,412,000	21½
500,000	75,803,000	9½	90,550,000	10½	156,362,000	21½
550,000	88,053,000	9½	105,987,000	21½	185,312,000	22½
600,000	100,651,000	10½	134,937,000	21½	214,262,000	22½
650,000	114,451,000	10½	163,887,000	21½	250,744,000	92½
700,000	139,950,000	21½	192,837,000	22½	336,044,000	106½
750,000	168,900,000	21¼	221,787,000	23½	421,344,000	115½
800,000	199,106,000	58½	297,460,000	92½	506,644,000	125½
850,000	250,328,000	80½	380,557,000	107½	591,944,000	141½
900,000	334,078,000	93½	465,857,000	116½	677,244,000	165*

* Estimated.

This table of storage volumes is considered a safe one to use for streams in New England. A similar table can be constructed for any locality where accurate data are at hand, and in such form the information can readily be used in estimating yields and storage volumes for other areas in the vicinity and for various parts of the same watershed.

As an example of the use of the table let it be required to determine the necessary storage-capacity to supply a constant daily draught of 7 million gallons from a watershed of 12 square miles having 10 per cent of water-surface. The draught from one square mile will be equal to 583,000 gallons, and by interpolation the reservoir capacity is about 125 million gallons per square mile, or a total capacity of 1500 million gallons. It will be below high water for a period of 1 year 9½ months.

367. **Capacity of a System of Reservoirs.**—If there are several reservoirs on the same watershed, the supplying capacity of any combination may be found by the use of the methods just described. Beginning with the reservoir farthest up the valley, the net flow is determined and plotted, from which the maximum possible average

draught is found. Whenever the supply exceeds this draught with full reservoir the excess passes to the next reservoir below and adds to the supply from its own tributary area. Its supply curve can now be drawn and capacity determined, and so on.

LOCATION AND CONSTRUCTION.

368. Considerations Affecting Location.—The proper location of an impounding-reservoir requires the consideration of many elements. In the first place the location determines the size of the tributary watershed, and, as the capacity is directly dependent upon the size of the watershed, different locations will call for different capacities to furnish like quantities of water.

The location is also very largely determined by the distance of the reservoir from, and elevation above, the point of distribution. Long distances require heavy expenditures for conduits or pipe-lines, but these expenditures are relatively less the larger the quantity of water dealt with. For large cities it will therefore be practicable to go much farther for water than for small cities. Regarding the elevation of the reservoir it is very desirable that it shall be sufficient to enable all or a part of the consumers to be served by gravity alone, and it will be economy to spend a relatively large sum of money for conduits or otherwise to secure this advantage. The necessary elevation for this purpose depends upon the required pressure at and elevation of the various points of distribution, and the head lost in conducting thence the water. These features are discussed subsequently and as separate problems, but in the actual case they are all interdependent and must be considered together. Distant locations at high elevations will often need to be compared in economy with near locations requiring the use of pumps.

The question of future extension is an important one, especially in large works, and the selection of a certain watershed for a supply may be determined not so much by the location of a single reservoir as by the existence of sites for several reservoirs in order that the capacity of the watershed may in time be developed to its fullest extent as the demand for water increases.*

In questions pertaining to cost the economy of a reservoir alone is

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*For important examples see Report Mass. Board of Health upon a Metropolitan Water-supply, 1895; Wegmann, The Water-supply of New York, 1896; Report upon Future Extension of the Water-supply of Brooklyn, 1896; Freeman's Report on New York's Water-supply, 1900.

measured by the cost per million gallons stored, but a more significant quantity is the cost per million gallons of daily supplying capacity of reservoir and watershed; or in case the reservoir is one of a system, the cost per million gallons supplying capacity added by the reservoir in question.

369. *Topographic and Geologic Features.*—The most favorable location for a reservoir as regards topography is a point where the valley forms a comparatively broad level area bounded by steep slopes at the sides, and below which the hills approach close together so as to form a good site for a dam. Such an ideal site is rarely found, and it will usually be necessary to compare two or more possible sites, for which purpose careful estimates of cost will be required. A site which will include much swampy area or involve a large amount of shallow flowage is objectionable on the grounds of quality.

To prevent the escape of water the floor of the reservoir should contain no outcrop of porous strata of any extent which may lead the water away underground, and in the vicinity of the dam or embankment it should be underlain by an impervious stratum at a depth that can be reached by that structure. In some cases these conditions cannot be secured and some loss through porous ground must be expected.

Besides the character of the substrata in the vicinity of the dam, the kind of soil, proximity of suitable stone for a masonry dam or of material for an earth embankment, are questions controlling to a large extent the location of a reservoir.

370. **Surveys and Preliminary Work.**—To make even a preliminary determination of reservoir-site in accordance with the preceding principles requires a fairly accurate knowledge of the areas of various portions of the watersheds and of their relative elevations. If this information cannot be obtained from existing maps, a reconnoissance survey must be made. For this purpose the transit and stadia method is well adapted.

After a tentative location has been decided upon, accurate levels must be run to connect the town with the reservoir-site, also surveys for conduit lines, and an accurate topographical survey of the area to be flooded and all that may be affected by the reservoir. This survey should include information as to all buildings upon and adjacent to the area in question, nature of the vegetation, location of roads, property lines, etc. In addition to this it will prove of much subsequent value to have a topographical survey made of the entire watershed, which may be less accurate than that for the reservoir. At the same time a complete sanitary survey of the watershed can be made, as outlined in Chapter VIII, Art. 154, and a good topographical map will prove of great convenience in this connection.

Determinations should be made of the character of the soil, amount of organic matter at various depths, especially in swamps or old ponds, and nature of substrata with reference to its permeability. At the site proposed for the dam numerous borings must be made extending to a considerable distance above and below the dam as well as on the flanks, and these must be supplemented by test-pits so that the nature of the supposed firm stratum can be accurately determined. If a suitable foundation cannot be reached at a reasonable cost, the site may have to be abandoned.

371. Depth of Reservoir.—Calculations of storage volumes for different depths can readily be made from the contour map. The areas enclosed by each contour can be measured by a planimeter and the volume between any two successive contours taken as equal to the average of the areas enclosed by the contours, multiplied by the contour interval. Where the slopes are very flat this method gives an appreciable error, and in that case it may be advisable to employ the prismoidal formula. By this formula, the volume of two successive slices in terms of the three areas a, b, and c (the two end and the intermediate areas) is equal to $(a + 4b + c)\frac{d}{3}$, where d is the contour interval. The volume up to any given contour having been determined, the necessary height of dam to hold any given quantity of water becomes known.

From considerations of quality, it should never become necessary to withdraw the water to the very bottom of a reservoir, so that the volume for a few feet in depth at the bottom should be omitted from the calculations. What this depth should be depends upon the character of the water and the shape of the basin. It may be taken, with very little loss of capacity, at one-fifth or even one-fourth the total height of the dam. With sediment-bearing streams some allowance should be made for the silting up of the reservoir, the amount of which can be estimated from analyses of the water. In the case of some streams this is a matter of importance and may involve considerable expense for the removal of the sediment from time to time.

A noteworthy case of the rapid silting up of a reservoir is that of the reservoir formed by the dam across the Colorado River at Austin, Texas. This dam was completed in 1893, and at that time formed a reservoir of a capacity of 17,000 million gallons, whereas in February, 1900, the capacity had become reduced to 8600 million, or only 52 per cent of the original amount. The cause of this large amount of deposit was the very large discharge of a silt-bearing stream as compared to the capacity of the reservoir.*

Where the volume is not fixed, as in the case of a series of reservoirs, the economical height is determined by comparative estimates of cost for various volumes. Such estimates must of course include expense for land, water rights, etc., as well as for the constructive features.

372. Cleaning the Site.—In Chapter IX the injurious effect upon the quality of the water of organic matter in reservoirs was discussed, and the necessity for the removal of all vegetation and perishable matter from the reservoir-site was pointed out. Still further it has been shown to be desirable and of great benefit to the water to remove the top soil to a sufficient depth to include most of the organic matter therein. Such stripping has for some years been done for some of the large reservoirs of the Boston Water-works and at other places, especially in Massachusetts, with the result that the impounded water from the first has suffered no deterioration by storage. In other reservoirs where this has not been done, trouble has been experienced for many years. Where the deposit of organic matter is very deep, and the expense of removing too great, a covering of gravel is advisable.

In investigating the soil from the site of the proposed Nashua reservoir, Dr. T. M. Drown of the Mass. Board of Health found that the amount of organic matter in a soil decreases rapidly with the depth. The percentage at the surface was usually 8 per cent or more, while at 10 or 12 inches below there was usually but $1\frac{1}{2}$ to 2 per cent. As a result of this study he suggests these lower figures as a provisional standard for the permissible percentage of organic matter which may be allowed to remain.†

As an example of soil-stripping on a large scale mention may be made of the work done on Reservoir No. 5 of the Boston Water-works.‡ The soil was in general removed to a depth of about 1 foot, but in places to a much greater depth. In one pocket of mud 20 feet deep the amount of organic matter at the surface was 75 per cent, and at 10 feet deep, the depth of the excavation, it was 5 per cent. The total amount removed in this way was about 4 million cubic yards, adding in this way about 10 per cent to the capacity of the reservoir. In the proposed Nashua reservoir the cost of such removal is estimated at nearly $3,000,000.

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* *Eng. News*, 1900, XLIII. p. 135.

† Report Mass. Board of Health, 1893, p. 383.

‡ *Eng. News*, 1897, XXVII. p. 130.

373. Shallow Flowage.—As a further protection to the quality of stored water it is desirable that there be as little area alternately flooded and exposed as possible, in order to limit the growth of vegetation. Here again the practice of the Boston Water Board is to be recommended. The minimum depth at high water allowed in Reservoir No. 5 is 8 feet. Shallow places are either excavated to this depth below high-water line, or are partly excavated and partly filled, the slopes being formed at 3 to 1 and covered with 2 feet of gravel.

374. Maintenance.—In maintaining a reservoir so as to preserve the quality of the water and to supply the necessary quantity regularly and certainly requires a considerable degree of care and attention. To keep the quality as good as possible requires first of all that the watershed and reservoir be kept free from organic pollution. To insure that this is the case the city should have sanitary supervision over the area in question, and inspection should be regularly made to see that all sanitary requirements are complied with.*

In addition to the prevention of pollution from animal sources it is desirable that the reservoir be kept as free as possible from vegetation. During seasons of low water, opportunity is offered for removing the vegetation from around the borders of the reservoir. Where there is more or less unavoidable organic matter present in the water there will at times be objectionable tastes and odors at certain depths. To obtain the best water available the depth at which it is drawn from the reservoir should be varied from time to time according to the condition of the water. Frequent analyses of water drawn from different depths are very valuable in this connection.

When reservoirs have become silted up to a considerable extent it may be necessary to remove the deposit. If the dam be located in a narrow valley, much can be removed by flushing through sluice-gates. The greater part of the material will, however, have to be taken out by methods similar to those used in ordinary excavations. In case the silt brought down contains much organic matter it should be removed frequently in order to prevent trouble from its decay.

Careful records should be kept at the reservoir of all matters which may be of any value in subsequent designs for enlargement or for new works. These should include records of rainfall, temperature, height of water in reservoir, amount passing over waste-weir, and data pertaining to the quality of the water at different seasons of the year.

The maintenance of dams and embankments should call for very

* For details of methods of making such inspection and the requirements which have been imposed in certain instances see Reports N. Y. State Board of Health, 1889, 1894. See also Literature of Chap. VIII.

little labor. Earthen embankments should be kept neat in appearance with slopes well sodded, or covered with large gravel so as to be permanent. The top of the embankment should of course be maintained at its full height, and the waste-weir and the channel below it kept clear and of the designed capacity at all times. Gates and other apparatus should be frequently inspected and kept in thorough repair. Flashboards should be used with great caution, if at all. They should always be so designed that they will fall or be washed away when the water begins to flow over them. Immediate attention should be given to any sign of increased leakage in the case of either an earthen or a masonry dam. Leaks or excessive seepage in masonry dams may be often stopped by plastering the up-stream face of the dam with rich cement mortar. Any visible cracks may also be filled with Portland-cement grout forced in under pressure. Earthen dams are repaired with difficulty. If the seepage-water flows perfectly clear, the indications are that no material is being carried away and that there is no immediate danger of the leak enlarging. If the seepage-water, on the other hand, be muddy and continue so, the water should be drawn off at once and the dam repaired. This may sometimes be accomplished by excavating to a moderate depth at the upper end of the leak and filling with puddle well rammed into place. If the leak is a serious one, it will probably be necessary to cut down from the top and fill with good material well bonded into the old part of the work, and compacted in the same manner as for a new embankment.

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CHAPTER XVI.

EARTHEN DAMS.

GENERAL CONSIDERATIONS.

375. The Requisites of a Dam.—The function of a dam is to prevent the passage of water. To this end it must be impervious, or sufficiently so to prevent excessive loss of water; and it must be stable. The possible consequences of a defect in the latter requirement are exceedingly serious, as has been demonstrated by the great loss of life and property caused by the failure of reservoir embankments in the past.

These two properties, imperviousness and stability, may or may not be independent of each other, according to the nature of the construction; but it will in any case be of advantage to consider them as distinct and separate. In other words, a dam must have an impervious body, and this must be safely supported.

376. Kinds of Dams.—Dams may be divided according to the material used into five classes: earthen dams, masonry dams, loose-rock dams, wooden dams, and iron or steel dams. These materials are also used in various combinations. The form of dam suitable for a given case depends upon the character of the foundation, the topography of the site, the size and importance of the structure, the degree of imperviousness required, and the cost. Of the above kinds of dams those of masonry and of earth are the ones usually considered and will here be fully treated. The other forms will be treated as fully as their importance seems to warrant.

377. The Dam as a Porous Structure.—Before proceeding with the discussion of the various forms it will be well to consider the action of the dam as a porous structure. Few dams are absolutely impervious and many are far from it, yet they may do good duty as impounders of water; but whether a dam is impervious or not is a matter that in most cases greatly affects the stability and should be carefully considered.

Let *ABC*, Fig. 68, be a body of porous material (whether very porous, as sand or loose rock, or only slightly so, as most masonry dams, is for the present immaterial), resting upon a porous foundation and retaining water at the level *D*. Under the assumed conditions some water will percolate through and under the dam and escape at *C* in a way similar to the flow of ground-water. The surface of the percolating water will be some curve, such as *DEC*, the slope of which at any point measures the relative resistance to flow. The amount of water percolating varies of course with the porosity, but even with fine sand or earth it will be very small, so that the requirement as to imperviousness may be met with a porous material.

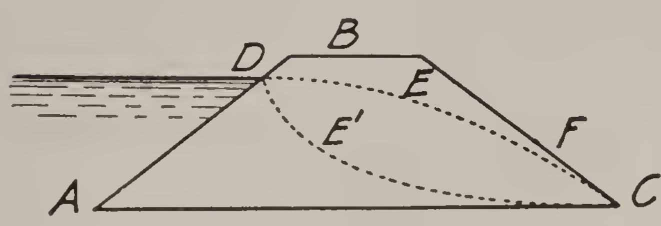

Fig. 68.

The requirements for stability depend somewhat upon the nature of the material employed. If a rigid material is used, such as masonry, the dam must be so proportioned as to resist sliding upon its foundations, it must not be overturned, and it must not be ruptured or overstrained in any of its parts. If a material such as earth, sand, or loose rock be used the foregoing requirements must be met so far as they are applicable to such materials, and in addition the important requirement that percolation must be controlled in such a manner as to prevent any material being carried away by the water. The effect of percolating water upon the character of the material must also be carefully considered.

As the stability of any form of dam is largely dependent upon the weight of the material it is important to inquire what effect any percolating water will have upon this factor.

378. *The Buoyant Effect of Percolating Water.* — If the material consists of loose grains like sand or earth, the loss of weight for the portion submerged is equal to the weight of the water displaced. If p is the ratio of porosity by volume, the volume of water displaced for each cubic foot of the material is $1 - p$, the weight of which is $(1 - p)\ 62.5$ pounds. Thus with a porosity of 40 per cent the loss in weight of sand, weighing, say, 100 pounds, will be $(1 - .40)\ 62.5 = 37$ pounds per cubic foot, leaving a net weight of 63 pounds.

If the material is cohesive like stone or cement, the water cannot exert an upward pressure upon the entire surface of any horizontal section, and the buoyant effect will be much less than the above. Assuming that the full effect will occur whenever the porosity is as great as $33\frac{1}{3}$ per cent (this being approximately the porosity of well-compacted sand), and that with a cohesive material of less porosity it will be proportional to the porosity, the buoyant effect in such cases will be

$\frac{p}{\frac{1}{3}}(1-p)\,62.5 = 3p\,(1-p)\,62.5$ pounds per cubic foot; and the upward pressure on any vertical column of masonry of 1 square foot in cross-section will be equal to $3p\,(1-p)\,62.5 \times h$, where h = distance the column is submerged below the water surface DEC (Fig. 68). Thus in a homogeneous masonry dam having a porosity of 5 per cent, the buoyant effect on the submerged portion will be approximately $3 \times .05\,(1 - .05)\,62.5 = 9$ pounds per cubic foot. That hydrostatic pressure may be readily transmitted through porous stone has been fully shown by experiments and would in any case hardly admit of doubt.*

The following table gives the buoyant effect in pounds per cubic foot for homogeneous masonry of various porosities, calculated in accordance with the preceding analysis.

Porosity, per cent.	Buoyant Effect, Lbs. per cu. ft.	Porosity, per cent.	Buoyant Effect, Lbs. per cu. ft.
1	1.9	6	10.6
2	3.7	7	12.2
3	5.5	8	13.8
4	7.2	9	15.3
5	8.9	10	16.8

In a structure of stone having a small porosity p, with mortar joints of large porosity p', the buoyant effect at the joints will be large and will be equal to $3p'\,(1-p)\,62.5 \times h$. If any loose joint exists, or any place where the water can enter in "thin sheets," then p' becomes the same as assumed for loose material, namely, $33\frac{1}{3}$ per cent, and the buoyant effect is again equal to $(1-p)\,62.5 \times h$, as for sand. To reduce this effect as far as possible in masonry it is thus seen to be necessary to make the joints of as little porosity as the stone itself. If the foundation be more porous than the masonry, and open to the percolation of water, the maximum buoyant effect will be at the bottom and measured by the porosity of the foundation.

379. Influences Affecting the Depth and Amount of Percolating Water.— From the discussion in Chapter VII relative to the flow of ground-water it is evident that the quantity of water percolating through a dam depends in general upon the thickness of the dam and the fineness of the material of which it is composed. The *depth* of this percolating water, or the form of the curve DEC (Fig. 68), depends primarily upon the uniformity of the material and not upon its fineness or the quantity of water passing. Since the weight and stability of a dam are decreased by this percolating water it is evidently of advantage

* See paper by Ross and Broenniman on the Hydrostatic Pressure in Masonry. *Jour. West. Soc. Engrs.*, 1897, II. p. 449.

either to prevent percolation altogether or to lower the water level DEC as much as practicable. In the case of low earthen dams it is important to keep this line low and also to reduce the amount of percolation to a small quantity, as any considerable amount of percolating water appearing along the lower face BC is likely to affect seriously the stability of the material in this part of the dam.

Now the slope of the curve DEC at any point measures the relative resistance to flow of the percolating water, hence anything which increases this resistance tends to increase the slope of the curve. An increased resistance near the up-stream side will thus cause the curve to take some such form as $DE'C$. This result is accomplished in various ways. In an earthen dam the material near the up-stream side may be made more impervious than that in the lower part of the dam, while in a masonry dam the upper face may be plastered or otherwise made relatively impervious. The drainage of the lower portion of the dam in the case of either an earthen or masonry structure, is another means for lowering the water-level and at the same time taking care of the percolating water. Where porous material must be used the amount of percolation is kept within safe limits by making the dam of great width. Core walls, if made relatively impervious (see Art. 385) will serve to lower the water-level in the material below the wall, but if the wall is not more impervious than the earth filling below then it will have little influence in this respect.

In Fig. 68 it has been assumed that the foundation is somewhat porous as well as the dam. In that case the percolating water, if not large in amount, may pass down stream entirely below the surface and give no trouble; otherwise it will come to the surface near the toe of the dam in the face BC or will appear below C as springs. If the foundation be entirely impervious, then any percolating water must appear along the face BC, the ground-water level intersecting the face BC at some point F, depending upon the nature of the construction as above explained. In either case special care must be exercised in the construction, as explained more fully in Art. 400.

Some very instructive observations were made in 1901 on several of the earthen dams of the New York Water Supply relative to the water level therein, by sinking small drive pipes at various points in the bank. Fig. 68a shows the results in the case of four of the dams tested. The dams are all built with core walls and the effect of such walls on the water level, as shown in these examinations, is of much interest. In the figure the water level in each test well is shown and between wells it is assumed to be a straight line. In the Titicus dam and the Carmel Auxiliary dam the water level falls abruptly at the core wall, but in the other dams the wall seems to have little effect upon the water surface. In the former case the wall is evidently more impervious than the earth below, while in the latter case it is not. Whether this is due to a more porous filling it is impossible to say. For maximum stability the condition of the first two dams is the more favorable. In the Amowalk dam the test wells Nos. 13 and 14 indicate that the percolating water flows away below the surface, while in the

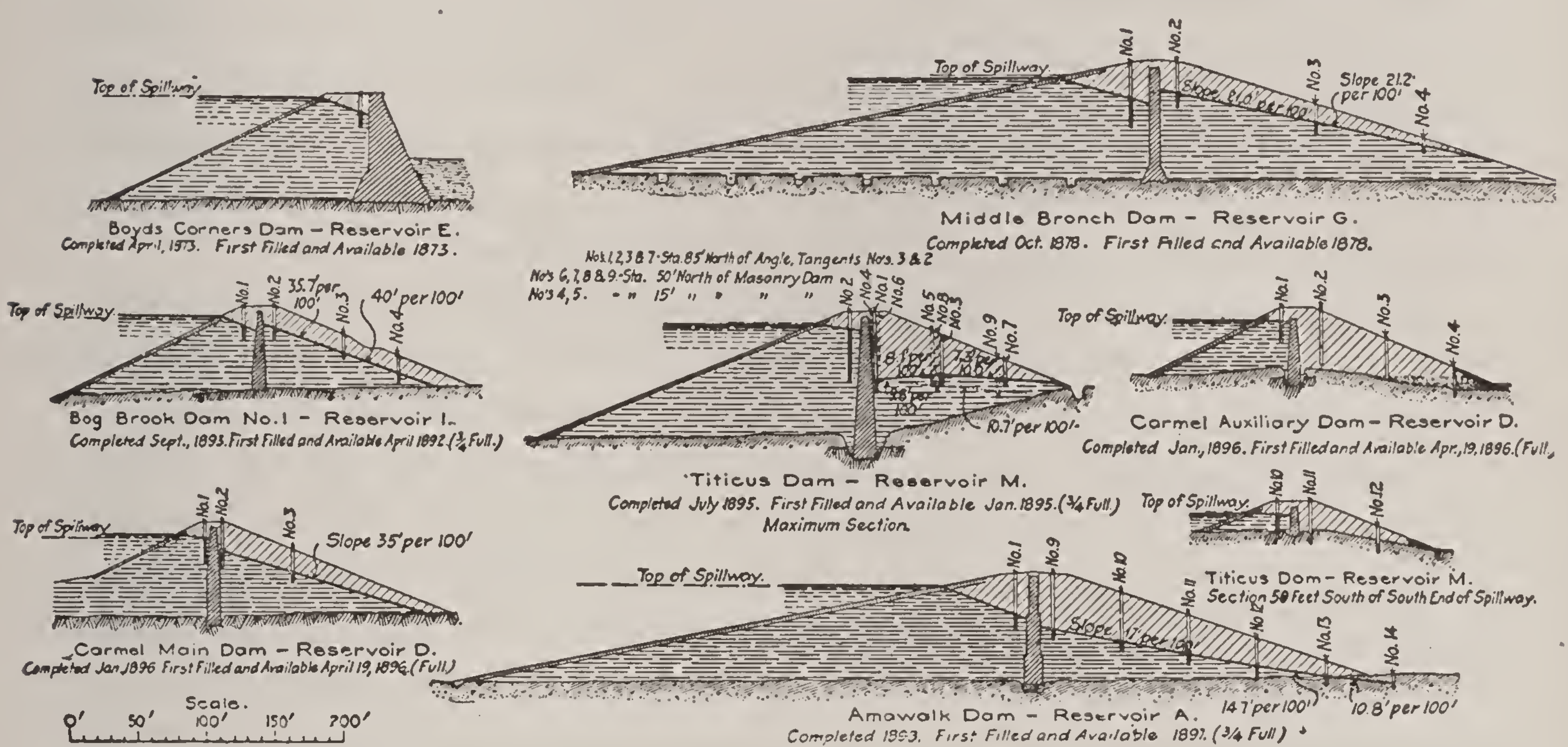

FIG. 68a. — SECTIONS OF DAMS IN CROTON VALLEY SHOWING GROUND-WATER LEVEL. (From *Engineering Record*, Vol. XLIV.)

Middle Branch dam it appears to come to the surface along the face of the bank. The difference in condition would seem to be due to a difference in amount of percolation or to different porosities of the underlying material. Evidently by the use of suitable material, or by proper drainage, the water level could be brought down as in the Amowalk or the Titicus dam.*

380. Advantages and Requisite Conditions. — The earthen embankment is the most common form of dam. It can be built on a variety of foundations; it is commonly the cheapest form, and when well designed and executed is an entirely safe and reliable structure. The stability of an earthen dam is, however, so closely dependent upon its imperviousness that, compared to some other forms of dams in which

* See report of Engineers on Changes in the New Croton Dam, *Eng. Record*, 1901, XLIV. p. 520; *Eng. News*, 1901, XLVI. p. 410. Also discussion in *Eng. News*, 1901, XLVI. p. 454, and 1902, XLVII. p. 33; and Trans. Am. Soc. C. E., 1906, LVI. p. 32.

these functions are more independent, the necessity for making the dam impervious is relatively great. The properties of the materials used are also less uniform and less well known than those of other materials, so that a very large margin of safety must be used.

Where flood-waters have to be passed over a dam some other material than earth must be used for at least the portion of the structure subjected to water action. Water flowing over an earthen embankment is inadmissible, many failures having been caused by such occurrence. If a suitable foundation can be had, masonry is to be preferred for high dams. It is more reliable, and where great pressures occur it furnishes a safer design for the outlet pipes. Several successful earthen dams have, however, been built of a height exceeding 100 feet. Many high dams have been constructed partly of earth and partly of masonry, the higher central portions being of the latter material.

381. The general requirements of a good foundation for an earthen dam are that an impervious stratum can be reached at a moderate depth, and that the material near the surface is sufficiently compact to support the load. A compact clay or hardpan makes the best foundation. Solid rock is also good if not fissured. Mere porosity is comparatively unobjectionable, but a rock through which water is liable to pass in cracks and fissures makes a very bad foundation for an earthen dam. Embankments of earth have been successfully constructed on foundations of sand; but in such a case it is important that the sand be fine and of a uniform character, containing no streaks of coarse material which will offer little resistance to the flow of water. Soft foundations have also been built upon in cases of necessity, but both porous and soft materials should be avoided if possible.

382. **Forms of Construction.** — Earthen dams are of a trapezoidal form with top width, side slopes, etc., proportioned according to the material used. Several types of embankments are employed, the one used in any case depending upon the material at hand and upon the individual preference of the engineer.

1. *The Homogeneous Embankment.* — Where good material is at hand in sufficient quantities the entire embankment may be made of uniform consistency and all as nearly water-tight as possible. Usually, however, it will be more economical and give as good results to put the best material near the upper side of the embankment, changing gradually to the more porous material towards the lower face.

2. *Embankments with Core-walls.* — Where good material is scarce, imperviousness is usually obtained by means of a wall of impervious earth or masonry placed near the center of the dam. If imper-

vious foundation is reached only at a considerable depth, this portion only of the embankment is carried to the extreme depth.

3. *Porous Embankments or Embankments on Porous Foundations* are sometimes necessary from lack of suitable material; they require special precautions to insure their stability.

383. **Stability of the Various Forms of Embankments.**—The chief danger of failure of an earthen embankment lies in its destruction through percolation or in being overtopped by floods. It is, however, desirable to consider also the stability against sliding on the base, and in some localities it is necessary to make the design with reference to the possibility of overloading the foundation stratum. The matter of soft foundations will be treated later, but the questions of imperviousness and frictional stability will be considered here.

The conditions would rarely be such as to make it likely that a dam could fail by actually sliding as a whole on its base. Lack of stability to resist water-pressure would be shown rather by slips of portions of the embankment at the outer slope. The stability in this respect is, however, approximately indicated by the frictional stability at the base and at other horizontal sections.

384. *The Embankment Without Core-wall.*—In case the embankment is impervious, either when made entirely of impervious material or when only the upper portion is so made, the internal water-pressure is zero, as it is assumed that no water percolates. The forces acting will then be as shown in Fig. 69. The force tending to cause sliding along the base is $P \sin \alpha$, and the maximum resistance would be Vc, where $c =$ coefficient of friction of the material. Here V is large, being equal to $W + P \cos \alpha$, hence the factor of safety against sliding is in this form relatively large and equal to $\dfrac{Vc}{P \sin \alpha}$. If imperviousness is not perfectly secured in a homogeneous dam, water-pressure will exist within it which will reduce the effective weight of the material, as explained in Art. 378. It was also there shown that to avoid this as much as possible the upper portion should be more impervious than the lower portion. In this way great stability can be secured with porous materials.

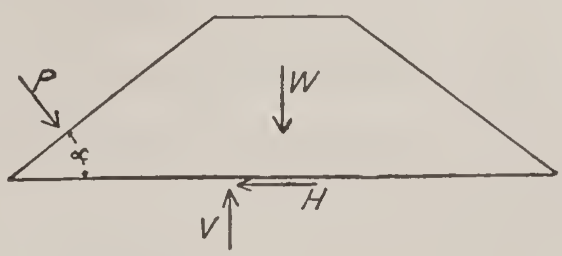

FIG. 69.

385. *The Embankment with Core-wall.* — In this form reliance is chiefly placed on the core for imperviousness. If it is placed at the centre, as in Fig. 70, and is more impervious than the material above it,

the line of pressure, AB, on the up-stream side must be assumed nearly horizontal. If the material below the core be relatively porous, as it should be, then there will be no water in the lower portion of the dam. The water-pressure will then be applied practically horizontally on the core-wall, and dependence for stability against sliding must be placed on the wall and material below. In Fig. 71 is shown the part of the embankment below the upper face of the core-wall. Let h = depth of water; bh = width of this portion of the dam at water-level; s = slope of outside face in terms of ratio of horizontal to vertical projection; w = weight of a unit volume of water; w' = weight of a unit volume

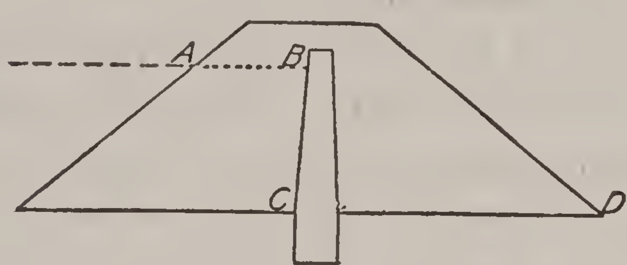

FIG. 70.

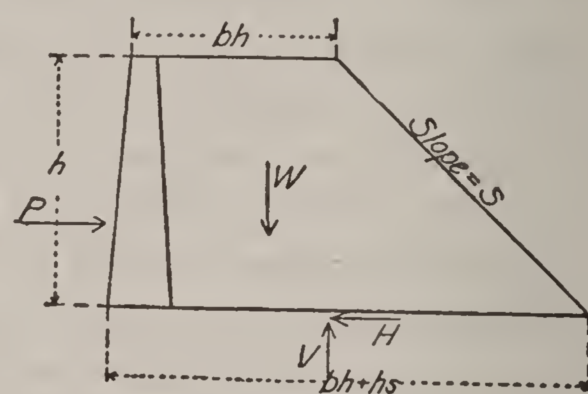

FIG. 71.

of dry embankment material; and W = weight of a slice of embankment one unit long. The pressure of the water will be $P = \frac{wh^2}{2}$. The weight of embankment $= W = w'h\left(bh + \frac{hs}{2}\right)$. If c = coefficient of friction, the factor of safety against sliding is

$$\frac{w'h\left(bh + \frac{hs}{2}\right)c}{\frac{wh^2}{2}} = \frac{w'}{w} \times c(s + 2b).$$

If the slope s is made as steep as the material will stand, then $c = \frac{1}{s}$ and the factor of safety becomes equal to

$$\frac{w'}{w}(1 + 2bc).$$

If $w' = 100$ pounds per cubic foot, and $w = 62.5$ pounds, then to secure a factor of two would require b to be equal to $\frac{.125}{c}$. If $c = \frac{1}{2}(s = 2:1)$, this would give a value of b equal to .25; that is, the width at the water-line must equal $.25h$. Usually the width is greater than this. To further increase the stability the outside slope should be made greater than $\frac{1}{c}$. It should thus be made about 2 to 1 if the material will just stand at $1\frac{1}{2}$ to 1 ($c = \frac{2}{3}$). With $s = 2$ and $c = \frac{2}{3}$, a factor of safety of 2 would be secured with $b = 0$.

In this discussion the possible pressure of water-soaked material upon the core-wall has been neglected. What this would be is impossible to say, but before sliding could take place it would also have a downward component against the wall, thus adding to the friction. The above analysis is, however, sufficient to show the desirability of rather flat slopes on the down-stream side, a considerable width at the water-line, and, in order to secure the full benefit of weight, the lower half of the embankment should be relatively porous and heavy. It is also plain that the weakest section is at the bottom.

386. Material for Embankments.—Various kinds of material can be used to make an embankment. Loam, sand, gravel, and clay, mixed in various proportions, are common. For the first three to be impervious they must contain a certain proportion of clay, the amount required depending upon the variation in size of the coarser particles. The suitability of a material for embankment construction can to some extent be determined by experiments. It should be strongly cohesive and plastic when mixed with water, and should be impervious; but the correct valuation of natural mixtures requires much experience in their actual use in construction. If sufficient impervious material is not available to form the entire embankment, the best is to be selected for this purpose and confined to the upper portion or to a puddle core. For the lower portion, coarse heavy material is suitable.

Considerable difference of opinion exists among engineers as to the best material for embankments or core-walls. English practice favors a puddle of clay with little or no gravel, while most American engineers favor a gravel and clay puddle. Impervious cores or embankments can be made of either material, and where fully protected from wash, and from becoming dried out, are equally satisfactory. Clay shrinks greatly in drying, thus forming cracks, and a pure clay will shrink much more than a mixture containing only 15 or 20 per cent of clay, an advantage in favor of the mixed material. The latter is also much the safer against wash in case of leaks, and is more suitable for the main body of the embankment and for use in exposed situations. It is also less easily attacked by woodchucks, muskrats, etc. Clay dissolves and washes away very easily on account of the minute size of the particles. It is therefore very essential to the stability of a clay wall that there be no percolation through it. For confined locations and in thin sections, a clay containing only coarse sharp sand is probably better than one with gravel.

If good material does not exist already mixed, artificial mixtures of gravel, sand, and clay may be used. A fairly uniform sand or gravel contains about 40 per cent of porous space. If then a mixture be made of coarse gravel, fine gravel, and sand, in each case just enough of the finer material being used to fill the interstices of the next coarser, there will be in the mixture a porous space equal to $.40 \times .40 \times .40 = 6.4$ per cent, which will represent the proportion of clay necessary to make the mixture impervious. In practice it will take considerably more to insure the filling of all the interstices, as much as 15 or 20 per cent, depending upon the nature of the gravel mixture. In any case the percentage of voids in an artificial mixture can be readily determined by tests with water.

Porous embankments may be formed of sand, gravel, or loam, and if properly constructed are in some respects safer structures than one made largely of clay. If the material is properly graded from fine to coarse from the upper side to the lower, the fine material will act to prevent water coming through with sufficient velocity to wash away the coarser particles below which furnish stability to the structure.

387. Core-walls.—*Puddle Cores.*—Much has been written regarding the use or omission of core-walls, and the material of which they should be made. Theoretically, core-walls are needed only when the body of the embankment cannot readily be made water-tight. With an abundance of good material there is no object in using a core-wall of earth except perhaps that the chances of getting good work done are better if attention is concentrated on a small section. With a smaller quantity of good material it is best to concentrate it in a body, and with material still more scarce it will naturally be placed in a wall or puddle core. As between a bank in which the clay is confined to a narrow wall, and one where the same amount is mixed with a suitable proportion of gravel and forms a larger part of the embankment, the latter will be preferable. In deep trenches, and especially where much water is met with, a concrete filling is frequently used for a part of the depth.

For a puddle wall the minimum thickness ordinarily used is 4 to 8 feet at the top and about one-third the depth of water at the bottom, with a uniform batter on both faces. The trench is also usually made with a batter, the width at the bottom being one-third to one-half that at the ground-level, with a minimum of 4 or 5 feet.

388. In Fig. 72 is illustrated the embankment of a distributing reservoir at Syracuse, N. Y., built with a low core-wall. The material composing the body of the embankment was a mixture of heavy clay and a small amount of gravel. All hard lumps were broken up and all stones more than 4 inches in diameter were removed. The trench was

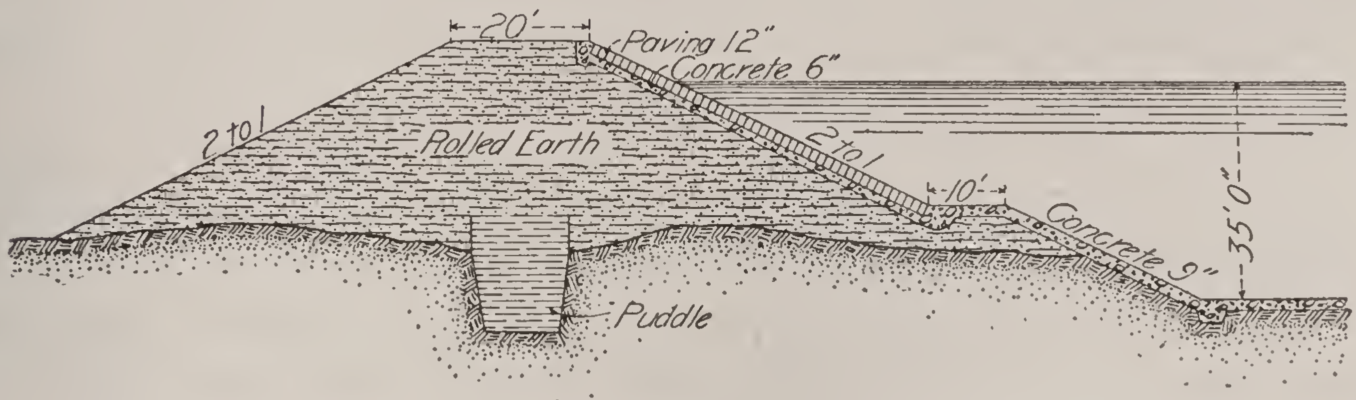

FIG. 72.—SECTION OF RESERVOIR EMBANKMENT, SYRACUSE, N. Y.

about 8 feet deep and 12 feet wide, and was filled with clay in 4-inch courses puddled in place.*

An embankment with full puddle core and selected material adjacent is shown in Fig. 73, which is a section of a reservoir embankment at Glasgow, Scotland.† The foundation stratum was of exceedingly varying nature, and at one place the trench was carried to a depth of 195 feet.

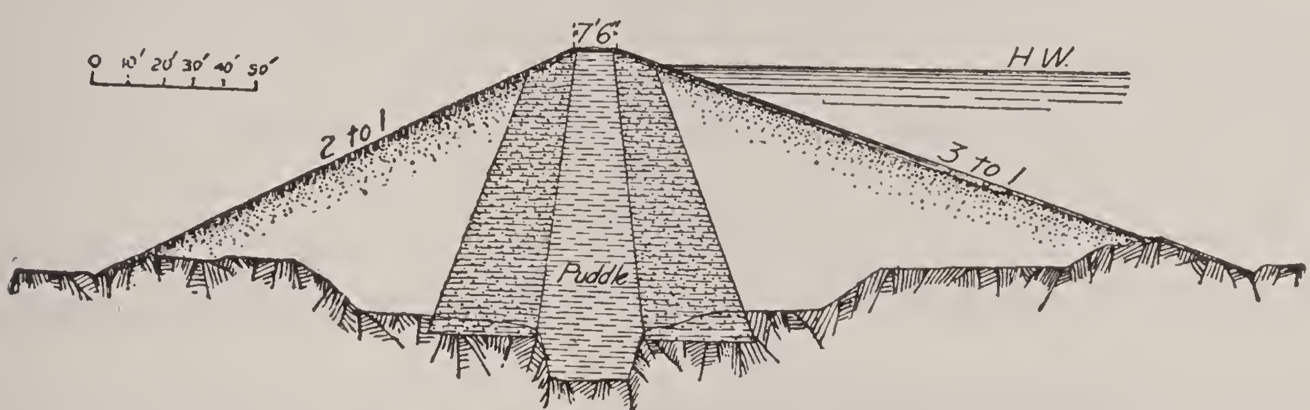

FIG. 73.—SECTION OF RESERVOIR EMBANKMENT, GLASGOW WATER-WORKS.

389. *Masonry Core-walls.*—Instead of a core of puddle, many engineers prefer a core of rubble masonry or of concrete, made as impervious as possible. The advantages of this over a core of puddle are its safety against attack by burrowing animals, safety against wash in case minute leaks occur, and the greater certainty with which a concrete wall can be made impervious, especially where it joins the foundation. It is also much better suited for placing in wet trenches, and, its thickness being less, the trench need not be as wide. Furthermore,

* Trans. Am. Soc. C. E., 1895, XXXIV. p. 37.
† *Engineering*, 1894, LVII. p. 704.

in case of an overflow the failure of the dam will be much delayed by a wall of masonry. The chief objections raised against it are its greater cost and the fact that with it the bank is less homogeneous and hence more difficult to construct, and more subject to injury by unequal settlement. Core-walls if made too thin are also liable to rupture from unequal earth-pressures. For these reasons it is even more important to avoid much settlement by a very careful compacting of the material than in the case where the embankments are entirely of earth.

On the whole, the practice in the eastern part of the country strongly favors masonry core-walls, especially for high embankments, and many engineers would use them as an extra precaution even where the entire embankment is of good material. In the West they are seldom used, and some of the highest embankments have been constructed without them.

Core-walls should be made with a batter, as this tends to prevent the separation of earth and masonry from settlement. Short buttresses constructed at intervals on the up-stream side are an additional precaution against the passage of water along the face of the core-wall.

To secure imperviousness the concrete should be relatively rich in mortar, and it is also advisable to plaster the upper face with neat Portland-cement mortar. This is the practice of the Boston Water Board, and experiments on certain dams thus constructed show little water in the bank below the core.

Masonry core-walls are made of various widths. Sometimes in case of embankments made of good material, they are made only a foot

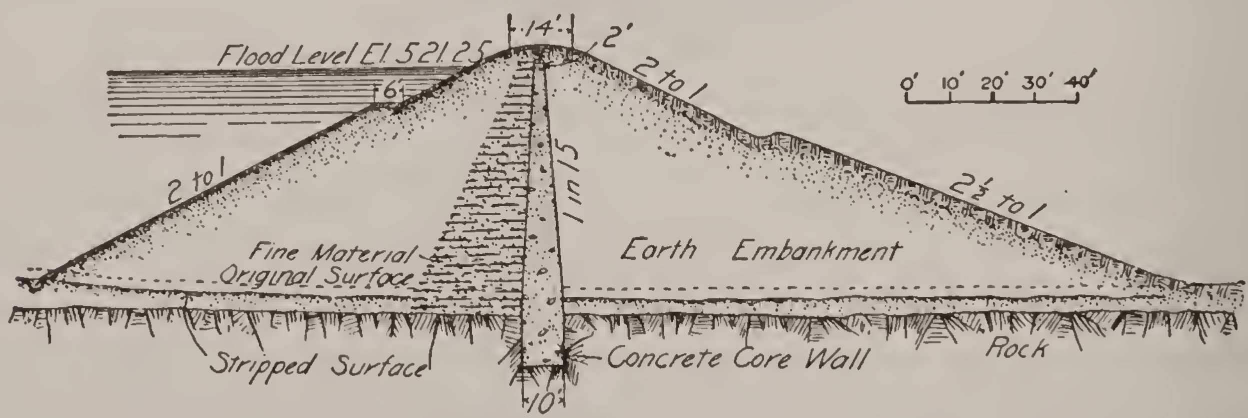

FIG. 74.—SECTION OF DAM NO. 5, BOSTON WATER-WORKS.

or two thick, their purpose being mainly to prevent the passage of burrowing animals. Ordinarily, however, a core-wall is made 2 to 4 feet thick at the top, with a batter of $\frac{1}{2}$ to $\frac{3}{4}$ inch per foot on each side down to the trench and then with vertical faces below. The height of a core-wall should be equal to that of the highest water-level.

390. Fig. 74 is a section of the earthen portion of Dam No. 5 of the Boston Water-works, and represents what may be considered as the most advanced practice in this type of construction.* Fig. 75 is the section, as designed, of the earthen portion of the New Croton Dam. The masonry core extends to solid rock. The dam as constructed is not so high above foundation as the section shown.†

391. *Core-walls of Wood and Steel.*—Sheet-piling is sometimes used to good advantage in the bottom part of an embankment, but to be durable it must be in a position where it will be kept constantly wet. It is especially serviceable with low embankments built on a porous foundation and in temporary work.

A core of steel imbedded in concrete has been used in a rock-fill dam at Otay, Cal. The steel varied in thickness from No. 0 to No. 3, Birmingham gauge (.34 inch to .259 inch). The plates were riveted and calked and coated with asphalt. This steel core was protected

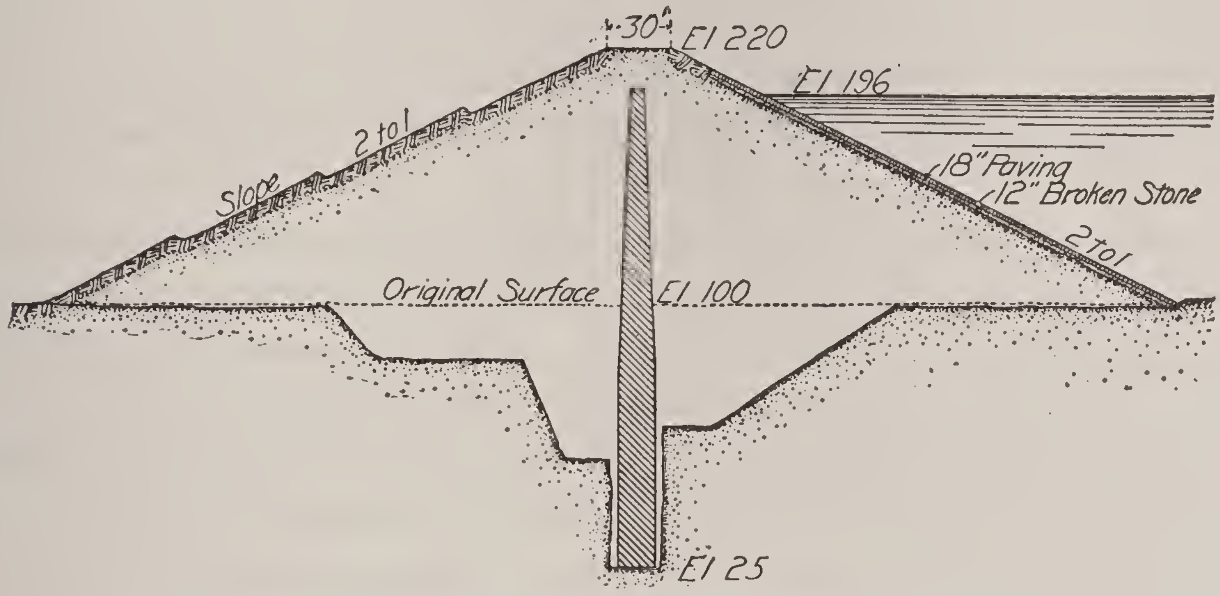

FIG. 75.—DESIGN FOR NEW CROTON DAM.

on each side by a wall of concrete 1 foot thick. (See Art. 455.) Such a wall in an earthen embankment would be absolutely safe against percolation even though slight cracks should form in the masonry. Compared to a wall entirely of masonry it could be made much thinner for the same strength, and as the cost of a $\frac{1}{4}$-inch plate would not be more than the cost of 1 or 2 feet of concrete, a considerable saving could be effected. At the bottom and ends of the dam the masonry wall should spread out to the ordinary width. In a thick core-wall the riveting and calking of the plates might be dispensed with.

392. *Position of Core-walls.*—In embankments for impounding-reservoirs core-walls are usually placed at or near the centre of the dam. The effective weight of the structure would evidently be increased by placing it near the up-stream face, and this is sometimes

* *Eng. News*, 1895, XXXIII. p. 230.

† Trans. Am. Soc. C. E., 1900, XLIII. p. 469; also 1906, LVI. p. 32. See *Eng. News*, 1904, LI. p. 335, for example of steel core in an earthen embankment.

done where made of puddle. The disadvantages of so doing are that much more puddle is required for the same thickness, it is not so readily placed, and is more exposed to frost and water action and to drying out when the water is low. There is also more danger of slips when the water is drawn down, such as have occurred in several places.

393. Embankment Slopes.—Much variation exists in practice in the matter of side slopes, even with similar material. On the water side the slope is usually protected from wave action and should only be sufficient to prevent slips. With coarse material this need not be flatter than 2 horizontal to 1 vertical. With finer material it may need to be $2\frac{1}{2}$ or 3 to 1, or in some cases even 4 to 1, since earth in a saturated condition has a comparatively small angle of repose. On the lower side the material will be dry if made more porous than the upper portion, and the angle of repose will be about $1\frac{1}{2}$ to 1, but the stability of the embankment is largely dependent upon the lower half, as pointed out in Art. 385, and it is desirable to use a somewhat flatter slope than that at which the material will just stand. A slope of 2 to 1 is therefore to be recommended, although $1\frac{1}{2}$ to 1 has frequently been used. If the material will stand at 1 to 1, as broken stone, for example, then a slope of $1\frac{1}{2}$ to 1 would be suitable. On high embankments, bermes placed 30 to 40 feet apart vertically are a desirable feature. On the up-stream side they form additional supports for the paving, while on the down-stream side they allow of lateral drainage by means of paved gutters, thus protecting the slope to a considerable extent from scour due to heavy rains. This arrangement also gives a little greater safety and stability at the bottom of the embankment where it is the weakest. Below the berme the slope is often made flatter than above, thus securing some additional width with little extra cost. This modified form is particularly adapted to soft foundations.

394. Height above Water-line.—The top of the dam should extend sufficiently above the high-water line to protect the material exposed to water action from frost and to give a safe margin against overflowing. This will be equal to the depth reached by frost plus an allowance of 2 to 5 feet for wave action, depending on the exposure to winds and the depth of the water. A formula given by Stephenson for height of waves in such cases is

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

in which H = height in feet and D = length of exposure, or "fetch," in miles. For very low embankments the height as determined above is not always attained, more dependence being placed upon width of top, which will be relatively great in such cases. In climates where protection from frost is not required there should be a larger margin of safety between the highest waves and the top of the dam, as an earthen embankment must not be overtopped by the water.

395. **The Width of Top** is frequently fixed by requirements for a roadway. Where not so fixed it is made to vary with the height,—from 6 to 8 feet for very low embankments to 20 or 25 feet for embankments 80 to 100 feet high, or, approximately, width $= \frac{1}{5} h + 5$ feet, where $h =$ height of dam.

The reasons for increasing the width with the height of embankment are to secure safety against the increased action of waves and of ice, and to add to the general stability of the structure. With no increase in top width a high embankment would be relatively less secure than a low one, while it is desirable to have it the reverse on account of the much more serious results of a failure of a high embankment.

396. **Preparing the Foundation.**—In preparing the foundation the surface-soil must be removed over the entire site of the embankment to a depth sufficient to reach good sound material. All roots, stumps, and other perishable material must be removed, as any such material by decaying offers a passage for water. For high and heavy embankments it is important that the excavation under the main body of the dam, and especially near the core-wall, be carried to a very firm foundation in order to avoid settlement. Near the toes of the dam the weight is much less and a softer material will support it. For the portion to be occupied by the core-wall, if one is used, and a certain width in any case, the foundation must be excavated to an impervious stratum of solid rock or clay, and penetrate for a short distance such stratum. Where disintegrated and fissured rock is met with, the construction of a safe embankment requires the most careful work in preparing the foundation. In some recent cases trenches have been carried to depths of nearly 200 feet before a secure bottom has been reached.

A sound bottom having been reached the surface should be roughened in order to give a better bond with the earth filling; and if the material is solid rock, all holes and crevices must be thoroughly cleaned and filled with cement or concrete. Springs of water met with on the foundation area are often very troublesome and dangerous, especially if under or near the core-wall. If flowing with a small head, they may be quite readily stopped up with concrete. If under considerable head, an attempt to smother them at first will likely cause them to break out at some other place. In such a case they are often dealt with by confining the water in a little well of concrete or in a pipe until a few feet of embankment have been built, then pumping out the well and quickly filling with concrete. Sometimes strong springs have been piped to the outside of the embankment, and this can safely be done where they occur in the down-stream portion of the dam, but this is otherwise a doubtful expedient.* Whatever seepage-water gets through an embankment should run perfectly clear, as muddy water denotes a washing out of the material.

397. **Construction of the Embankment.**—After the foundation has been prepared the trench is first filled with the material selected. If puddle, it should be placed in 4- to 6-inch layers well rammed, or cut and cross-cut with thin spades reaching well into the layer below, just enough water being used to render the material plastic. Where puddle is used in a narrow wall it is advisable to prepare it before placing by thoroughly pulverizing and tempering it with water, no more water being used than absolutely necessary. A pug-mill is very useful for this purpose, especially where artificial mixtures are employed. Puddle should be thoroughly worked and homogeneous. If concrete is used, special care must be taken to secure thoroughly good work in mixing and ramming, and in filling all irregular spaces in the excavation.

After the core is built to the surface, or a little above in the case of concrete, the main embankment is started. If the material used varies in quality, the finer and better should be placed above and adjoining the core-wall, and the coarser placed on the down-stream side and near the faces. If no core-wall is used, the better material should still be placed in the up-stream portion of the embankment. Stones exceeding 3 or 4 inches in diameter should not be allowed in the embankment except along the faces. The embankment is compacted usually by placing the material in layers 6 to 12 inches thick, wetting, and rolling with a heavy grooved roller weighing 200 to 300 pounds per lineal inch. These rollers are often made by stringing cast-iron disks on an axle, the alternate disks being 2 or 3 inches different in diameter. Specially made rollers can also be had for this purpose.

Much importance is attached to the work of compacting, and only by the best of supervision can good work be secured. The use of water should be just sufficient to render the material plastic and capable of being packed, and no more. An excess of water makes rolling more difficult and increases subsequent settlement. Many

* See description of Tabeaud dam, *Eng. News*, 1902, XLVIII. p. 26.

apply the water before the layer is placed, instead of afterwards, with good results.

Embankments have been built of dry material, and if thorough ramming could be secured, this method would probably give a bank tighter and less liable to settlement than by the use of water. With some material, however, it is desirable to use a certain amount of water to reduce the lumps. If well-compacted, the settlement will be very slight, as small an amount as ½ inch in 50 feet having been reported by Mr. FitzGerald. With a masonry core-wall it is especially important that the settlement of the embankment be small.

During the construction of the main body of the embankment the core-wall if of puddle is carried up simultaneously therewith, thinner layers of material being used in this part, and more care in rolling. A concrete core-wall must be kept a few feet in advance of the earthen portion, and the latter well rammed against it.

398. Hydraulic Dam-construction.—In the western part of the country many good embankments have been built by this method. The material is, where practicable, obtained from the adjacent hillsides, from which it is loosened by a water-jet and conveyed to the top of the dam by water flowing through pipes or flumes. There it is allowed to settle in a pond maintained by keeping the edges of the dam higher than the centre. The cost of construction where the conditions are favorable is exceedingly low.

The following description of a dam constructed at Tyler, Texas, will further explain the process: *

The dam is 575 feet long, 32 feet high, and contains 24,000 cubic yards, the inner slopes being 3 to 1 and the outer 2 to 1, with a 4-foot berme on the inside 10 feet below the top. All of the materials used in the dam were sluiced in from a neighboring hill at a cost of 4¾ cents per cubic yard, including the plant and all the appurtenances of the reservoir. Water was pumped through a 6-inch pipe and directed against the hillside from a nozzle at a pressure of 100 pounds per square inch. The material washed down consisted of 65 per cent of sand and 35 per cent of clay.

"In beginning the work a trench 4 feet wide was excavated through the surface-soil down into clay subsoil, a depth of several feet, and this was first filled with selected puddle clay sluiced in by the stream. Then the form of the dam was outlined by throwing up low sand ridges at the slope lines, which were maintained, as the dam rose in height,

* From U. S. Geolog. Survey, 1896–97, pp. 654–5.

by men with hoes. A pond of water was thus maintained over the top of the dam, the water being drawn off from time to time, either into the reservoir or outside as preferred. The material was transported from the bank in a 13-inch sheet-iron pipe, with loose joints, stove-pipe fashion, extending from near the face of the bluff, where the jet was operating, across the centre line of the dam. These were so arranged as to be easily uncoupled at any point, so as to direct the deposit where required to build up the embankment uniformly. It was found that the quantity of solids brought down by the water varied from 18 per cent in solid clay to 30 per cent in sand."

The entire cost of the dam with all its accessories is given at $1140. Mr. L. W. Wells was the engineer in charge.

399. Slope Protection.—The up-stream slope must be protected from wave and ice action. This protection is usually afforded by a closely laid pavement about 18 inches thick laid on 6 to 12 inches of broken stone or gravel. Below low-water line a good layer of riprap is frequently substituted, the paving ending at a berme. The foot of the paving should be well supported by large blocks of stone or concrete. Where large gravel and boulders are abundant the face can be well protected by such material placed loosely, the larger stones being on the outside to resist the impact of the ice and waves. Paving should preferably not be put in place until all settlement has ceased. (For impervious linings see Chapter XXVII.) The down-stream face is usually sodded for sake of appearance and as a protection from rain, but may be protected by gravel and coarse material if more convenient. The edges of the embankment should have rounded rather than angular outlines. Where considerable seepage exists it is desirable to fill in at the outer toe to some depth with broken stone, as this aids in drainage and in maintaining the slope.

400. Embankments and Foundations of Porous Material.—Sand and ordinary porous earth have been successfully used in embankments of considerable size. In their construction it is necessary to bear in mind the effect of percolation on the stability, both in tending to wash out the material, and to decrease its effective weight. Percolation should in the first place be limited as far as possible, and to this end the embankment should be made broad and with flat slopes, especially on the lower face or the lower portion of the lower face. The upper slope may be made as usual if protected by paving. To prevent the material from washing out, the upper part should, if possible, be made of finer material than the lower, the change from one to the other being gradual. The velocity of the percolating water is thus much less in the lower portion of the dam where the material is unsupported, but where the particles are larger and less easily moved by the water.

If the foundation is also porous, as is apt to be the case, it is also necessary to prevent a high velocity of percolation through it. This is accomplished by the broad embankment which forces the water to pass farther through the material, and can also be aided by driving a line of sheet-piling along the center of the dam. The entire dam and foundation should thus be built like a gigantic filter, the object being, in the first place, to prevent percolation as far as possible by the use of the finest available material on the up-stream side, and, in the second place, to so support this material as to permit any percolating water to escape without causing damage to the dam or its foundation. In case the foundation material is very soft the embankment must be spread out to reduce the load carried. To prevent the squeezing out of the foundation material, it may be excavated to a considerable depth at both toes and replaced by gravel or concrete. It may also be desirable to load the earth to a considerable distance below the embankment proper by means of a low bank. Drainage at the outer toe is serviceable in lowering the line of saturation and maintaining a drier slope.

The great Gatun dam on the Panama Canal is designed on the principles here set forth. (See Fig. 76.) In section it is about 135 feet high, with inner slope of 1 : 2 and outer slope of 1 : 25, and a base

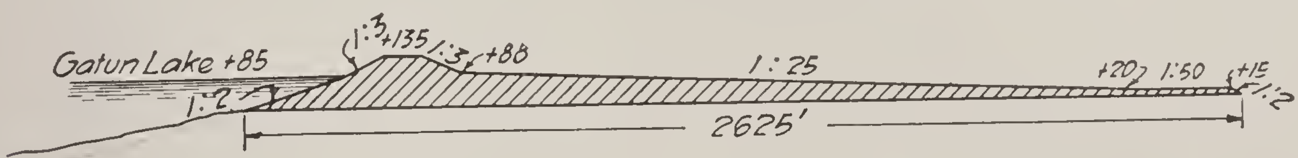

Fig. 76.—Section of Gatun Dam.

width of about one-half mile. It is virtually an artificial hill in which the percolating water will act exactly as ground-water.*

401. **Outlet-pipes.**—The design and construction of the outlet arrangements is one of the most important and at the same time most difficult features of the work. This is chiefly because of the difficulty of laying pipes or building masonry conduits through earth embankments in such a manner as to secure a perfect and reliable connection between the two materials. Poor work at this point is one of the chief causes of the many failures of earth embankments.

In reservoirs of any considerable depth it is desirable so to arrange the outlets as to enable the water to be drawn off at different levels, not exceeding 10 or 15 feet apart, in order that water of the best available

* For a full discussion of this subject see paper by George Morison on the Bohio Dam, Trans. Am. Soc. C. E., 1902, XLVIII. p. 235.

quality may at all times be obtained. Provision must also be made by suitable gates or valves for controlling the flow or turning it into a conduit or other channel; and to make the operation as reliable as possible, all valves or other working parts should be readily accessible.

The outlet-pipes are usually of cast iron and may either be laid underneath the embankment and surrounded thereby, or a culvert of masonry may be constructed in the embankment and the pipes laid therein, or they may be laid in a tunnel constructed in the natural ground at the end of the embankment or at some more remote point in the reservoir. A gate-chamber containing the necessary valves is located at some point along the outlet-pipe or conduit. The size of the pipe must be such as to deliver water at the required rate without too great loss of head as determined by considerations of economy, or by the head available. This will usually limit the velocity to 4 or 5 feet per second. For large quantities two or more outlet-pipes are used.

402. *Pipes Placed in the Embankment.* — In the case of reservoirs with comparatively low embankments the outlet-pipes are usually laid beneath the embankment at or near the lowest point. They should be laid on a good firm foundation in the natural ground, and should preferably rest upon and be surrounded by a bed of 8 to 12 inches of

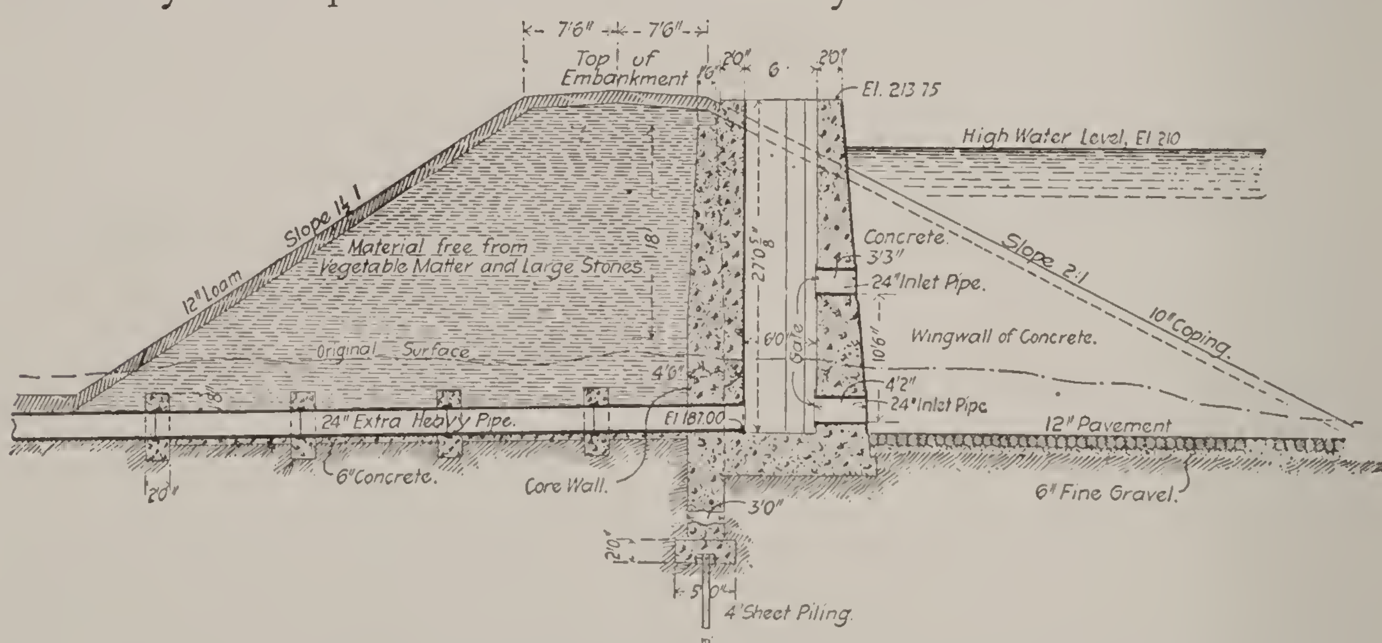

FIG. 77. — SECTION THROUGH OUTLET-PIPE, NEW LONDON RESERVOIR.
(*From Engineering Record*, Vol. XLVI.)

rich concrete, well rammed into the trench and left rough on the outside. To enable the earth to be more thoroughly bonded with the concrete, cut-off walls should be built projecting out from the main body of the concrete, $1\frac{1}{2}$ to 2 feet, as shown in Fig. 77. If concrete is not used, then the pipes should be provided with wide flanges. They should be very carefully laid and tested under pressure before covering.

If the embankment has a masonry core-wall, a good secure connection can readily be made at this point between pipe and embankment.

If the core is of puddle, great care must be taken in thoroughly ramming the puddle about the concrete. If the trench extends below the pipe, it should be filled underneath with concrete rather than puddle, as otherwise settlement and rupture are very liable to occur. The great difficulty of securing reliable work at this point, and the failures which have occurred, have led many English engineers to strongly favor the use of tunnels.

403. *Culverts.*—For some reasons an open culvert is much to be preferred to a pipe. Once constructed, additional pipes may be laid therein at any time; the pipes may also be readily inspected, and any leaks that occur in them do not endanger the structure, a matter of especial importance where the pipes are under heavy pressure. The culvert may also be conveniently made to act as a wasteway for the stream during construction.

The same precautions must be taken in the construction of culverts as in the laying of pipes. They must have a good firm foundation and a good bond with the surrounding embankment. The cross-section must be amply strong to resist all lateral and vertical pressures, the latter being assumed to act upwards as well as downwards, and, in the upper portion of the embankment, to be equal to the full water-pressure of the reservoir. Reinforced concrete is especially well suited for work of this character. Imperviousness is secured by the use of a rich mortar and by plastering on the outside with Portland-cement mortar neat or 1 to 1. Cut-off walls or projecting courses should be built around the outside at intervals as described for pipe outlets. At the connection with the gate-house a cut-off wall is put in through which the pipes pass, and which must sustain the full head of water.

Fig. 78 illustrates a culvert constructed through an embankment of an impounding-reservoir, with outlet-pipes laid therein and opening into a gate-chamber at the upper toe. Where the gate-chamber is placed just above the core, the culvert may stop at that point and pipes be used to conduct the water from reservoir to gate-house. Where the water is turned into the natural watercourse below, the pipe may be dispensed with, the water passing through the open culvert.

The outlet arrangements of the new settling reservoirs for Cincinnati are shown in Fig. 79. Here the culvert is constructed in the natural ground and has a very heavy section. A $\frac{1}{2}$-inch coat of Portland-cement plaster on the outside insures imperviousness.

404. *Tunnels.*—If a tunnel be used, it may be made straight and pass underneath the embankment, or may turn an angle and pass around it altogether (the gate-chamber being placed at the angle), or

it may cut through a narrow place in the divide and lead the water into another valley, a rare but very favorable arrangement. With deep

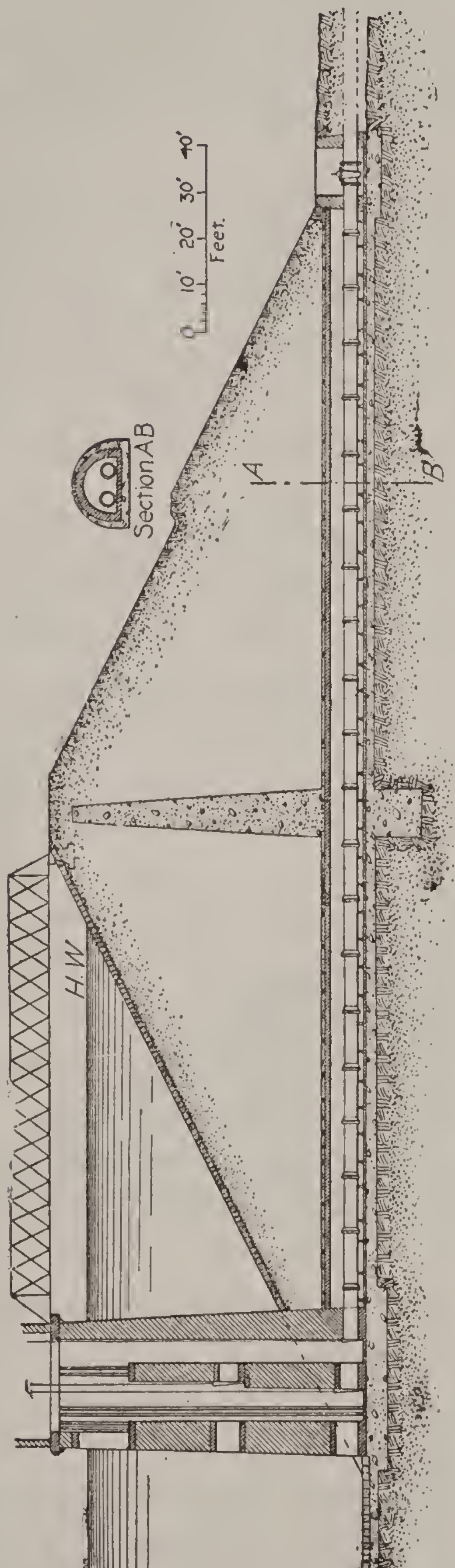

FIG. 78.—SECTION THROUGH EMBANKMENT, CULVERT, AND GATE-CHAMBER.

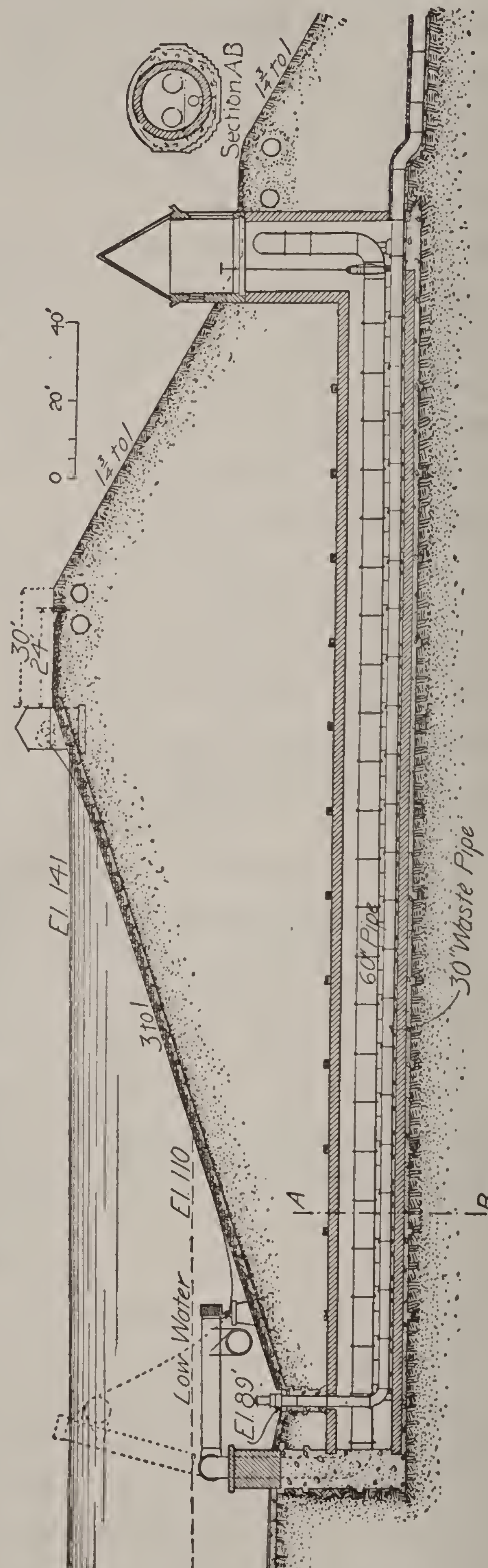

FIG. 79.—RESERVOIR EMBANKMENT, CINCINNATI, O.

puddle trenches or soft foundations it is desirable to entirely avoid cutting into or under the dam.

If the material through which the tunnel passes is anything but hard impervious rock, the tunnel must be lined with brick, and back of the lining the excavation must be thoroughly filled with concrete. In rock a tunnel is entirely satisfactory, but in earth it is difficult to avoid disturbing the strata, and the back-filling is much more difficult than in the case of a culvert constructed in open trench. Sometimes in solid rock a trench instead of a tunnel is dug around the end of the dam, and the gate-chamber located therein.

405. **Gate-chambers.**—The gates or valves controlling the flow through the outlet-pipes are placed in small masonry chambers, which, besides allowing of convenient operation of and access to the valves, also usually contain screening-chambers and valve arrangements whereby water may be drawn from different levels.

406. *Position of the Gate-chamber.*—Gate-chambers are preferably placed at or near the upper end of the outlet-pipes in order that the pressure therein may be under control. They are, however, sometimes placed at the outer toe of the embankment, but this is undesirable, as it is impossible to shut off the water from the pipes in case of leakage except by the use of divers. This point is of more importance with large dams than in the case of small distributing reservoirs. In dams with core-walls the gate-chamber may properly be placed anywhere between the core-wall and upper toe, and with core-walls of masonry it is conveniently placed just above and adjoining the masonry core.

An advantage gained by placing the chamber at the inside toe is that it enables arrangements to be easily made for drawing water from different levels. Fig. 78 shows a gate-chamber in this position. A foot-bridge is here necessary to allow of access to the gate-house. This position exposes the chamber to severe stresses from the action of ice and is therefore more suitable for large than for small structures. If the chamber is placed farther back in the embankment, the necessity of a bridge is avoided and the structure is much better protected from the action of ice, but the drawing of water from different levels is not so convenient. It may be drawn from the bottom by a continuation of the outlet-pipe or culvert to the upper face of the embankment. It can also be drawn from near the top by an inlet in the masonry wall or by a short inlet-pipe. To draw from intermediate levels, inlet-pipes or sluices must be extended to the face, as in Fig. 81; or an adjustable inlet-pipe may be employed, as is common with distributing-reservoirs and as illustrated in Fig. 79; or the embankment may be removed from the upper face of the chamber and supported on the sides by heavy wing walls, thus enabling the water to be drawn through ordinary sluiceways as in Fig. 77. This last method becomes very expensive with high embankments. In Fig. 81 the first and last methods are combined.

407. *General Arrangements.*—The various forms of gate-chambers may be further described in connection with the examples illustrated by the figures. The simplest form is shown in Fig. 77, page 358, and consists merely of a single chamber built over the valve in the single outlet-pipe. A separate waste-pipe is here provided. The illustration refers to a distributing reservoir, but the arrangement is suitable for small reservoirs where screens are not required and where it is necessary to draw water from but one level.

Fig. 80 illustrates a design suitable for small reservoirs. This

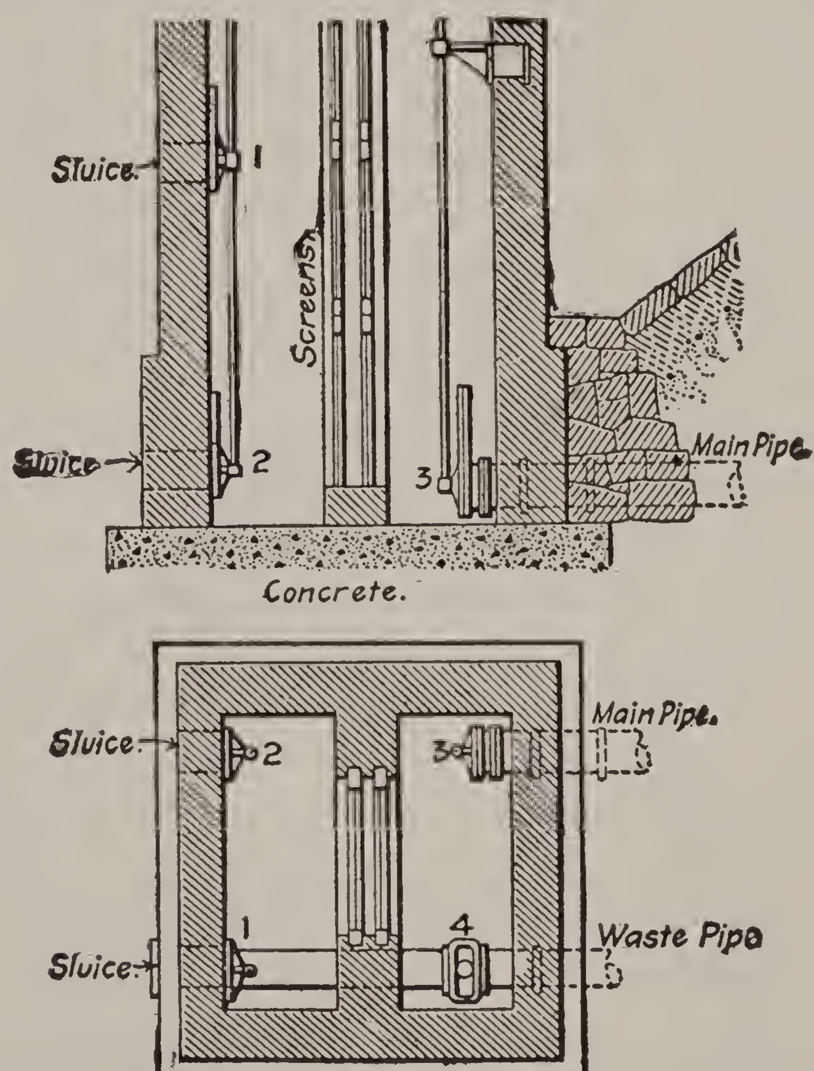

Fig. 80.—Gate-chamber, Ipswich, Mass. (Goodell.)

arrangement permits of drawing water from near the bottom and from about mid-depth. Screening is also provided for.

Fig. 81 illustrates a form adapted to larger works and shows how pipes may be arranged to draw from different levels when the gate-chamber is placed in the body of the embankment. Grooves are provided for screens and for stop-planks for regulating the flow from the surface of the reservoir. A waste-pipe is shown and also an overflow, the reservoir in question being a distributing-reservoir.* (See Chapter XXVII for further details of distributing-reservoirs.)

In Fig. 78 is shown a still more elaborate gate-chamber suitable for the largest reservoirs, and similar to that used in some of the reservoirs of the New York and Boston Water-works. (See also Fig. 99, page 397.) The structure is divided longitudinally into two chambers. The division-wall contains the sluice-valves for drawing water at different levels, admission to the outer chamber being through large openings placed opposite the valves. In the outer chamber are grooves for screens which may also be used for wooden stop-planks in case of emergency. As an additional measure of safety the upper end of the

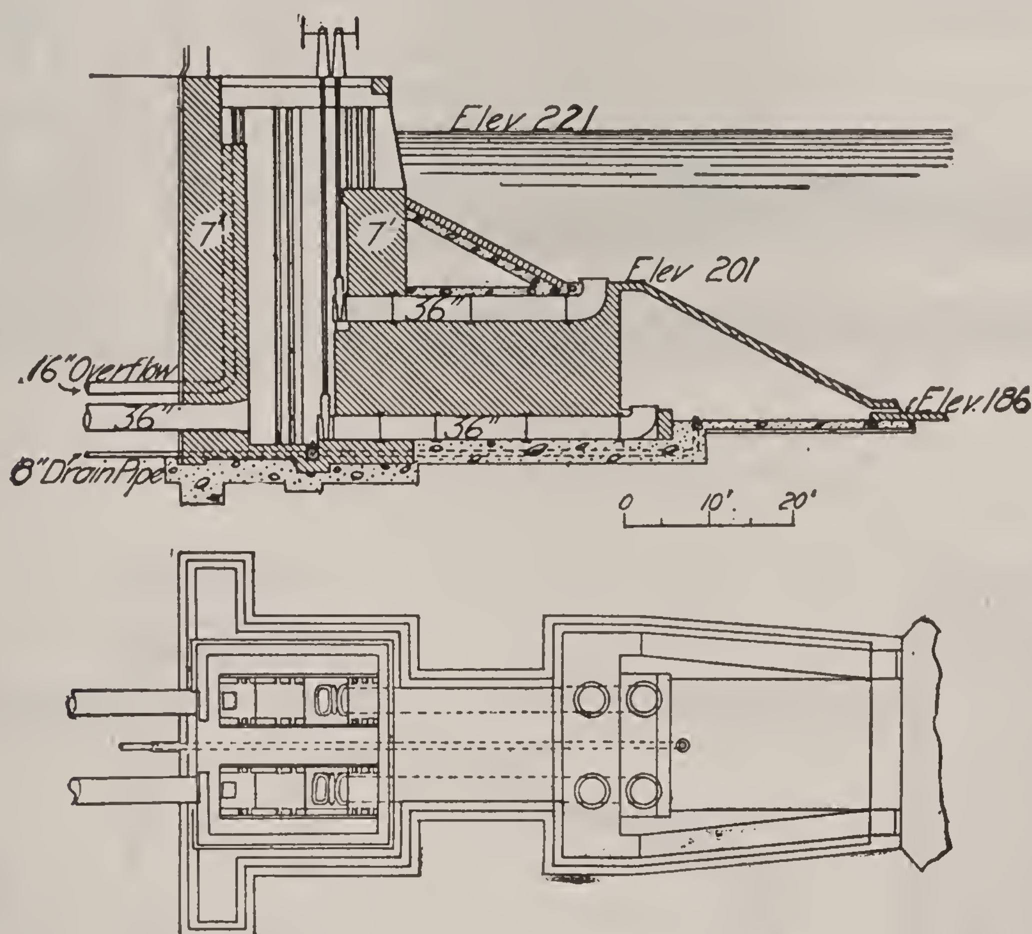

FIG. 81.—OUTLET-CHAMBER, SYRACUSE WATER-WORKS.

inlet-pipe may be provided with a valve as shown. At the lower face of the dam is usually placed another valve-chamber containing valves for directing the flow into waste-pipes, or into a conduit, or otherwise, as the case may be. This also provides a more convenient place for the daily regulation of the flow. Where there are two or three outlet-pipes the chamber is divided into a corresponding number of divisions, each of them arranged to be operated independently. A preferable arrangement to that shown would be to place the gate-chamber just above the core-wall, which is the usual Boston practice. One of the methods

* Trans. Am. Soc. C. E., 1895, xxxiv. p. 46.

described in Art. 406 would then have to be adopted if water is to be drawn from different levels.

Fig. 82 illustrates a large gate-chamber on one of the distributing-reservoirs of the New York Water-supply. This design combines many of the desirable features already mentioned. Note that access is had to the pipe-line from the gate-chamber.

A form of inlet-tower used much in English practice is shown in

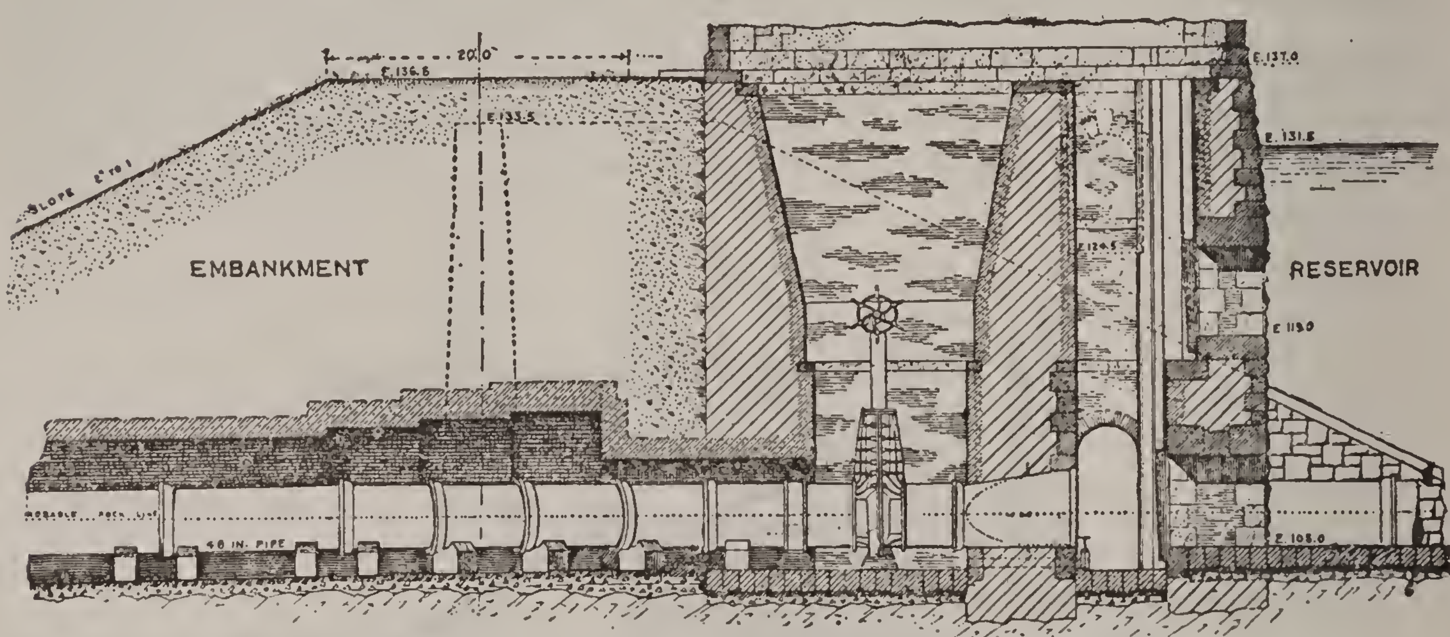

FIG. 82.—JEROME PARK GATE-HOUSE.
(From Wegmann's "Water-supply of New York.")

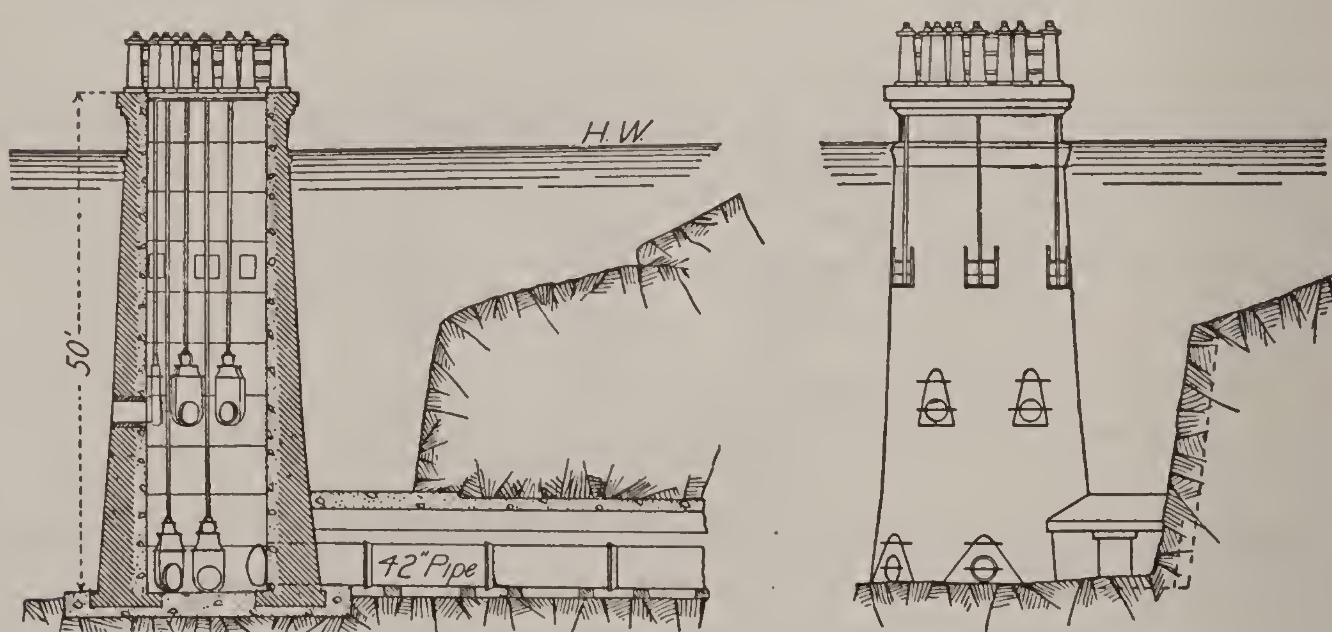

FIG. 83.—INLET-TOWER, GLASGOW WATER-WORKS.

Fig. 83. It consists of but a single chamber, the inlets being placed at various levels. A separate screen-chamber is built on shore.* For towers having a single chamber the circular form is to be commended.

Arrangements differing considerably from those above described have also been used with satisfactory results. The proper one to adopt

* *Engineering*, 1894, LVII. p. 738.

in any particular case depends upon local conditions and is determined by considerations of safety, economy, and convenience of operation.

Gate-chambers are sometimes entirely dispensed with, and the sluice-gates built into the sloped embankments, with rods for operating them carried up the inclined face to mechanism above. This arrangement is suitable only in mild climates where trouble with ice is not to be feared. It is cheap, but not as reliable or as convenient in case of stoppage as the gate-chamber.

408. *Details.*—The masonry of the inlet-tower is usually of heavy rubble, faced with ashlar and lined with hard brick or cut stone. Reinforced concrete is also well adapted to this work, as it may be quite closely calculated to resist the forces acting and will usually effect considerable saving over the use of stone masonry.

The tower as a whole when located at the toe must be able to resist ice and wave action, and each wall the unbalanced pressure of the water. Walls of stone or brick masonry will vary in thickness with their unsupported length. The exterior walls are usually made 3 to 4 feet thick at the top, with an increase of about three-fourths inch to 1 inch in thickness per foot of depth, the batter being made on the outside for convenience and to furnish a better bond with the earthwork. Interior walls may be made of slightly less thickness. Reinforced concrete walls, where subject to impact from ice or other cause, should be made considerably thicker than the static pressures require. The foundation should be prepared with great care. If the gate chamber is placed near the toe, the load will be much heavier than the surrounding earth embankment, and unequal settlement is liable to occur, causing cracks in the masonry of the culvert and displacing the outlet-pipes. The bottom of the gate-chamber should be constructed under the supposition that full water-pressure will exist underneath the chamber when empty. It is best made of reinforced concrete.

Fish-screens are usually copper-wire screens with $\frac{1}{8}$ to $\frac{1}{4}$ inch mesh, fastened to wooden or iron frames and arranged to slide in grooves in the masonry. They are arranged in pairs, and each screen is made up of several elements of a size convenient to handle.*

The gate-chamber is surmounted by a gate-house in which is located the operating mechanism of valves and screens. As this building is frequently quite prominent, it is important that it be given an artistic treatment suited to the surroundings. Two very commendable designs are illustrated on page 367, and show what may be done in

* For details of screen and mechanical lifter used at distributing-reservoirs of the Boston water-works, see *Eng. News*, 1900, XLIV. p. 218.

this direction. The former illustration is taken from Wegmann's "Water-supply of New York," and the latter is from a photograph loaned to the authors by the engineer, Mr. L. M. Hastings.

The bottom of the reservoir should be paved near the gate-chamber and the lower sluiceway placed close to the bottom; or a separate drain-pipe may be provided as shown in the illustrations. This is a necessary feature in small distributing-reservoirs requiring frequent cleaning. If the gate-chamber is not located at the very bottom of the valley, a drain-pipe may lead to such point and be operated as a siphon when it is desired to drain the reservoir. Where much sediment is deposited it is desirable to have a large sluice-gate at the very bottom to use in flushing out the material near the dam.

409. Valves and Sluice-gates.—The inlets into the gate-chamber are made to correspond in size with the outlet-pipe. For small inlets the most convenient form is a small piece of pipe built into the walls with an ordinary gate-valve attached thereto, as shown in Fig. 81, or a small sluice-valve as shown in Fig. 80. Large valves require a good broad support, and in narrow walls and chambers it is more convenient and also cheaper to use in most cases cast-iron sluice-gates of the latter form. These large gates are usually of special design, made with ribbed faces on the side towards the water-pressure, and plane on the other side, as is ordinarily done with cast-iron plates. The gate is made to slide in grooves faced with brass or bronze, and the sliding surfaces of the gate are similarly faced.

Where the water-pressure tends to force the gate off its seat, some form of wedge arrangement must be used to force the gate to its seat when nearly closed. Such an arrangement is shown in the gates of the St. Louis intake (Fig. 43, page 265), the wedge being formed by an additional groove with brass facing. Instead of a continuous inclined groove such as this, a series of adjustable blocks is sometimes employed against which bear corresponding projections on the back of the gate. When the pressure holds the gate to the face a simple groove is sufficient.

The frame of the gate is usually of cast iron, bolted securely to the masonry, in which case the opening is lined with cut stone; or cast-iron pipes or sluices may be built in the masonry and at the same time serve as attachments for the frames. The latter method is employed at Syracuse, and Fig. 86 illustrates the sluice-gate there used.*

Small sluice valves are operated by hand-wheel, larger ones by

* Trans. Am. Soc. C. E., 1895, XXXIV. p. 27.

FIG. 84.—CENTRAL PARK GATE-HOUSE, NEW YORK. (WEGMANN.)

FIG. 85.—PAYSON PARK GATE-HOUSE, CAMBRIDGE.

worm-gearing proportioned according to the pressure and available power. When convenient, hydraulic power, using a mixture of water and glycerine, as at St. Louis and Cincinnati, is very suitable, each cylinder being readily proportioned according to the load. The cylinders can be so arranged that in case of failure of the pressure they may be operated by a hand-pump.

410. Waste-weirs.—As already noted, one of the most fruitful causes of reservoir failures is insufficiency of waste-weir capacity, resulting in the overflowing of the dam and its rapid destruction. Mention need only be made of the terrible Johnstown disaster in 1889.

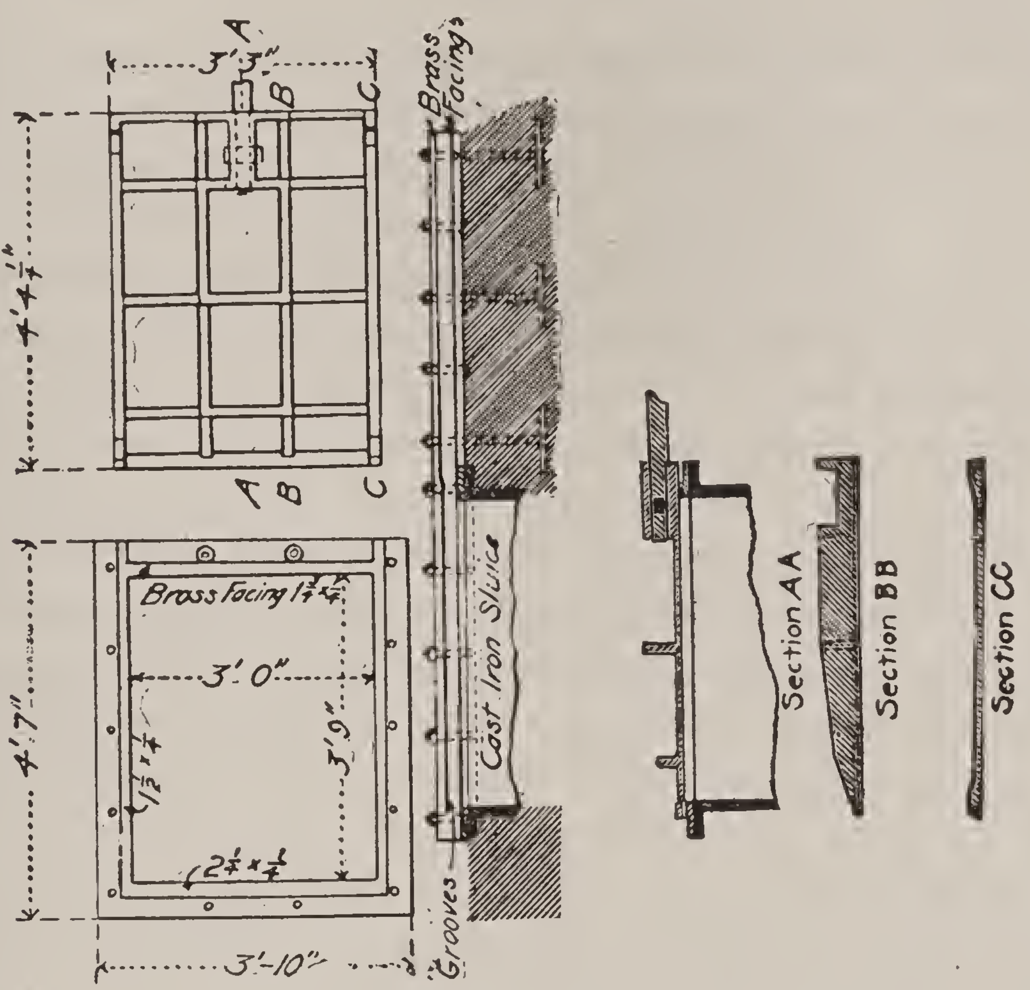

FIG. 86.—SLUICE-GATE, SYRACUSE WATER-WORKS.

where, on account of insufficient wasteway, an earthen embankment was destroyed, resulting in the loss of over 2000 lives and the destruction of property valued at 3 to 4 million dollars.*

In Chapter VI the subject of maximum flood-flows was fully discussed. The maximum flood having been estimated, it remains to provide some safe means whereby it may be passed to the valley below.

This is done in three different ways: (1) A wasteway may be excavated in the natural ground at one or both ends of the dam. Where the foundation is of rock this is a very safe and effective form of

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* See Report of Investigating Committee in Trans. Am. Soc. C. E., 1891, XXIV. p. 431.

wasteway, but care must be taken to have it of sufficient slope and cross-section at all points to carry the required amount of water at the assumed depth. On earth foundations the slopes of such a channel would need to be thoroughly protected with heavy solid masonry in cement. It will, however, seldom be economical to construct a wasteway of this kind in earth.

(2) The wasteway may sometimes be formed at some low point in the dividing ridge, and the water led to another valley. This is likely to require considerable attention in providing a safe channel for the increased quantities of water carried in the other valley, particularly at its upper end.

(3) The third form of wasteway is provided by making a portion of the dam of masonry designed as a spillway, and placed at about the axis of the valley. The forms of such dams are discussed in detail in Chapter XVII. At the junction of the masonry and the earth portions, the lower slopes of the embankments must be retained by heavy wing walls built out from the masonry dam. The upper slopes may be likewise protected, or they may be carried around in front of the masonry weir throughout its entire length. Where the earth and masonry portions join, great care must be taken to ram the earth solidly in place. Particular attention should also be given to the connection between core-wall and masonry. The back of all walls touching the earth should be left rough and be built with a batter. The advantage of a masonry core-wall is here obvious. Fig. 99, page 397, shows the plan of wing wall at the junction of a weir and an earthen embankment which well illustrates the foregoing points.

411. *Proportions of Waste-weirs.*—The requisite capacity being known, the length and depth of weir are to be determined. Either may be assumed and the other computed by means of a weir formula, but in each case there are certain proportions that will be the most economical. A low weir requires a greater length, whereas a deep and short weir requires, for the same storage volume, that the rest of the dam be made higher. The proper proportions are thus dependent upon the relative cost of weir length and of extra height of dam, and is largely a question of topography. Weir heights will ordinarily range from 2 to 4 or 5 feet, with lengths of 50, 100, or even 500 feet, or more, depending on the required capacity. In any case the flood line determines the height of the other part of the dam, while the weir crest determines the storage. The difference is the available depth of weir. For weir formulas see Chapter XII.

412. **Care of Floods during Construction.**—One of the most trouble-

some and expensive features of construction is the provision for passing the floods over or through the works. At the start an artificial channel or flume can readily be constructed at one side of the valley, and the culvert or outlet-pipe put in place, if there be such. The ordinary discharge and moderate floods can then be passed through this. Heavy floods may be allowed to pass over an uncompleted masonry weir or be carried over the embankment at a point protected by timber aprons.

In constructing the Titicus dam of the Croton Water-supply the river was first turned into an artificial channel by means of a temporary dam 24 feet high and about 1000 feet above the main dam. Afterwards a timber flume was used having two compartments, each 9 feet by 7 feet 9 inches, placed 25 feet above ground where it crossed the dam. After the dam was raised above the flume, the water was turned into the gate-house and discharged through two outlet-pipes 48 inches in diameter. Extreme floods were allowed to pass over the uncompleted portions of the masonry wasteway at a low point left for the purpose. It was considered that the damage thus caused was less than the expense of constructing a flume large enough to carry the water. Two heavy freshets were thus taken care of. The tributary area was 22.8 square miles.*

413. Cost.—The cost of reservoir embankments when constructed in the usual way will range about as follows: Excavation 25 to 30 cents per cubic yard; embankment 30 to 40 cents; puddle 50 to 75 cents; dry paving $2.00 to $3.00; riprap $1.50 to $2.00; sodding 20 to 30 cents per square yard. For the cost of various classes of masonry see Art. 449 of the next chapter.

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(See also Chapter XXVII.)

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CHAPTER XVII.

MASONRY DAMS.

THE DESIGN.

414. General Conditions.—Dams of masonry can safely be built only upon very firm foundations. Low dams of a height of 20 or 30 feet, and occasionally higher, have been founded on firm earth, but high masonry dams should be constructed on nothing less substantial than solid rock. In any case it is necessary to prevent practically all settlement, for with a material such as masonry any appreciable settlement is quite certain to cause cracks. Given, however, a firm foundation, a masonry dam is much superior to an earthen embankment in several respects. Its design can be more certainly and precisely determined upon; it is more durable; outlet pipes and conduits can be constructed through it with much greater safety; and, when properly designed, flood-waters may pass over it without danger to the structure. For very high dams, such as those above 100 feet, masonry is much to be preferred. As regards economy, the masonry dam may even be cheaper in some cases than one of earth, this question depending mainly upon the convenience of obtaining suitable material.

Earthen dams are largely designed according to empirical rules, but with a solid material such as masonry it is possible to apply to a considerable extent the principles of mechanics in determining the proper forms. Moreover, as masonry is a relatively expensive material, it is very desirable for the sake of economy to make the theoretical investigation as thorough as is consistent with the accuracy of the data.

415. The External Forces Acting upon a Dam.—Dams are built either straight or curved in plan. In the former case, it is assumed that all forces act in a plane perpendicular to the dam, and that the dam resists by gravity alone; in the latter case, arch action may exist to a greater or less extent, thus involving other than normal forces. The first case only will be here treated, and it will be sufficient to consider a length of dam of one unit. (For a discussion of the curved form see Art. 433.)

The principal external forces acting upon an impervious dam, $ABCD$ (Fig. 87), resting upon an impervious base are, the water-pressure P, the weight of the masonry G, and the reaction R. In addition to these forces, certain others require consideration, such as ice and wave pressure near the top, wind pressure, and back pressure of water on the side BD. Furthermore, if the dam or foundation is more or less porous, a certain amount of uplift will exist to reduce the effective weight G, as shown in Art. 378. However, with good mortar joints and good material for a foundation this uplift will be very small. It will for the present be neglected, as is the usual practice, but this and the other forces mentioned will be considered later.

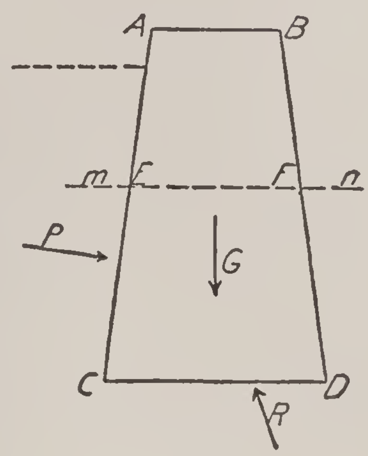

FIG. 87.

Assuming only the three forces P, G, and R as acting, they are all readily determined for any given section.

416. Internal Stresses. — In order to investigate the internal stresses, pass any horizontal section mn through the dam and consider the portion $ABEF$ (Fig. 88). Represent here the external forces by P and G, quantities readily determinable; and the internal stresses on the section which are necessary for equilibrium, by the two components V and H. The resultant of all vertical stresses on the section, tension and compression, is thus represented by V, and H is the resultant shear. The distribution of these direct and shearing stresses is yet to be determined.

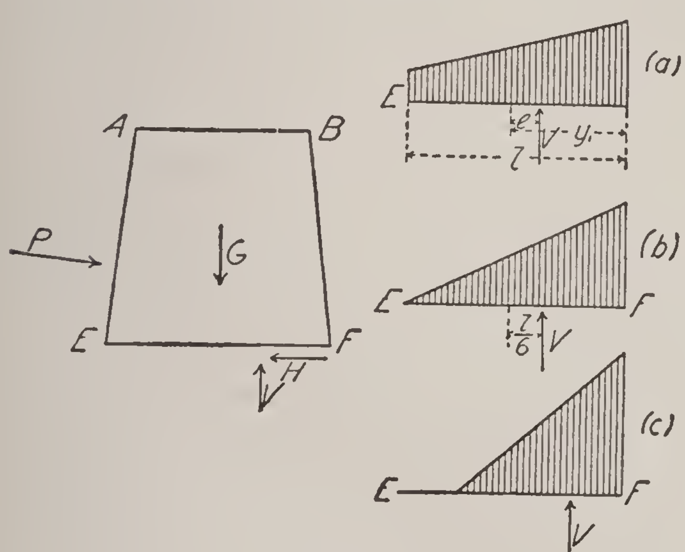

FIG. 88.

417. *Ordinary Assumptions as to Stress Distribution.* — As regards V it is assumed, first, that the stress varies uniformly across the section, as in the ordinary theory of beams, retaining-walls, etc. If e is the eccentricity of V (distance from the centre of EF), the stress upon the section may be considered as due to a compression V, uniformly distributed, plus a stress due to the moment Ve. The maximum compressive stress will then be at F, and, as in a beam, will be equal to

$$f_{\text{max.}} = \frac{V}{l} + \frac{My_1}{I} = \frac{V}{l} + Ve \cdot \frac{\frac{l}{2}}{\frac{1}{12}l^3} = \frac{V}{l}\left(1 + \frac{6e}{l}\right). \quad . \quad (1)$$

At E the stress would be equal to

$$f_{\text{min.}} = \frac{V}{l}\left(1 - \frac{6e}{l}\right). \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

So long as e is less than $\frac{l}{6}$, or the resultant V remains within the middle third of l, compression exists at all points and the distribution of stress is as represented in Fig. (a). If $e = \frac{l}{6}$, then the stress at E is zero, and at F is $\frac{2V}{l}$, or double the average, as in Fig. (b). If e is greater than $\frac{l}{6}$, then by the formula there would be tension at E. In masonry structures the tensile strength is not to be relied upon, and it is therefore assumed that there is no tensile stress. The distribution would then be as shown in Fig. (c), and the entire load would be carried by a length of joint equal to $3\left(\frac{l}{2} - e\right)$. The stress at F would then be $\dfrac{2V}{3\left(\frac{l}{2} - e\right)} = \dfrac{V}{l}\left(\dfrac{4}{3 - 6\frac{e}{l}}\right)$.

The shear H is not usually considered except as requiring a sufficient frictional resistance along the plane EF.

418. *Errors Arising from the Ordinary Assumptions.*—There are two sources of error in the above method of treatment. One is due to the assumption that the maximum intensity of stress is in a vertical direction. It will really be inclined, and greater than as above figured, its amount and direction at any point depending upon the intensities of V and H at that point. It will vary from the vertical direction both on account of the inclination of the resultant of V and H (R), and on account of the inclination of the exterior faces of the dam. At the points E and F the direction of the maximum compressive stress must be parallel to the respective faces, while at intermediate points it will vary between the two extremes. Even were the material homogeneous it would be impossible to determine these compressive stresses and their direction, but the lines of maximum pressures would probably be somewhat as shown in Fig. 89.

To allow for the inclination of R some writers use, instead of V, the force R, and consider it as acting on a plane equal in length to the projection of EF parallel to R. This is equivalent to using $\dfrac{V}{\cos^2 \alpha}$ in place of V as above, where α is the angle of inclination of R with the

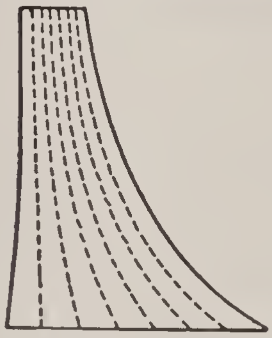

FIG. 89.

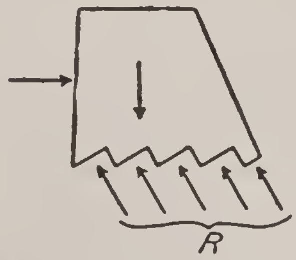

FIG. 90.

vertical. This assumes that the resistance of the masonry is due entirely to compressive stress on the inclined surfaces perpendicular to R (Fig. 90), and neglects the shearing stress on the other surfaces. It also does not take into account the effect of the inclined faces of the dam in varying the direction of the internal stresses, which alone would make the local stresses inclined at the faces even though the resultant R be vertical. The most common method of treatment is to use V only, and to allow something for the greater intensities of stress in an inclined direction by using a lower working intensity for the down-stream face.

The other error in the ordinary theory is the assumption that the pressures are uniformly varying. For a section like a high masonry dam the greater length of the outside toe renders that portion somewhat more elastic than the other part, thus tending to reduce the stresses at this point and to increase them elsewhere. Whatever this effect may be, it is on the side of safety.

419. **Conditions of Stability.**—Considering any section EF, as in Fig. 88, the conditions usually imposed to secure stability are three:

(1) The maximum compressive stress due to V at F or at E shall not exceed safe limits.

(2) There shall be no tensile stress at any point of the section. This requires, as shown in Art. 417, that the resultant of V and H or of P and G shall not cut the section outside its middle third.

(3) The resistance to shearing or sliding shall be greater than the total horizontal force at the level of the joint.

Another condition is sometimes stated as an independent one, namely, that the dam shall not overturn; but if condition (2) is met, there can be no possibility of overturning. When the resultant pressures for reservoir full and reservoir empty both cut the edge of the middle third, the factor against overturning is 2. All of the above conditions must be fulfilled at the foundation as well as at all sections of the structure, and if the foundation material is less strong than the masonry of the dam this must be allowed for under (1) and (3).

Besides these conditions of stability that of imperviousness is of course understood, although this requirement is not absolute, but merely relative.

420. *Resistance to Shearing or Sliding.*—To fail by sliding on a horizontal joint, or a succession of horizontal and vertical joints, the cohesion of the mortar must be overcome as well as the friction. The latter is, in the body of the dam, nearly always more than sufficient for stability. That the cohesion of the mortar is also to be largely counted upon is shown by the stability of numerous concrete dams, where the mortar must not only resist shearing on a horizontal plane, but on planes inclining outwards and downwards, on which the intensity of stress is much greater and the friction much less. By limiting the compressive stress we at the same time provide for shearing stresses, and they need not be further considered.

At the base of a dam the sliding tendency must be well looked after. Where the foundation is clay, or perhaps a timber platform, special precautions will be needed. Table No. 59 contains coefficients of friction which will be useful in this connection. They are from Baker, Fanning, and others. With rock foundations the conditions will be similar to those in the body of the structure if the masonry be well bonded to the bed-rock.

TABLE NO. 59.

COEFFICIENTS OF FRICTION OF VARIOUS MATERIALS.

Material.	Coefficient.
Granite (roughly worked) on gravel and sand (wet)	0.41
Pine (sawed) on gravel and sand (wet)	0.41
Granite (roughly worked) on sand (dry)	0.65
" " " " " (wet)	0.47
Masonry, on clayey gravel	0.577
" " dry clay	0.510
" " moist clay	0.325
Point-dressed granite (medium) on like granite	0.70
" " " " " common brickwork	0 63
" " " " " smooth concrete	0.62
Fine cut granite (medium) on like granite	0.58
Dressed hard limestone (medium) on like limestone	0 38
" " " " " brickwork	0.60
Beton blocks (pressed) on like Beton blocks	0.66
Common bricks on common bricks	0.64
" " " dressed hard limestone	0.60

421. Allowable Pressure.—This will depend upon the kind of masonry adopted. With large rubble masonry, as ordinarily employed, the safe pressures are taken all the way from 8 to 15 tons per square foot. With first-class concrete a pressure of 8 to 10 tons may be used. Many of the existing high dams sustain maximum pressures equal to the latter figures, and several exceed 14 tons. The Vyrnwy Dam has a maximum pressure of 8.7 tons. The Quaker Bridge Dam was designed for a maximum of 16.6 tons, and the new Croton Dam has practically the same profile. The Periyar Dam, Madras, of concrete, sustains 8 tons. The San Mateo concrete dam sustains a pressure, reservoir full, of about 7.7 tons, and 10 tons with reservoir empty.

422. Weight of Masonry.—The specific gravity of rubble masonry or good concrete is usually taken at from $2\frac{1}{4}$ to $2\frac{1}{2}$, corresponding to weights of 140 and 156 pounds per cubic foot. The latter value was adopted in designing the Quaker Bridge Dam, experiments giving 156.5 pounds. In the Sweetwater Dam it was estimated at 164 pounds, the stone being very dense and joints narrow. Concrete blocks cut out of the Vyrnwy Dam had a specific gravity of from 2.48 to 2.55.

A. *Stability of Low Dams.*

423. Conditions of Stability.—Dams up to 30 or 40 feet in height are usually made trapezoidal in form, the saving obtained by making the faces curved or broken not being enough to justify the extra trouble. For such low dams the only condition of stability requiring consideration, besides that of friction on the base, is that of tension in the joints. This requires that the resultant pressure shall keep within the middle third; for economy, it should just cut the edge of the middle third.

424. Calculation of Section.—Let $ABDC$, Fig. 91, be a section of a trapezoidal dam. Let the dimensions be as represented in the figure. Further, let $w =$ weight of a unit volume of water, and w' the weight of a unit volume of masonry. Let $g =$ specific gravity of the masonry $= \frac{w'}{w}$. The components of the water-pressure are P_v and P_h.

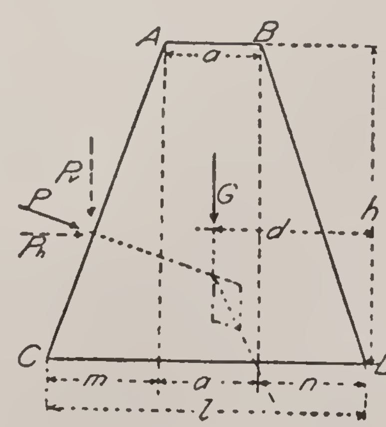

FIG. 91.

For dams of this class there will usually be but two cases: first, when the front face BD is vertical or nearly so, and, second, when the back face AC is vertical, or is given a definite small batter. In the first case n is assumed and l or m is required; in the second case m is given to find n or l. The problem is to find a value of l such that the resultant of P and G will just cut the edge of the middle third. The bottom section will be the dangerous one, and the only case to be considered is for reservoir full.

It is assumed for safety that the water rises to the top. The value of P then equals $wAC\frac{h}{2}$; its point of application is $\frac{h}{3}$ above the base. The value of G can readily be expressed in terms of w' and the dimensions, and its line of action found by rules of mechanics.

We have then, briefly,

$$P_h = \frac{wh^2}{2}; \quad P_v = \frac{wmh}{2}; \quad G = w'\frac{a+l}{2}h.$$

By dividing the moment about D of the several partial areas of the section by the total area, we find

$$d = \frac{\frac{2}{3}n^2 + a^2 + 2an + \frac{1}{3}m^2 + am + mn}{a+l}.$$

Equating now to zero the moments of P_h, P_v, and G about the outer edge of the middle third, we have, for stability,

$$P_h \cdot \frac{h}{3} - P_v\left(\tfrac{2}{3}l - \frac{m}{3}\right) - G\left(d - \frac{l}{3}\right) = 0.$$

Substituting in this equation the values of the forces given above, and the value of d, we get an expression containing l, m, and n; but noting that $l = a + m + n$, we can eliminate either n or m.

We thus have for the case where n is given, putting $\frac{w'}{w} = g$,

$$l = \sqrt{h^2 - (a+n)^2(g-1) + n^2g + \frac{n^2g^2}{4}} - \frac{ng}{2}. \qquad (3)$$

If the front face is vertical, $n = 0$, and we have

$$l = \sqrt{h^2 - a^2(g-1)}. \qquad (4)$$

For the second case, having m given, we derive the expression

$$l = \sqrt{A + B^2} - B, \qquad (5)$$

in which

$$A = a^2 + 2am + \frac{h^2}{g} + \frac{m^2}{g},$$

and

$$B = \frac{m}{g} - \frac{m}{2} + \frac{a}{2}.$$

If the back face is vertical, $m = 0$, and we have

$$l = \sqrt{\frac{5}{4}a^2 + \frac{h^2}{g}} - \frac{a}{2}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

If $a = 0$, eq. (4) gives $l = h$, and eq. (6) $l = \frac{h}{\sqrt{g}}$. It will thus give a more economical design to have the back face vertical or nearly so, rather than the front face. The latter form is, however, sometimes used for very low dams where designed as weirs, or where built in connection with such weirs. The front usually has then a batter of 1 to 2 inches per foot, and the rear whatever is necessary to give stability.

If the pressure on the base is desired, it can be found by the equations of Art. 417, using for V a force equal to the resultant of G and P_v. The force tending to slide the dam on the base is P_h. The frictional resistance is $V \times$ coefficient of friction. If a section should be given and it is desired to investigate its stability, the value of the resultant pressure and its point of application can best be found by graphics.

B. *Stability of High Dams.*

425. General Statement of the Problem.—For dams exceeding 30 or 40 feet in height, it is economy to build the lower face in the form of a curve or broken line. In designing a curved profile a certain height is soon reached, when it becomes necessary to investigate the stability of the dam for reservoir empty, and at a still greater elevation the condition that the stress shall not exceed a certain maximum value becomes the controlling factor. The design of the cross-section is therefore a somewhat complicated problem, and it is impossible to represent by a formula a profile which will exactly fulfil all the conditions.

Various formulas and methods of designing a profile have been proposed from time to time, differing more or less, but most of them based on the requirements for stability enumerated in Art. 419. Probably as simple a method as any is that adopted by Wegmann as the result of his studies for the Quaker Bridge Dam. The method is a general one and will be here briefly stated, and the working equations as derived by Wegmann will be given.*

* For a full discussion of the subject see Wegmann's "Design and Construction of Dams," New York, 1907.

426. Wegmann's Method of Determining the Profile.—This method consists in determining at successive horizontal sections, beginning at the top, the necessary width of section to fulfill the conditions stated in Art. 419, assuming the area enclosed between adjacent sections to be trapezoidal. In this way by taking the sections sufficiently close the profile may be determined with any desired degree of exactness.

As it is necessary that a dam shall have a certain top width, a, the upper portion will consist of a rectangle until such a depth is reached as to bring the line of pressure with reservoir full at the outer edge of the middle third. Below this point the down-stream face will be battered and the other face will be continued vertically downwards until the resultant with reservoir empty just cuts the inner edge of the middle third. The inner face will then begin to receive a slight batter. Finally, a depth will be reached below which the length of the joint will be determined by the limiting pressures at the edges.

The water is assumed to rise to the top of the dam in order to provide for extreme conditions. Furthermore, at the lower sections of the dam, where the back face becomes slightly inclined, the vertical component of the water-pressure is neglected. The error arising therefrom is slight except in very high dams and where the allowable pressure is low, and is on the side of safety.

427. The calculation of the profile is divided into five different stages, corresponding to the different sets of conditions to be met.

First Stage.—Depth of rectangular portion. The depth at which the line of the resultant pressure will cut the edge of the middle third of a rectangle may be found by making $l = a$ in equation (6), page 381, and solving for h; we get

$$h = a\sqrt{g}, \quad . \quad . \quad . \quad . \quad (7)$$

where h = height or depth, a = top width, and g = specific gravity of the masonry.

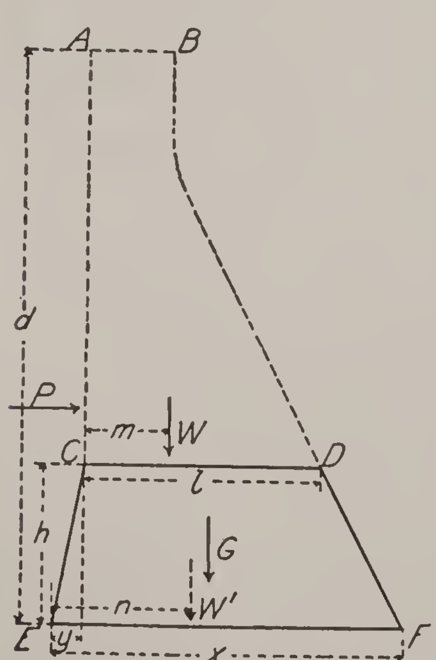

FIG. 92.

Second Stage.—The back to be continued vertical and the front battered. The section from here down is determined by considering successive trapezoidal blocks. In Fig. 92 let $CDFE$ be such a trapezoidal section of small thickness h situated immediately beneath the portion $ABDC$ already designed, which portion may be of any form, but whose weight, area, etc., are known. The following notation will be used:

W = weight of portion $ABDC$;
G = weight of portion $CDFE$;
W' = resultant of W and G;
A = area of $ABDC$;
A' = area of $CDFE$;
m = distance of line of action of W from C;
n = distance of line of action of W' from E;
l = length of joint CD;
x = required length of joint EF;
y = batter of CE;
h = thickness of section;
d = depth of water at E = height of dam above this point;
p = limiting intensity of pressure at F;
q = limiting intensity of pressure at E, usually greater than p;
w = weight of a cubic unit of water;
w' = weight of a cubic unit of masonry;
g = specific gravity of masonry $= \dfrac{w'}{w}$.

The value of x, then, so long as CE can be made vertical, is given by the equation

$$x = \sqrt{B + C^2} - C, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

in which $B = \dfrac{d^3}{gh} + \dfrac{6Am}{h} + l^2$, and $C = \dfrac{1}{2}\left(\dfrac{4A}{h} + l\right)$.

The value of n is given by the equation

$$n = \frac{(x^2 + lx + l^2)\dfrac{h}{6} + Am}{A + A'}. \quad . \quad . \quad . \quad . \quad . \quad (9)$$

Equation (8) can be used so long as n is greater than $\dfrac{x}{3}$.

In treating the next trapezoidal section the portion $ABFE$ is now the known portion, the various properties of which are to be substituted for like properties of $ABDC$ in the above equations. Thus the new value of m is n of eq. (9), etc.

Third Stage.—For the next series of courses the face CE must be battered so that n shall always be equal to $\dfrac{x}{3}$. The value of x is given by the equation

$$x = \sqrt{\frac{d^3}{gh} + \left(\frac{l}{2} + \frac{A}{h}\right)^2} - \left(\frac{l}{2} + \frac{A}{h}\right), \quad . \quad . \quad . \quad (10)$$

and the value of y is

$$y = \frac{2A(x - 3m) - hl^2}{6A + h(2l + x)}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

Fourth Stage.—When by the use of (10) and (11) the value of the pressure on the front face would exceed p, the formula is

$$x = \sqrt{\frac{wd^3}{p}}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

This value of x is to be used as soon as it becomes larger than the value given by (10). The batter is still given by (11); also, $n = \frac{x}{3}$.

Fifth Stage.—When the pressure on the back face becomes equal to q, then the formula is

$$x = \sqrt{D + E^2} + E, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (13)$$

in which $D = \frac{1}{g} \cdot \dfrac{d^3}{\dfrac{p+q}{w'} - h}$, and $E = \dfrac{A + \dfrac{lh}{2}}{\dfrac{p+q}{w'} - h}$, and the batter is

$$y = \frac{A(4x - 6m) + lh(x - l) + x^2\left(h - \dfrac{q}{w'}\right)}{6A + h(2l + x)}. \quad . \quad . \quad . \quad (14)$$

Equation (13) is to be used when it gives a value of x greater than that found by eq. (12). For this case, $n = \frac{2}{3}x - \frac{1}{6} \cdot \dfrac{qx^2}{w'(A + A')}$.

The foregoing equations are all that are needed in designing the profile of any high dam. In fact equations (12), (13), and (14) will not be used until a height of 100 feet or more is reached, depending upon the assumed values of p and q.

Graphical methods of determining lines of pressures, and of checking the results found by algebraic processes, will readily suggest themselves to the student.

428. Effect of Approximations in the Foregoing Treatment.—The effect of neglecting the vertical component of the water-pressure on the inclined upper face is very small until the height becomes very great. Then this additional component acts to throw the resultant nearer the upper face and therefore to increase the pressures near this face and to decrease those near the lower face. In the last respect it tends to compensate for the error due to considering vertical forces only. The effect is greater the lower the allowable pressure intensities.

The effect of neglecting the inclination of the resultant pressure on any section is of course to derive a pressure less than the actual. To take account of this where the pressure determines the profile, the value of p in eqs. (12) and (13) may be reduced in the ratio of $\cos^2 \alpha$ to 1, α being the inclination of the resultant with the vertical. The value of p can thus be readily varied to accord with the change in α as the design proceeds. By making p constant and somewhat lower than q, as is done by Wegmann, the effect of inclined resultant can be approximately allowed for.

429. Use of a Standard Profile.—Fig. 93 represents Wegmann's "practical profile No. 2," constructed for a dam 100 feet high without reference to pressures and with some simplification of the theoretical outline. The value assumed for g was $2\frac{1}{3}$, corresponding to a weight of masonry of 145.8 pounds per cubic foot. Such a section when once calculated can be used for any height of dam so long as the safe pressures are not exceeded, by simply cutting off a dam of the desired height from the standard section, or by changing all the dimensions proportionately, or by both processes, as may be necessary to secure the required top width. If the safe pressures are exceeded, eqs. (12), (13), and (14) will have to be made use of for the lower sections.

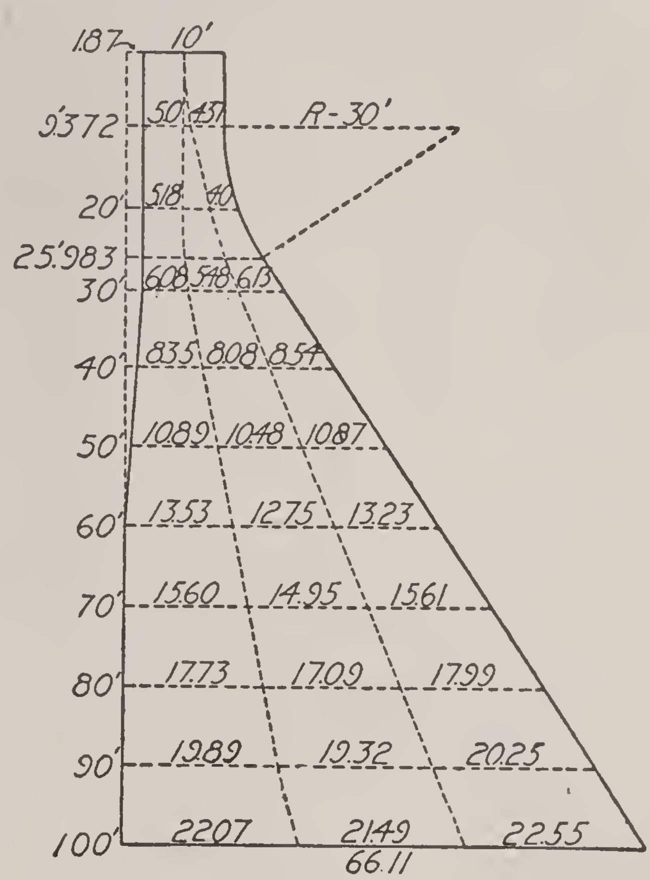

FIG. 93.

If, for example, a profile is required for a dam 50 feet high and with 8 feet top width, proceed as follows: Get by proportion, from Fig. 93, a profile with top width 8 feet and height 80 feet, and then use the upper 50 feet of such profile. For masonry with a different specific gravity a new standard profile would have to be calculated.

The pressures at the various depths for the profile of Fig. 93 are given in Table No. 60, the data for which are from Wegmann. In dams of other heights but in which the dimensions are proportional, the pressures will also be proportional. Thus in a dam 120 feet high, made similar to Fig. 93 (top width 12 feet, bottom width $= 66.11 \times \frac{120}{100} = 79.332$ feet), the pressure at the bottom will be $7.16 \times \frac{120}{100} =$ 8.92 tons. At a point 80 feet below the top the pressure will be proportional to that given for the 100-foot dam at a point $= \frac{80}{120} \times 100 =$ 66.7 feet from the top, $= 4.9 \times \frac{120}{100} = 5.88$ tons per square foot. For a dam 150 feet high the maximum pressure is $7.16 \times 1.5 = 10.74$ tons per square foot. With safe values of 8 to 10 tons per square foot as commonly used it is seen that a standard profile such as here given would be suitable for dams up to about 150 feet in height.

TABLE NO. 60.

PRESSURES FOR WEGMANN'S PRACTICAL PROFILE NO. 2. (FIG. 93.)

Distance from Top of Dam in Feet.	Pressures in Tons per Sq. Foot.		Distance from Top of Dam in Feet.	Pressures in Tons per Sq. Foot.	
	Down-stream Face, Reservoir Full.	Up-stream Face, Reservoir Empty		Down-stream Face, Reservoir Full.	Up-stream Face, Reservoir Empty
9.372	0.95	0.68	60	4.45	4.35
15	1.84	1.69	65	4.78	4.73
20	2.52	1.77	70	5.11	5.11
25.983	2.77	2.45	75	5.45	5.48
30	2.80	2.82	80	5.78	5.86
35	2.97	3.06	85	6.13	6.22
40	3.23	3.30	90	6.48	6.59
45	3.51	3.55	95	6.82	6.96
50	3.81	3.81	100	7.16	7.33
55	4.13	4.08			

430. Approximate Triangular Profile.—A profile very closely approximating the type illustrated in Fig. 93 can be quickly determined from any assumed data as follows: Assume first the triangular profile ABC, Fig. 94, with vertical back face. For reservoir full the value of x is given by eq. (6), page 381, by putting $a = 0$ and replacing l and h by x and d respectively. It is $x = \frac{d}{\sqrt{g}}$. This value is proportional to d and hence the line of pressure, reservoir full, cuts the outer edge of the middle third at all sections. For reservoir empty the resultant evidently cuts the inside edge of the middle third at all points, so that until a depth is reached where the allowable pressures are exceeded the triangle exactly satisfies the conditions of stability and is the most economical form. A zero top width is, however, impracticable, and to get a practical profile the block AFK of width a is added. The effect of this block is slightly to disturb the positions of the pressure lines, but for high dams the

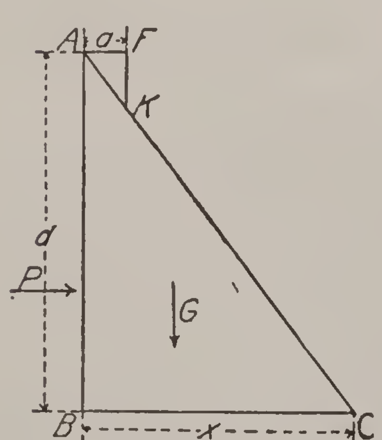

FIG. 94.

variation is so small as to be negligible. The line of pressure, reservoir full, is brought slightly within the middle third, while that for reservoir empty passes a very little outside. Such a profile, rounded off slightly at the point K, can therefore be used with practical exactness for dams of such height that the pressures need not be considered. If more exact methods are desired, this form may be used for preliminary plans. It also affords a ready check on more elaborate determinations.

The maximum pressure intensity, p, of the triangular profile is equal to $\frac{2G}{x} = w'd$. For a value of $w' = 145.8$ and $d = 150$, $p =$ 10.93 tons per square foot as compared to 10.74 tons for the profile of Fig. 93. The coefficient of friction necessary for stability against sliding is equal to $\frac{P}{G} = \frac{1}{\sqrt{g}} = .65$ for a value of $g = 2\frac{1}{3}$.

431. Forces not Considered in the Preceding Analysis.—As already remarked, the upward pressure of the water is usually neglected. In one dam built recently, the Gileppe Dam, 154 feet high, this action was allowed for, resulting in a greatly increased section; but the continued stability of many high dams in which this element is neglected indicates that it need not be taken into account.

It has been shown that with good mortar joints and good connection with bed-rock the uplift cannot possibly be more than a few pounds per cubic foot. For small areas near crevices where springs occur it might be very considerable, but such areas would in any case be but a small fraction of the whole. A system of drains such as used in the Vyrnwy Dam, page 405, would avoid all possibility of such action beyond that due to the head of water on the lower face. A dam founded on a loose porous foundation would of course be differently conditioned, but such would scarcely be a masonry dam.

There is usually a certain depth of water on the lower face. The pressure of this should be taken into account when this part of the section is reached; also any considerable unbalanced earth-pressure.

Wind-pressure, reservoir empty, will add slightly to the stresses, but the amount is not sufficient to be considered. Wave action will add something to the pressure of the water, but this may be considered as amply provided for in assuming the water-level at the top of the dam.

The pressure of ice is sometimes very great, but what allowance should be made for this is impossible to say. The maximum pressure would be measured by the crushing strength of ice, which may be taken at about 400 pounds per square inch. Such great pressures would doubtless seldom occur, but may be approached in confined locations for either a high or a low dam.* The pressures due to ice moved by the wind in the spring would be very much less and would correspond to a strength of ice of probably not over 30 or 40 pounds per square inch, perhaps 4000 or 5000 pounds per lineal foot for ordinary cases. In the case of the Quaker Bridge Dam it was the opinion of the board of experts that ice-pressure should be taken at 43,000 pounds per lineal foot. The effect of such a force can be taken account of by combining it with the horizontal pressure of the water. Usually a sufficient margin of strength to resist ice-pressure will be afforded by the mass of masonry above high-water line dimensioned according to empirical rules of practice.

432. **Top Width and Height above Water-line.**—If the dam is to be used as a driveway, the top width will have to be at least 8 feet besides width of parapets. Otherwise the width and height above high-water line must be such as to secure stability against wave and ice action as just noted, and to prevent waves from washing over the top. A formula for height of waves was given in the previous chapter (page 352). In practice the width varies from a minimum of 4 to 5 feet for low dams to 15 or 20 feet for very high dams; and the height above high-water line from 2 or 3 feet to about 10 feet. In some cases much larger dimensions may be required for low dams than those given.

433. **Curved Dams.**—*Arch Action alone Considered.*—Up to this point it has been assumed that a dam resists overturning by gravity action alone. Obviously if a short dam be built with a sharp curvature convex up-stream, with its flanks resting against rigid supports, overturning will also be resisted by arch action. In investigating the stresses of such a dam it may be looked upon as a section of a circular open well constructed in the middle of a reservoir. Omitting any resistance by gravity action and assuming each horizontal lamina to support the water-pressure against itself independently of the others, the horizontal compressive stress in a lamina 1 foot thick will be equal to wdr, where $w =$ weight of water, $d =$ depth of lamina below water-surface, and $r =$ radius of curvature of the dam. If this pressure be assumed as uniformly distributed over the cross-section of the lamina, the pressure per square foot will then be equal to $p = \frac{wdr}{t}$, where $t =$ thickness of wall at the depth d. For a constant value of p the thick-

* For account of failure of a dam at Minneapolis by ice-pressure, see *Eng. News*, 1899, XLI. p. 307.

ness t should vary with d, thus giving a triangular profile in which $t = \frac{wdr}{p}$. Taking $p = 10$ tons $= 20,000$ pounds, we have $t = \frac{62.5\,dr}{20,000} = .031\,dr$. The value of t for a gravity dam with triangular profile was shown to be equal to $\frac{d}{\sqrt{g}}$. Putting $g = 2\frac{1}{3}$, this becomes $t = .66\,d$. Theoretically, therefore, the two sections would be equal when $r = \frac{.66}{.0031} = 213$ feet. This rough calculation indicates that the only situation where a purely arch type can be economically considered is in a very narrow alley.

434. *Gravity and Arch Action.* — A curved dam with its base securely fastened to the foundation cannot wholly fail to resist by gravity. A gravity dam may be looked upon as a vertical cantilever beam which when loaded will deflect until certain internal stresses are developed sufficient to resist the load. (The vertical force of gravity produces a longitudinal compression in this beam and prevents any of the stresses from becoming tensile). If such a dam be now curved in plan, the downward deflection of the top will also be resisted by the circumferential stresses or arch action. The relative amounts of beam and arch action will be proportional to the rigidity of the two paths over which the load passes. Thus a massive dam of long radius would be very much more rigid as a beam than as an arch, and the arch action would therefore be very small. On the other hand a thin wall of short radius would be, especially towards the top, of relatively great flexibility as a beam, and such would be mostly supported by arch action. No curved dam will therefore resist wholly by arch action, nor by gravity or beam action.

It would theoretically be possible, by taking account of both actions, to design a curved dam section that would be less in area than either the gravity or the arch dam. However, the variation in length of dam from top to bottom, the variation in thickness and in the elasticity in different directions due to differences in compactness, are some of the elements that make the problem too uncertain and complicated to admit of this being readily done.

The Lake Cheeseman Dam, Colorado, is designed as a gravity dam with curved plan, the radius of curvature being 400 feet and the total height 225 feet, the lower 60 feet being in a very narrow gorge. Calculations of the relative amounts of gravity and arch actions, made by

Mr. S. H. Woodard, assuming for this purpose a height of 165 feet, gave results as follows : *

Depth below Top.	Percentage of Gravity Action.	Percentage of Arch Action.
15 feet.	53	47
45 "	90	10
75 "	94	6
105 "	97	3
135 "	99.8	0.2

435. *Methods Followed in Practice.* — In practice there are three methods followed : (1) to make the dam straight and therefore a gravity dam ; (2) to give the dam a sharp curvature when conditions will permit, and rely more or less on arch action ; and (3) to build a gravity dam in a curve, and consider any arch action as an additional element

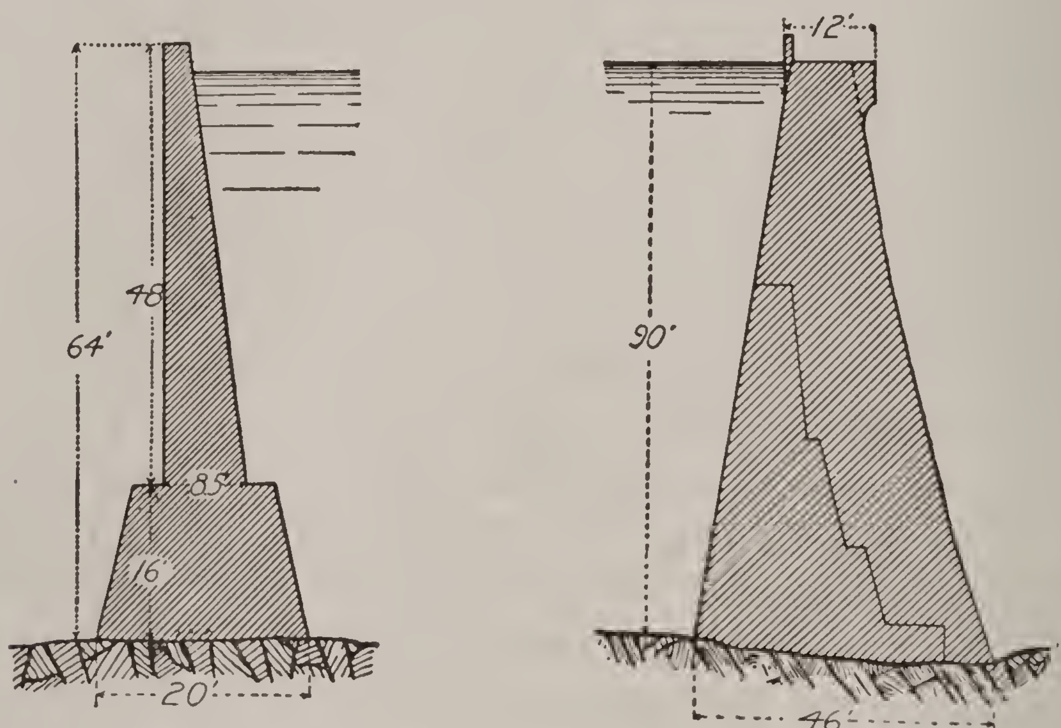

FIG. 95. — BEAR VALLEY DAM.

FIG. 96. — SWEETWATER DAM.

of safety. In the case of moderately short dams the third method is considered preferable by most engineers. For long dams, however, the advantage gained by using a curved plan of long radius would be very slight and not commensurate with the extra trouble and expense involved. For very short dams where radii of 200 or 300 feet can be used the second method may be employed. It is to be noted that in the bottom of a narrow valley where the thickness of a gravity dam is perhaps greater than its length, arch action may take place even in a straight dam.

436. *Examples.* — A few dams have been built in which the section is materially less than that required for gravity. The boldest of these is the

* For discussion of this subject and methods of calculation see paper on Lake Cheeseman Dam in Trans. Am. Soc. C. E., 1904, LIII. p. 89. The discussion contains descriptions and calculations for a unique arch, or "dome," type of dam at Ithaca, N. Y

Bear Valley Dam of California, illustrated in Fig. 95. The radius of the top is about 250 feet. It is built of uncoursed rubble. If it be assumed to act as a gravity dam, the resultant pressure would pass many feet outside the base. Calculated as an arch dam the pressures near the base are 40 tons or more per square foot. Another dam of this type is the Zola Dam in France. It is 123 feet high and 41.8 feet thick at the base and has a radius of curvature of 158 feet.

The Sweetwater Dam of California may also be considered of this type, although designed as a gravity dam (Fig. 96). Assumed as such, the line of pressure falls at about the middle of the outside third. It has a radius of curvature of 222 feet and undoubtedly acts partly as an arch. In 1895 it was overtopped for 40 hours by a high flood without injury, the water standing 22 inches above the parapet. It is built of uncoursed rubble, great care having been taken in executing the work. The masonry weighs about 164 pounds per cubic foot.*

The Barossa Dam in South Australia is a modern example of the arch type of dam. It is shown in section in Fig. 96a. The radius of the dam is 200 feet. By substituting this type for the gravity type a saving of about 50 per

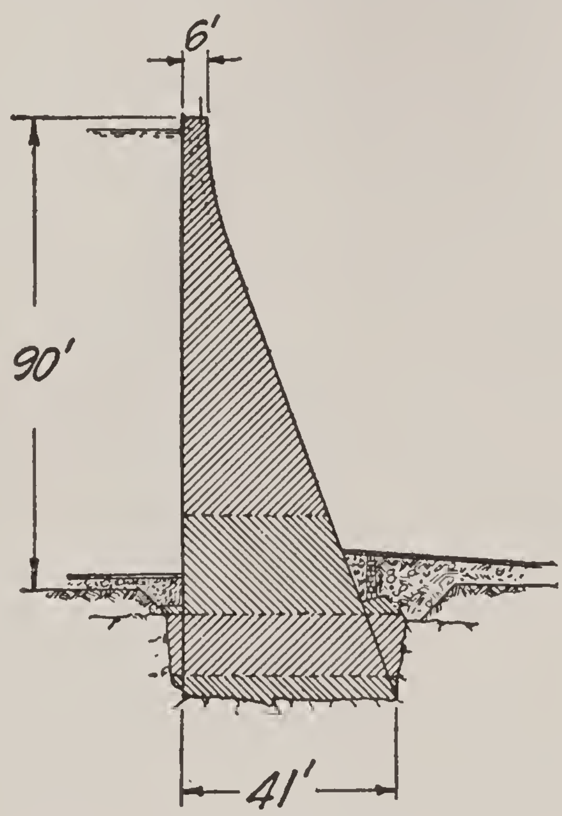

FIG. 96a.—BAROSSA DAM.

cent of the estimated cost was effected. Rubble concrete was employed in its construction. In the upper part of the dam several horizontal rows of 40-lb. steel rails were inserted to add strength and rigidity. Observations regarding movements, due to temperature changes, showed a movement of $\frac{7}{8}$ inch of the top, resulting from a change of 50° F.†

Another very bold arch dam is the Upper Otay Dam of the Southern California Mountain Water Co. Its maximum height is 84 feet and width of base 14 feet. The radius of curvature is 359 feet. It is of concrete, reinforced partly with steel cables and partly with steel plates.‡

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* Trans. Am. Soc. C. E., 1888, XIX. p. 201.

† *Eng. News*, 1904, LI. p. 321.

‡ *Ibid.* p. 326; *Eng. Record*, Nov. 1903, p. 590.

CONSTRUCTION.

437. The Foundation.—For large dams the foundation should be solid rock. In preparing the foundation surface all loose and partially decomposed material should be excavated until a firm base is reached. If the bottom is smooth it should be roughened by excavating shallow cavities in the rock. At points where crevices occur the excavation must be carried down to a solid bottom and all loose material must be removed. After an acceptable surface is reached it should be thoroughly washed or scrubbed with water in order that there may be a secure bond between the foundation and the masonry. Many engineers follow the practice of coating the prepared foundation with a layer of neat cement. The great care necessary in this part of the work is illustrated by the following specification relating to the construction of the concrete dam at Butte, Mont., Chester B. Davis, Mem. Am. Soc. C. E., engineer:

"Whenever the slope of the solid bed-rock of the dam-site makes a greater angle than 5° with the horizontal it must be rough-stepped by removing the least amount possible of bed-rock. Where the rock beneath the dam is smooth and free from cross-seams it must be made rough either by stepping or blasting holes with a superficial area of from 6 to 12 feet and a depth of from 1 to 3 feet.

"Each square foot of the natural bed-rock beneath the proposed structure, and to include an area to an elevation of 20 feet above the flow-line, and for at least 200 feet above and 100 feet below the upper and lower toes of the dam, must be carefully examined and everything not natural, true, and perfectly solid granite rock over this area be removed.

"Each crevice, joint, or other opening beneath the structure must be examined and tested and all material removed which would be started or stirred by a pressure up to at least the maximum load on the base or abutments, or by a minimum strain of 25 tons per square foot. Each crevice, joint, crack, or other opening must be filled with granite or concrete after completing the blasting for the portion of the dam where located. Openings outside the limits of the structure must be filled flush with the surface and rammed where possible until perfectly compact. Openings beneath the dam must be treated in the same manner, unless large enough to be properly filled with the concrete used for the base of the dam. In all cases these openings must first be grouted." *

* *Eng. News*, 1892, XXVIII. p. 554.

In building large dams the excavation for the foundation becomes a matter of considerable difficulty, especially where a great depth of earth overlies the rock. The excavation in such a case becomes very broad, and as a consequence is usually made with such slopes as to be self-supporting, no attempt being made to use bracing. Ample pumping capacity is here a prime requisite. At the New Croton Dam the foundation was 1300 by 500 feet by 130 feet deep. The stream was diverted by means of a temporary channel and large wing dams constructed above and below the excavation.

438. *Earth Foundations.*—Low dams of masonry are quite often founded on hard clay or even compact sand, a construction often made necessary where waste-weirs are placed in earthen embankments. In building upon such foundations great care must be observed to avoid overloading the material and to prevent seepage under the dam. Plank foundations are very commonly used to aid in distributing the load, and sheet-piling driven well into the foundation at the upper edge of the dam is of great value in reducing seepage.

A good example of a dam built on earth foundation is the one at Southington, Conn., shown in Fig. 97. In the construction of this

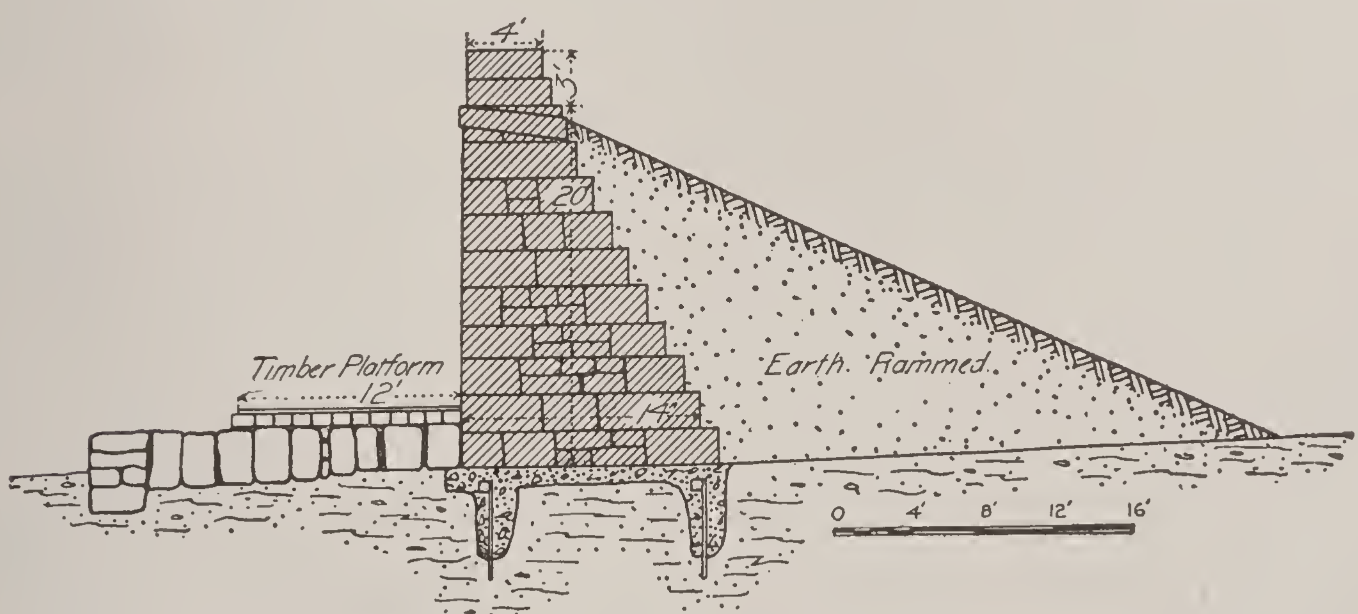

FIG. 97.—SOUTHINGTON DAM.

dam the bed of the stream, which was a very fine quicksand, was prepared by excavating two trenches parallel to the face of the dam and of a depth of about 3 feet. Sills were laid at the bottom and the top of the excavation, and sheet-piling driven and spiked to them. The trenches were then filled with concrete and the entire foundation covered with a layer of concrete 1 foot thick by 15 feet wide. The dam is built of granite rubble.* (See also description of Dunning's Dam, page 403.)

* Trans. Am. Soc. C. E., 1886, xv. p. 887.

439. *Percolation of Water beneath the Dam.*—It is quite frequently the case that considerable trouble is experienced from water seeping through at the foundation surface and appearing in the form of large or small springs. In handling these springs the same general methods are used as described for earthen dams. Great care must be taken to avoid water-pressure existing over any considerable area of the bottom of the dam, as such pressure is usually assumed not to exist. The great importance of this matter is apparent when we consider the excessive section used in the Gileppe Dam where full water-pressure was provided for. The failure of the Bouzey Dam is attributed to water getting into cracks caused by tension in the masonry due to a too narrow section.

If the water is present in large quantities, the most certain way of avoiding upward pressure is to lead the water out to the lower face of the dam, as was done for the Vyrnwy Dam. A French engineer, Maurice Lévy, has suggested the construction of a guard-wall in front of the dam and connected therewith by means of short buttresses. By this arrangement any water percolating through the wall could be readily drained out from the spaces between wall and dam. Percolation and resulting pressures are to some extent avoided by making the dam itself as impervious as possible, and also the foundation for some distance above the upper face of the dam.*

In preparing the foundation of the New Croton Dam the greatest care was exercised in removing all unsound material and in building over springs of water in such a way as to avoid as far as possible all upward pressure. The rock foundation was carefully scrubbed and all erosions and cracks were traced out by drilling numerous holes in their vicinity. Such cracks were usually piped and filled with grout forced in under pressure. Where a flow of water was encountered pipes were also led to an adjacent drain or sump and the water permitted to escape until the masonry had been built up for some distance. The pipes were then filled with grout. For a detailed description of this important work, see paper by C. S. Gowan in Trans. Am. Soc. C. E., 1900, XLIII. page 469.

440. **Construction of the Masonry.**—Uncoursed rubble or concrete is usually employed in dam construction. The object to be attained is to secure a homogeneous structure, free from all through joints or weak places of separation. Concrete, well placed, is in this respect an ideal material. Rubble masonry, in which all joints are thoroughly

* Mr. Freeman in his report on New York's Water-supply suggests the use of a thin sheet of lead placed vertically in the masonry a few feet back of the face.

filled with mortar, and larger spaces with concrete, has been used for most of the high dams. It is in fact a rubble concrete where the mortar is reduced to as small a proportion as possible. The material to be adopted in any case will be determined largely by the question of expense.

Rubble is often faced with broken-range ashlar. This adds strength to the face, but is objected to on the ground of its greater rigidity and therefore its tendency in settling to separate from the rubble backing. Such facing should be well bonded to the body of the structure. Several recent important dams, among which are the Nashua Dam and the New Croton Dam, have ashlar facing. In the Croton Dam the facing courses vary in size from 30 to 15 inches. The joints are not to exceed ½ inch for 4 inches from the face. In each course every third stone is to be a header, with a length of at least 4 feet. The stretchers are to be not less than 3 feet wide and not more than 7 feet long.*

Beds are as a rule made horizontal, except in the facing, but in the Remsheid Dam, completed in 1891, the joints were made to vary somewhat according to the line of pressure as shown in Fig. 98. Greater resistance against shearing is thus obtained.

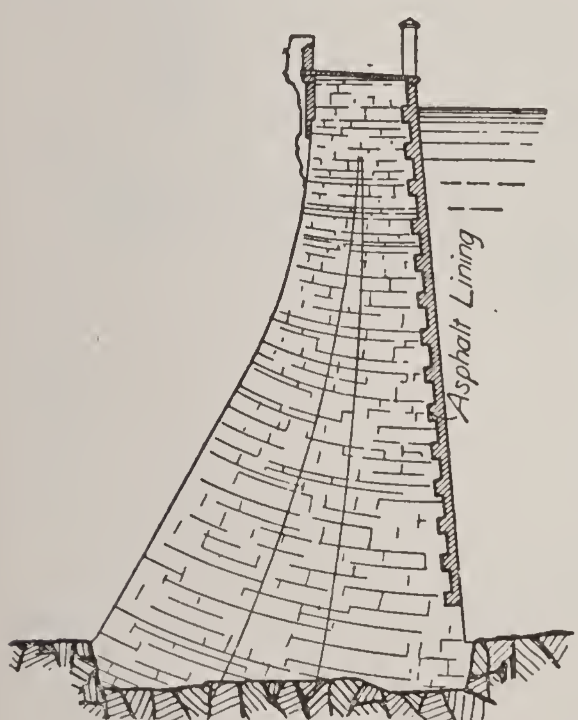

FIG. 98.—THE REMSHEID DAM.

Cement mortar should be made in such proportions as to be practically impervious, particularly near the up-stream face. Portland or Rosendale cement mortar 2 to 1, or Portland 3 to 1, is usually employed, but the last is not entirely impervious. It is desirable to use the stronger mortar where the heavier stresses exist and also near the faces.

The size of stone to be used in rubble masonry depends chiefly on the matter of convenience. In some of the modern dams stones measuring 6 to 8 cubic yards have been used. Large spaces are left between these which are filled with cement or with smaller stones and mortar.

In constructing the masonry the principal points to be emphasized are clean surfaces, irregular surfaces, joints absolutely filled with compact mortar, no grouting, great care to give good bedding, and constant supervision. Mortar and cement should be thoroughly rammed into all spaces, using for this purpose suitable forms of rammers.

* Wegmann. The Water-supply of New York, p. 207.

Concrete to be practically impervious should not usually have a greater proportion of sand and stone than that given by the mixture of 1 : 3 : 5. Larger proportions of stone have been used, however, with good results, such as 1 : $3\frac{1}{4}$: $7\frac{1}{2}$.* The greater the proportion of stone the better, as long as all voids are filled, but with high ratios of stone greater care is required in the manipulation. Close supervision in the mixing and laying is very necessary to secure a good concrete.

The water of streams is cared for during construction by methods similar to those described in the preceding chapter (page 370).

441. **Imperviousness.**—Imperviousness is very difficult to secure, and in fact most masonry dams leak slightly. That it can be practically obtained is, however, shown by the results reported in the case of several of the modern dams. The result in this respect depends chiefly upon the care taken in executing the work. Special precautions may, however, be used to good advantage, such as the use of a more impervious mortar near the up-stream face of the dam, or the plastering of the upper face with neat or 1-to-1 cement mortar. In the Remsheid Dam a continuous joint of asphalt was used just back of the face-stones and on the foundation surface for a short distance above the dam.

Whether cracks will necessarily form in dams is a disputed point. In some they have occurred and in some apparently not. In long narrow walls cracks are very sure to form, due to temperature changes, but in the massive walls of dams the changes in the interior are very slight, and it is undoubtedly true that in some of the modern dams at least, no cracking of the interior has occurred. In the Vyrnwy Dam the effect of temperature changes has been measured at a height of 80 feet. A maximum movement of 0.366 mm. due to variations in temperature from day to night has been noted.† In the Remsheid Dam, curved at 410 feet radius and 82 feet high, a movement of the crest of $1\frac{1}{16}$ inches, due to filling of the reservoir, and of $\frac{7}{8}$ inch, due to temperature changes, has been observed. The curved form was here considered to have prevented cracking.‡

442. **Earth Backing for Masonry Dams.**—In the construction of dams of moderate height, earth backing is often carried up to the water-level with a slope of 2 or 3 to 1, as in an earthen dam. Such a backing, if more porous than the dam, will not reduce the pressure

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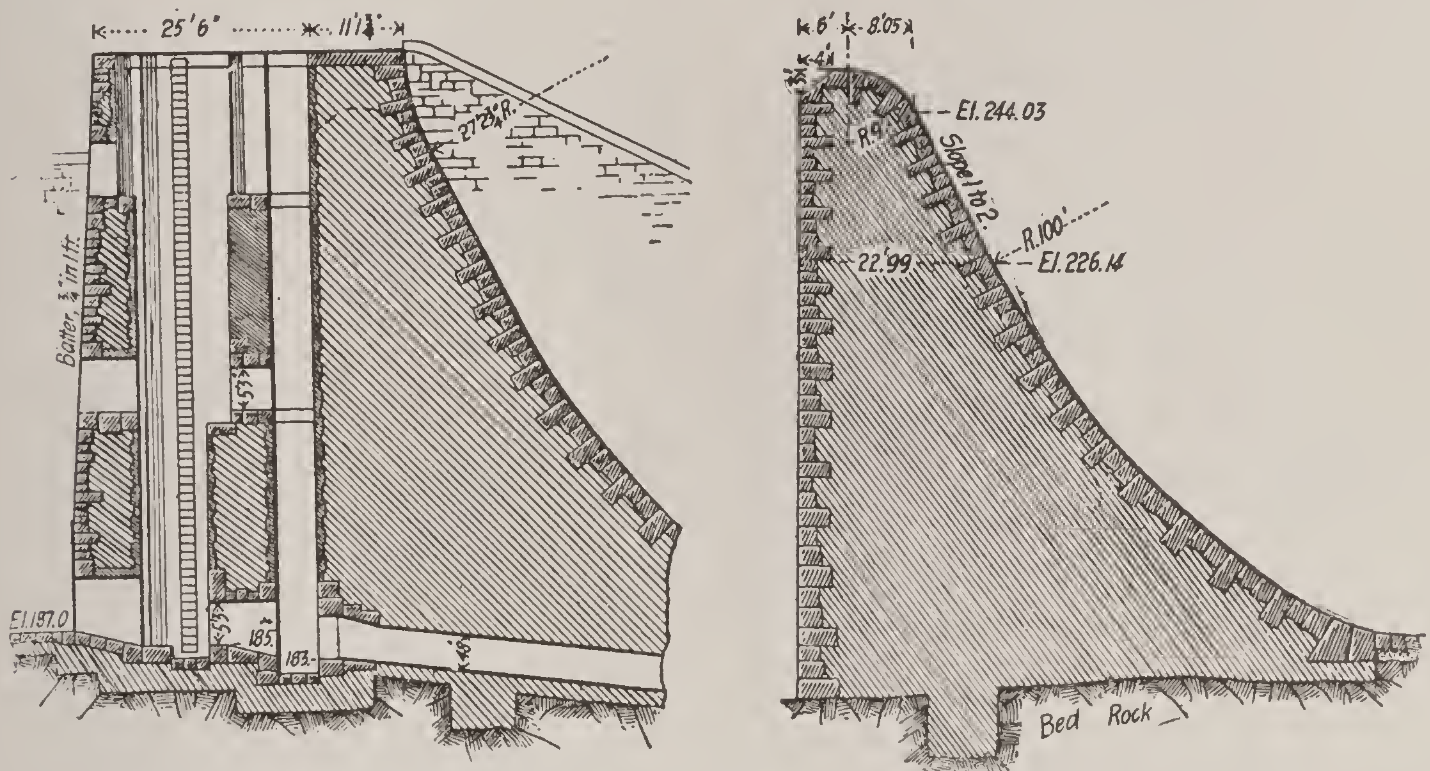
* See description of Indian River Dam in *Eng. News*, 1899, XLI. p. 310; also a paper by G. W. Rafter on the Theory of Concrete in Trans. Am. Soc. C. E., 1899, XLII. p. 104.

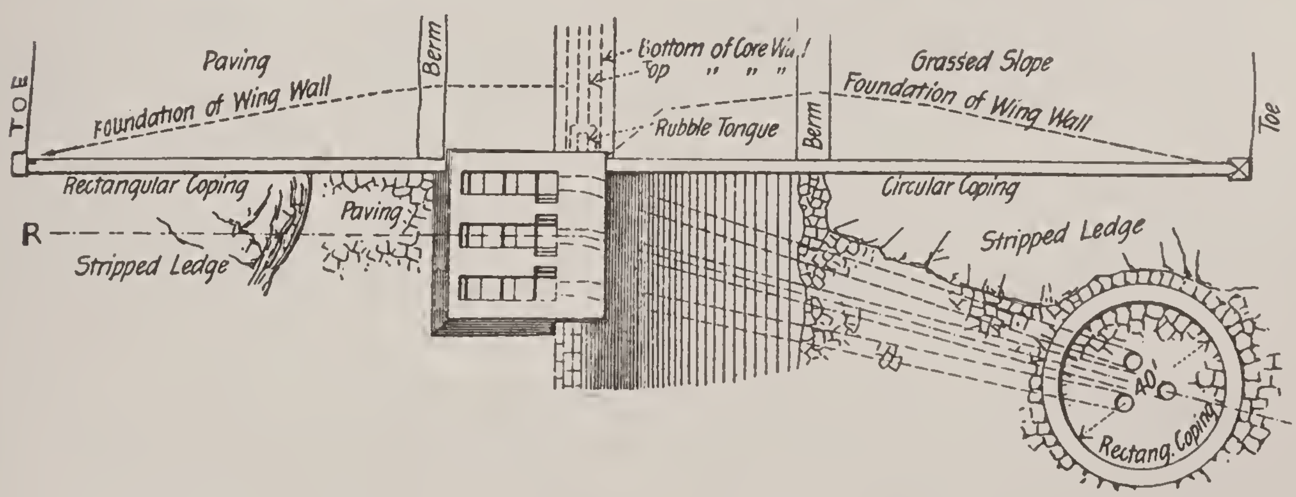
† Proc. Inst. C. E., CXV. p. 117.

‡ *Eng. News*, 1896, XXXV. p. 76.

against the wall, but will rather increase it and is ordinarily of doubtful advantage. If, however, a dam is located on a porous or bad foundation or on one of earth, a good, compact backing will much reduce the percolation under the dam, and therefore the tendency of any upward pressure, and will add considerably to the safety of the structure. It is especially applicable to spillways in earthen embankments. The earth backing in that case acts also as a protection for the back of the masonry against injury from ice and driftwood. (See Figs. 97 and 104.)



Section through Gate-house. Section through Waste-weir.



Plan of Gate-house and Wing Wall.

FIG. 99.—DAM NO. 5, BOSTON WATER-WORKS.
(From *Engineering News*, vol. XXXIII.)

443. Draw-off Arrangements.—The arrangements for drawing water from the reservoir are similar in general to those described in the last chapter. The outlet-pipes are built in the masonry at or near the lowest point of the dam, and terminate in a gate-chamber constructed just above and in connection with the dam. The gate-chamber has the same functions as explained in the case of earthern embankments. No danger is here to be apprehended from constructing the pipes in the body of the dam.

An outlet arrangement of common form is shown in Fig. 99, which illustrates details of Dam No. 5 of the Boston Metropolitan Waterworks. The figure shows the weir, gate-house, and wing walls at the junction of the earth embankment and masonry dam. The gate-cham-

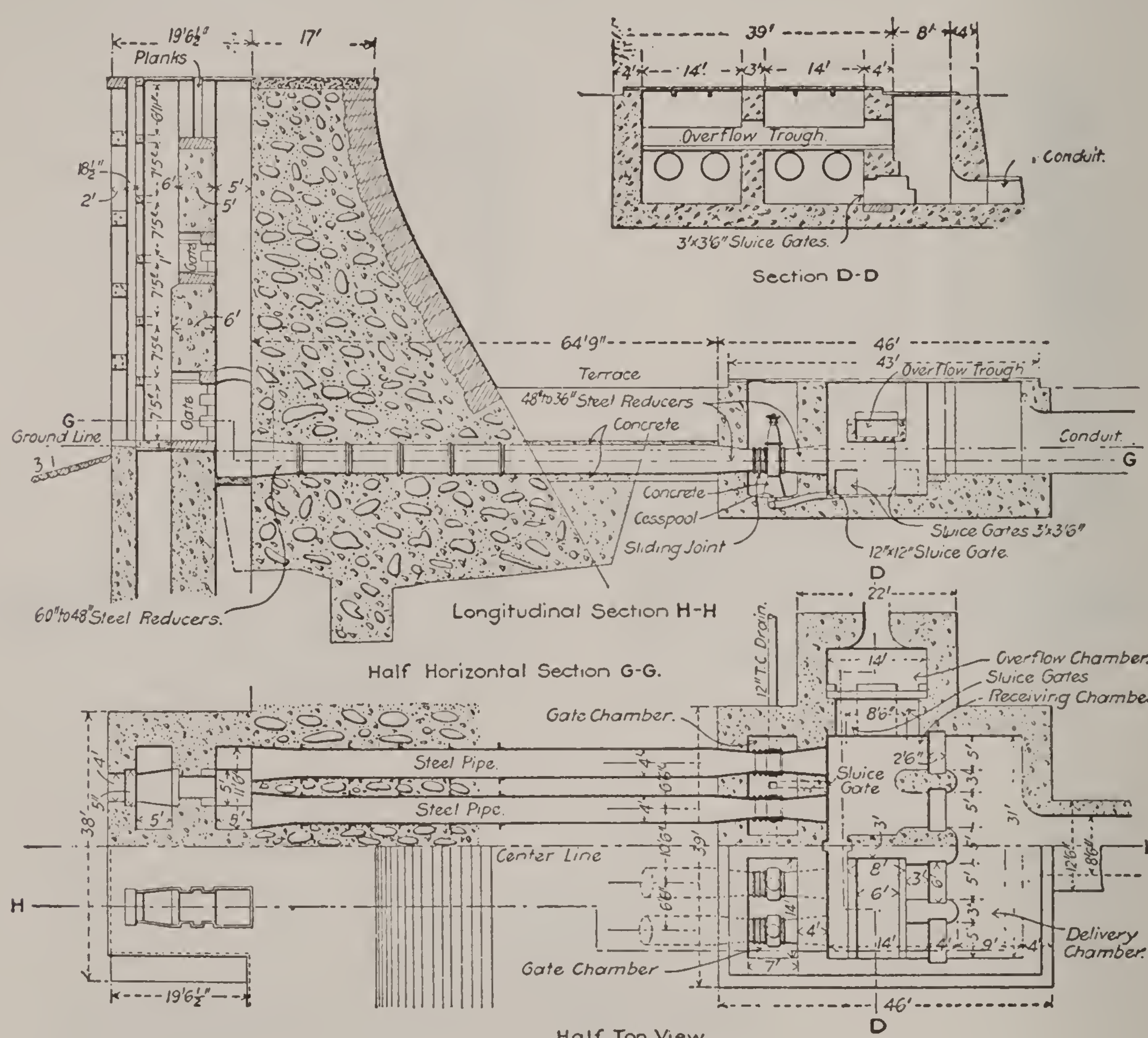


FIG. 99a.— THE BOONTON DAM.
(From *Engineering Record*, vol. XLIX.)

ber is very similar to those used in several of the dams of the New York Water-works. (For section of the earth embankment, see Fig. 74.)

Another very good example of gate-chamber and draw-off arrangements is shown in Fig. 99a. Notice the large steel outlet-pipes and reducers permitting the use of 36-in. valves on 48-in. pipes.

Simpler arrangements than the above may often be adopted to advantage. Thus if screening is not required, a single chamber answers every purpose. Even this is dispensed with in some cases, as for example, in the construction of the large dam at Butte, Mont., and more recently in the dam at Plymouth, England. In these cases the outlet-pipes pass through the dam and terminate in short vertical pipes just above the upper face. Cover-valves are fitted over the ends of these pipes and are operated by chains from windlasses above. Details of the valves used at Plymouth are illustrated in Fig. 100. As shown in the sectional elevation, the valve is made in three sections which are successively raised when the valve is opened. This form of construc-

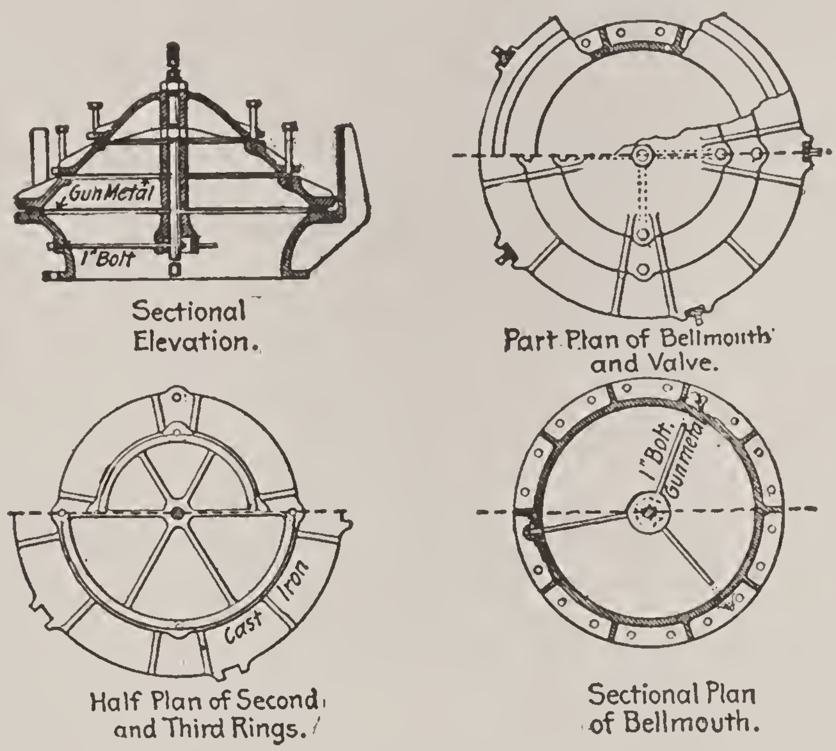


FIG. 100. — COVER-VALVES, PLYMOUTH RESERVOIR, ENGLAND.
(From *Engineering News*, vol. XLII.)

tion is best suited to the case where the valves need not be often operated. In the winter the ice would have to be kept cut away from around the chains or pipes.

Where a dam is built across a narrow valley a scouring-sluice or large waste-pipe placed at the lowest point will enable much of the silt deposit to be removed by flushing. These deposits may be prevented to some extent by building small barricades or dams at the entrance of the various streams into the reservoir, thus forming small settling-basins which may be more readily cleaned than the large reservoir. Flood-channels are also sometimes constructed in the case of small streams which are used to lead flood-waters that are not needed around the end of the dam and thus prevent to some extent the accumulation of sediment.

444. Masonry Waste-weirs.—Masonry dams are not usually designed to allow water to pass over their entire length, but a certain portion only is made to act as a waste-weir. As was the case with earthen dams, the waste-weir is often located at the extreme end of the dam, the overflow passing down a prepared channel in the hillside. Whether it is so placed, or located more nearly in the axis of the valley, depends chiefly upon the topography and nature of the foundations.

The form of a masonry weir depends much upon local conditions, chief of which are height of dam, character of foundation, amount of ice and driftwood to be expected, and quantity of water to be provided for. A weir is essentially a dam with its top and lower face so constructed as to permit the water to pass over it without damage. Besides the design of the profile, the protection of the stream-bed below the dam is a very important feature, as many dams have been undermined by failure at this point even where the bed has been solid rock.

With respect to the form of construction, masonry weirs may be divided into three classes: (1) weirs with a nearly vertical front face, allowing a free fall to the water; (2) weirs with a curved lower face; (3) weirs with a stepped lower face.

445. (1) *Weirs Allowing Free Fall.*—These are ordinarily used for low falls of 10 to 20 feet, depending on the character of the bottom. The front face is made at a batter of 1 to 2 inches per foot, and the rear face whatever is necessary to secure stability. The top width is made sufficient to resist the impact of ice, logs, etc., 5 to 8 feet usually being sufficient. The cap stones should incline downwards up-stream, to relieve them from blows on the back edge. They must be large and well laid, and, where subject to severe shocks, well doweled and clamped together. It may also be necessary to anchor the masonry to the bed-rock. With earth foundations, an earth backing, finished with gravel or paving, is often carried up flush with the back edge. The advantage of this has been noted in Art. 442.

If the stream-bed is not solid rock, it must be well protected by an apron of timber or stone, the former being quite temporary unless constantly wet. A timber apron is usually made as a continuation of the foundation platform with additional layers of thick planking. A stone apron varies in construction according to the requirements from a mere paving, to a heavy apron of broken stone, concrete, and one or more layers of heavy paving set in cement.

With falls greater than 10 or 20 feet, aprons alone are not sufficient security against scour, and even with rock bottom the wear becomes too great, especially if large quantities of ice and logs pass over the weir.

Free falls for greater heights may still be used by protecting the bed by means of a water-cushion, formed by a subsidiary weir built a short distance below the main weir. This reduces the height of fall and also forms a pond into which the water falls and which absorbs its energy. The depth of such a water-cushion depends on the mass of water and character of the bed. It is frequently made one-fifth or one-fourth the height of the main weir.

An example of a weir of considerable height having a free fall is the Macoupin Intake Dam of the East Jersey Water Company, illustrated in Fig. 101, Clemens Herschel, Mem. Am. Soc. C. E., engineer. The coping-stones are well doweled together and bolted to the body of the dam. The stream-bed is solid rock. (See also Fig. 97, page 393.)

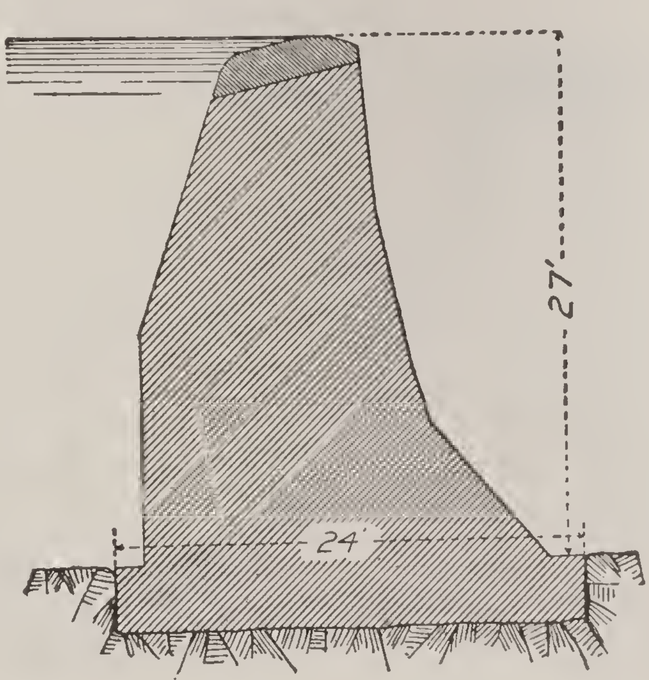


FIG. 101. — MACOUPIN INTAKE DAM.

446. (2) *Weirs with a Curved Lower Face.* — The object of this form is to guide the water smoothly over the dam, and at the bottom to deliver it tangentially with respect to the stream-bed. In this way the water arrives at the bottom with nearly the same velocity as with a free fall but with changed direction, a great advantage where logs and ice pass over the dam. The scouring effect is, however, very great, and in high weirs a water-cushion is here also necessary where large volumes are dealt with. If the depth of water is slight, the velocity may be reduced by leaving the surface of the weir very rough, as in the Vyrnwy Dam. For high weirs the section is designed as for a high dam (making due allowance for the extra pressure due to the superelevation of the water-surface, and for shocks, etc.), and then rounded off. The rear face is made nearly vertical, as in high dams.

The convex top curve to be given to a dam should be full enough to prevent the water leaving the surface. This will be given by the parabolic curve which the water would take in a free fall with the initial horizontal velocity corresponding to the depth on the weir. According to the formula for weirs, the average velocity of the water is $v = c \cdot \frac{2}{3}\sqrt{2gH}$. (See page 229.) In time t the abscissa of the parabola is $x = vt$, and the ordinate is $y = \frac{1}{2}gt^2$, whence $y = \frac{g}{2v^2}x^2$ is the equation of the parabola. In a long weir with ends not freely exposed to the entrance of air the normal pressure is not maintained under a sheet of water, and it will be forced by the exterior pressure to follow a sharper curve than the parabola above. Such action is very observable in many weirs.

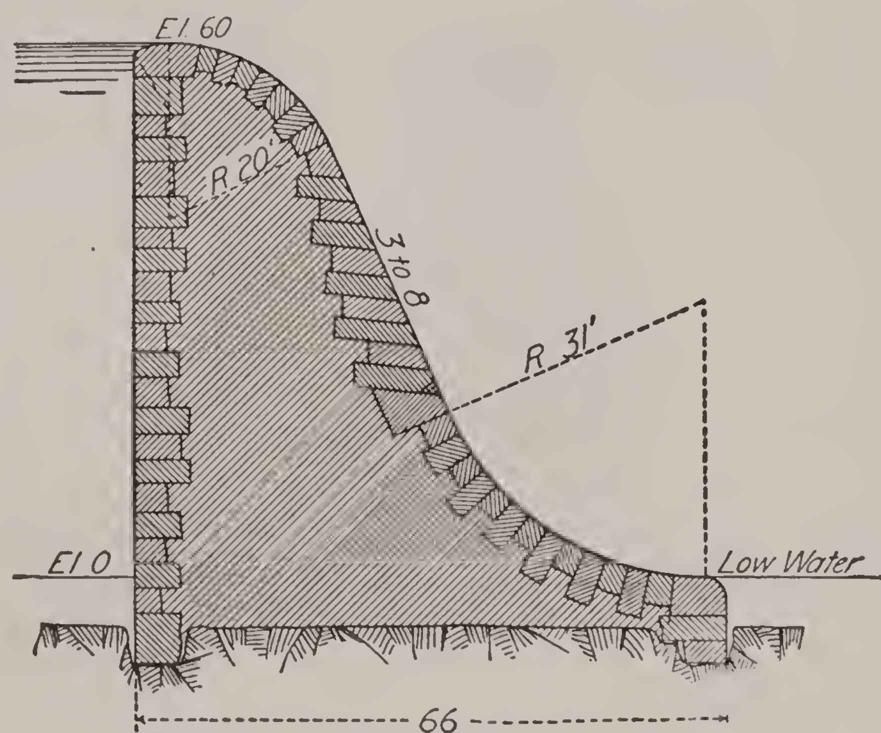


FIG. 102.—COLORADO RIVER DAM AT AUSTIN, TEXAS.

A noteworthy example of a large dam made to act as a weir is the dam across the Colorado River at Austin, Texas, built for water-power purposes (Fig. 102). This structure is 1275 feet long and is built of rubble with granite facing. It was designed to pass flood-waters to a depth of about 15 feet on the crest, but on April 7, 1900, during a flood in which the depth of water flowing was about 11 feet, a large section of the dam failed, a portion sliding down-stream and remaining upright, while a portion was broken up and washed away. The cause of the failure is not definitely known, but some weakening of the foundation is evident, due either to erosion by percolation or by the water falling below the dam.* This dam is an exception in respect to its height and the great volume of water to be provided for, and the protection of the stream-bed from the action of the great mass of water is in such a case a matter of very great importance.

Fig. 103 illustrates another dam built for power purposes and designed for a large flow. The facing of this dam is also of granite, the curve for the

* See *Eng. News*, April 12, 1900, *et seq.*; *Eng. Record*, April 14, 1900, *et seq.*

upper portion being a parabola corresponding to the curve of the flowing water when 4 feet deep. The stones are thoroughly doweled together. The lower portion of the dam is cycloidal, and the upward slope of the toe is introduced so as to form somewhat of a water-cushion.*

447. (3) *Weirs with a Stepped Profile.*—In this form the lower face is stepped instead of curved, with the object of breaking the fall into several small steps and absorbing the energy of the water before it reaches the bottom. This very much simplifies the problem of scour,

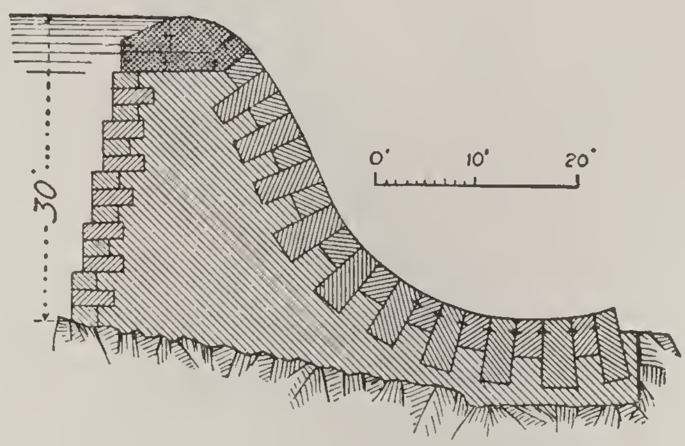


FIG. 103.—THE NEW HOLYOKE DAM ACROSS THE CONNECTICUT RIVER.

and at the same time gives a form cheaper to construct than the curved outline. It is well suited to carry moderate quantities of water. With the stepped profile the wear comes more on the dam, while with the curved form it is more on the stream-bed. The masonry of the steps requires to be of the heaviest and most substantial character. Single stones should be used extending well under the masonry above.

An example of a spillway of considerable height is shown in Fig. 104, a section of the Dunning's Dam, E. Sherman Gould, Mem. Am. Soc. C. E.,

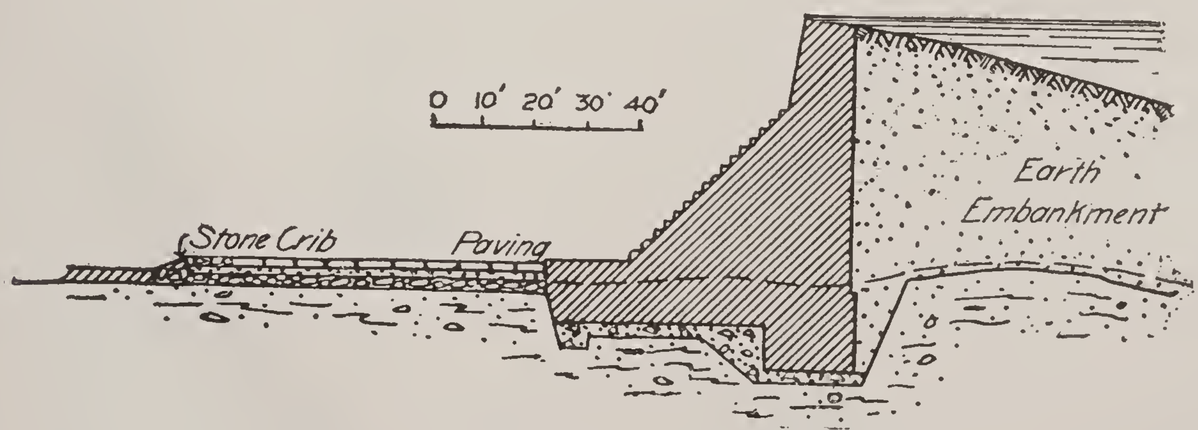


FIG. 104.—THE DUNNING'S DAM.

engineer. The dam is noteworthy as being partly founded on rock and partly on earth, conditions very difficult to deal with. The weir is founded on clay and fine sand. The apron consists of, first, a filling of large stones, then one foot of concrete, then a heavy paving in cement mortar. Below is a timber crib filled with stones, and farther down, the channel is riprapped. The dam

* *Eng. News*, 1897, XXXVII. p. 292.

is backed with earth, which is considered by the designer as being a valuable safeguard for a masonry dam.*

Fig. 105 is a section through the highest portion of the spillway of the

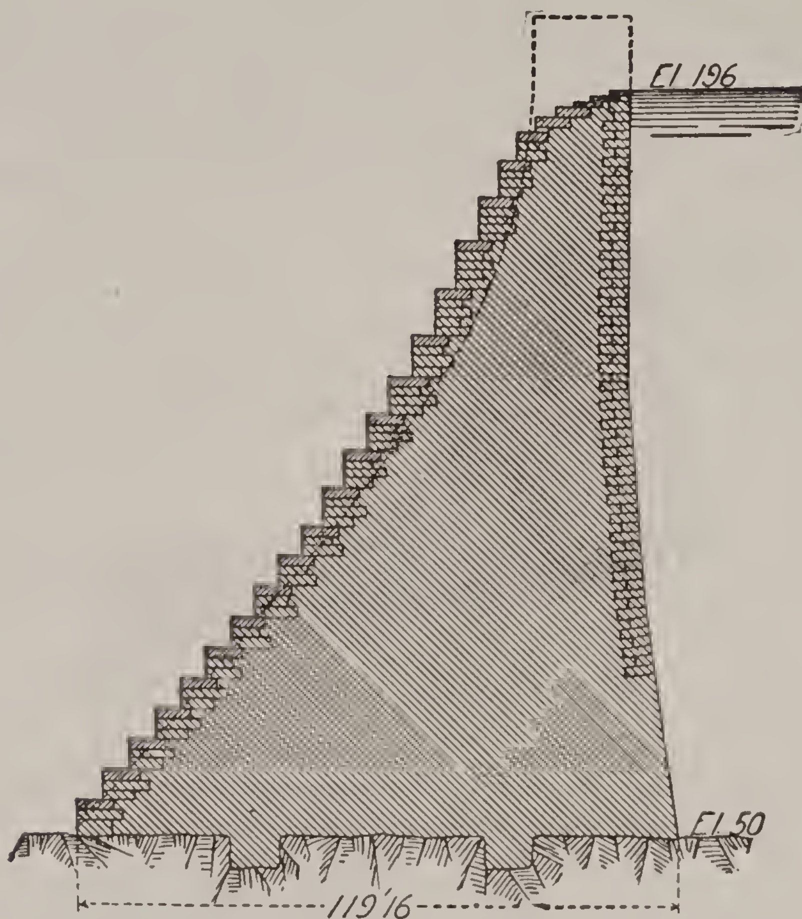


Fig. 105. — Spillway, New Croton **Dam.**

New Croton Dam. This design may be considered as well representing modern practice in this direction.

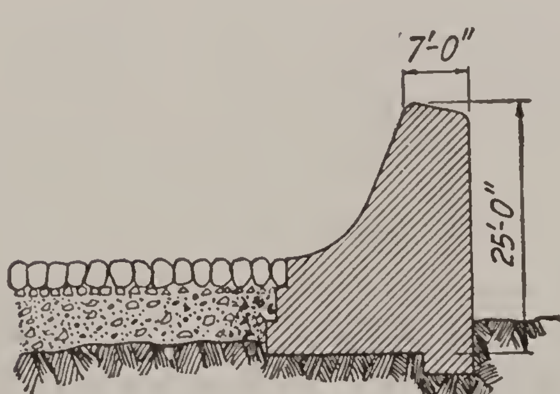


Fig. 106. — Dam at Troy, N. Y.

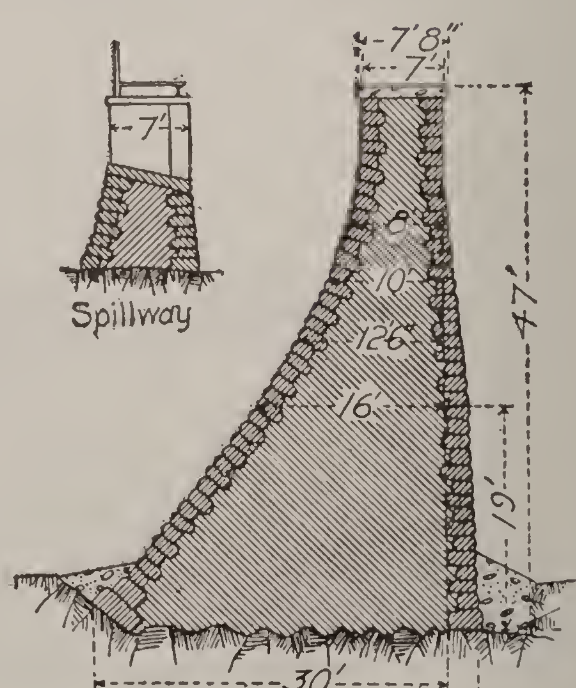


Fig. 107. — Indian River Dam, N. Y.

* Trans. Am. Soc. C. E., 1804, XXXII. p. 737.

448. Other Examples of Dams. — Fig. 106 illustrates a concrete spillway for a dam of moderate height. It constitutes a part of the water-works of Troy, N. Y. This spillway adjoins an earthern embankment, the abutments being partially shown in the figure.*

Fig. 107 illustrates a small modern dam across Indian River, N. Y., in the upper Hudson valley, built for water-power and navigation purposes. The dam is of rubble masonry with spaces filled with concrete.†

On page 406 are illustrated the sections of several of the largest dams of the world, all sections being drawn to the same scale. Of these dams the New Croton will, when completed, be the largest. This dam consists of a masonry portion, of the section shown, for a length of about 700 feet, a curved masonry spillway at one end 1000 feet long (Fig. 105), and at the other end an earthen embankment with masonry core-wall (Fig. 75, page 351). The masonry dam has a maximum height above foundation of about 290 feet, measuring to the deepest pocket. In section it is practically the same as adopted for the Quaker Bridge Dam. The height of spillway varies from 10 to 150 feet. Rubble masonry is used for hearting, and ashlar for facing. The steps on the waste-weir are to be made of block masonry, and of sufficient depth to bond under the step above. The estimated cost of this dam is nearly $5,000,000. It was begun in 1892, and will be completed about 1903.

The Vyrnwy dam, which is a part of the Liverpool Water-works, is noteworthy on account of the great care taken to obtain strong, impervious masonry, and in the provision made for drainage. It is built of large rubble masonry, a large proportion of the blocks ranging from 2 to 8 tons in weight. All stones were washed and scrubbed with jets of water under 140 feet pressure. The mortar was made of Portland cement 1 to 2 and 1 to $2\frac{1}{2}$, the sand being composed of pulverized rock mixed with natural sand. Large stones were bedded upon a 2-inch layer of mortar which was first beaten to expel the air. The stones were also beaten into place by blows from hand-mauls. The spaces between the stones were filled with small rubble or concrete rammed into place. The crushing strength of the concrete, one year old, was about 187 tons per square foot. The specific gravity of the masonry was found to be about 2.5. The maximum pressure at the upper face is 8.7 tons and at the lower face 6.36 tons per square foot. To prevent the existence of hydrostatic pressure in the dam a system of drains was constructed in the foundation. These drains are 9 to 12 inches square and are kept 25 feet from the front face. They connect with a concrete tunnel 4 feet by 2 feet 6 inches wide running longitudinally through the dam and $46\frac{1}{2}$ feet above the base. This opens out to the surface by a cross-tunnel. Length of dam = 1350 feet. Maximum height = 136 feet. It is designed to act as a waste-weir.‡

The San Mateo Dam in California is noteworthy as having been built entirely of concrete blocks, each of about 9 tons weight.

The Furens Dam, France, is famous as being the first one constructed on scientific principles, and until recently the highest dam in existence. It was completed in 1866.

The Periar Dam, Madras, is another notable concrete dam. The maximum pressure intensity is stated to be 8 tons per square foot.

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* *Eng. News*, 1904, LII. p. 300.

† *Eng. News*, 1899, XLI. p. 310.

‡ Proc. Inst. C. E., CXXVI. p. 24.

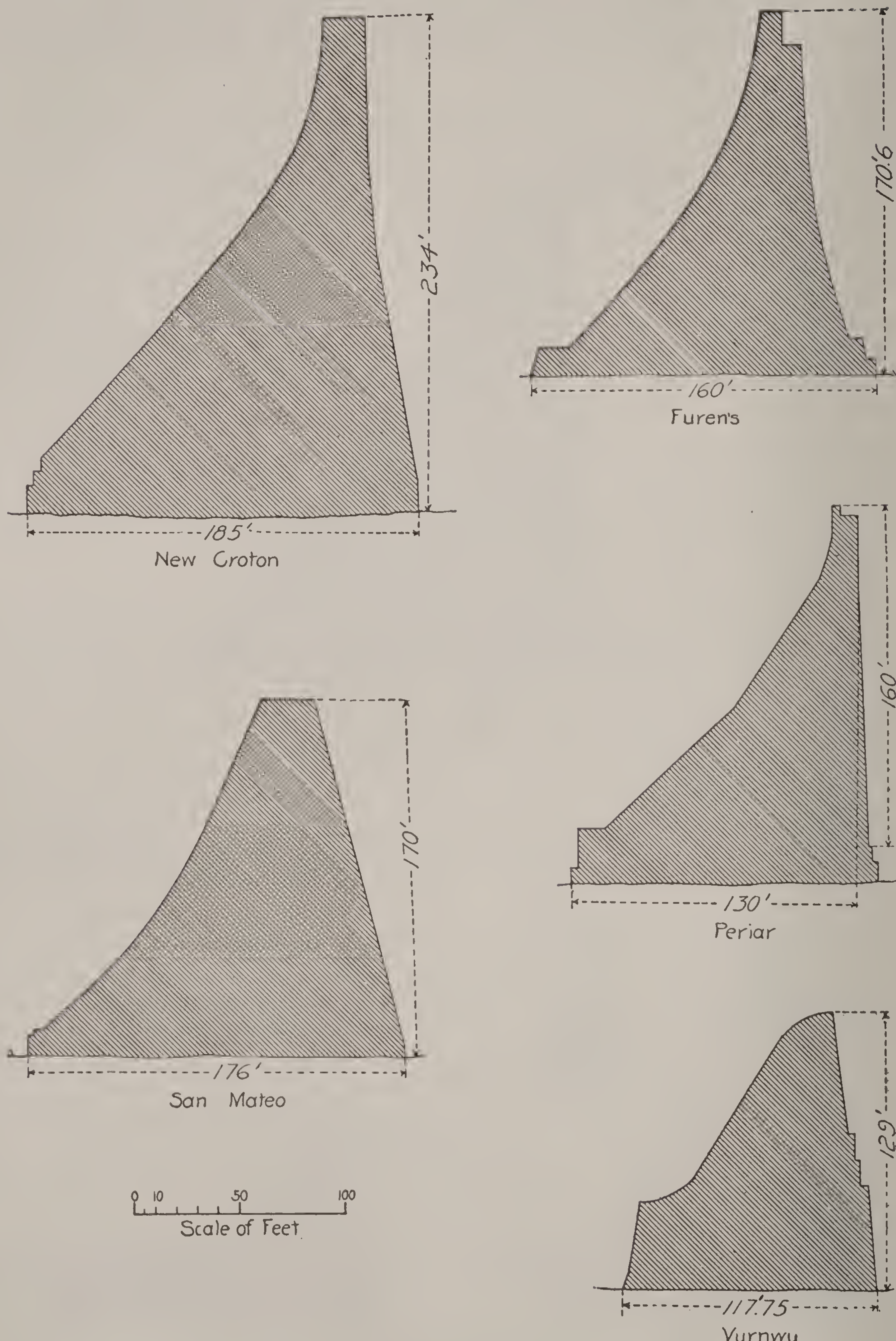

FIG. 108. — PROFILES OF SOME HIGH MASONRY DAMS.

448a. Dams of the Buttress Type. — Considering again the two portions of a dam, the impervious part and the supporting part, the question arises if a portion of the material of a masonry dam, which serves merely as supporting material, might not be omitted. This can be done in various ways.

The up-stream face may be made in the form of masonry arches and these supported on piers or buttresses; a steel facing may be employed in place of the masonry arches; or a covering of reinforced concrete may be used. Since the development of reinforced concrete the last named method has been employed in several cases with resulting economy.

In the design of a buttress type of dam the buttresses are proportioned for the entire pressure. For high dams they must therefore be made considerably broader than the base of a dam of solid section. The position of the line of pressure can readily be varied by varying the slope of the face. A special advantage of this type of dam in certain cases is that it permits the buttress foundations to be constructed as separate piers.

Fig. 108a illustrates the concrete dam at Ogden, Utah, built for the Pioneer Power Plant. The piers are concrete walls 16 feet thick with 32 feet in the clear, and the concrete arches vary from 6 to 8 feet in thickness. The arches are protected and rendered more impervious by a covering of steel plates, although this covering is not essential. In this dam, 100 feet high and 400

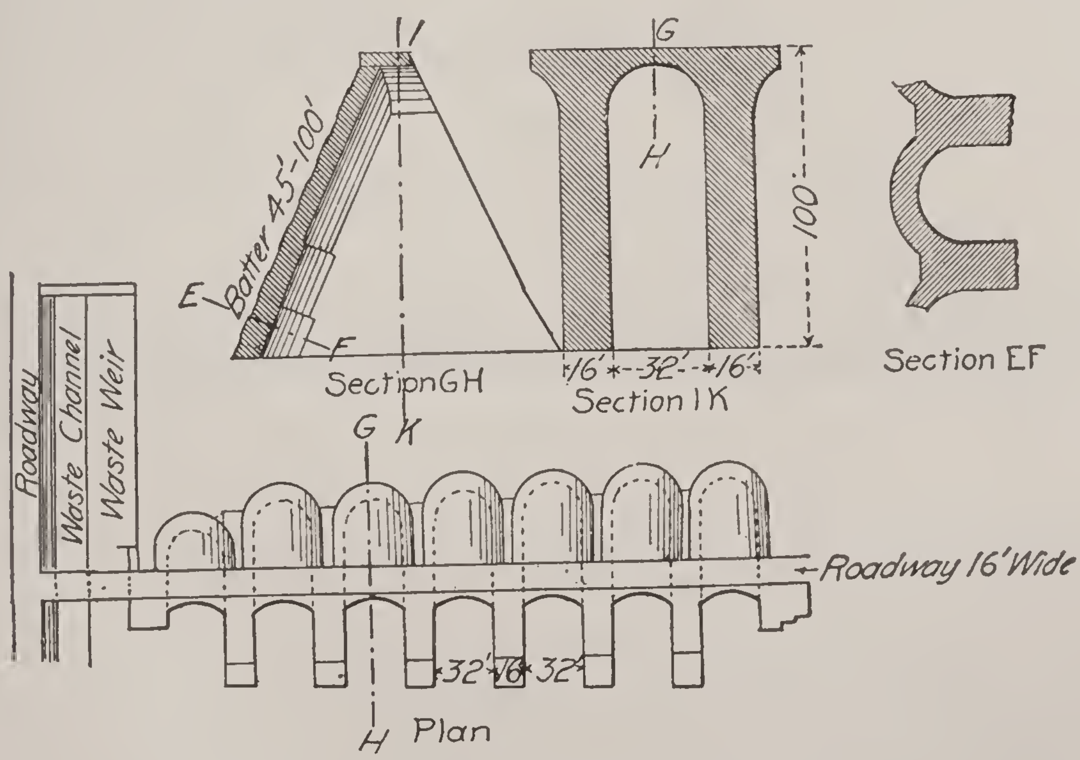

FIG. 108a. — CONCRETE DAM, OGDEN, UTAH.

feet long, the quantity of masonry is given as 26,000 cubic yards as compared to 37,200 cubic yards estimated for a dam of ordinary section. The actual cost, including steel covering, was 12 to 15 per cent less than that of an ordinary dam. The maximum pressure is 10.7 tons. Instead of concrete arches, a steel face formed of steel plates was also considered, but was found to be more expensive than the adopted design.*

The other dam of this sort is on the Belubula River, New South Wales. It was there adopted on account of the ridgy nature of the bottom. The height is 60 feet, length 431 feet, with six buttresses 28 feet apart center to center, 40 feet long and 5 to 12 feet thick. Brick arches were used, 4 feet thick at bottom and 1 foot 7 inches at top, built at an angle of 60 degrees with the horizontal.†

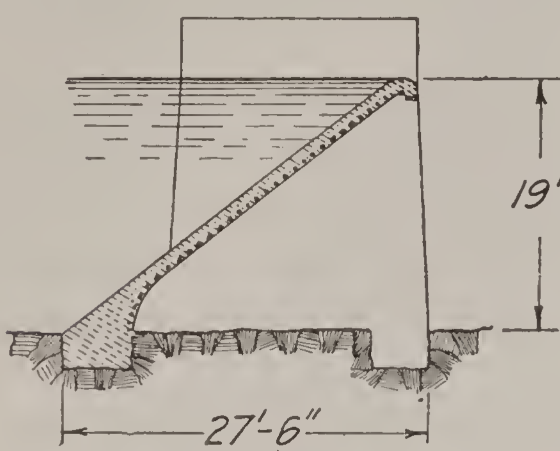

Fig. 108b.—Dam at Ellsworth, Me.

Fig. 108b illustrates the usual form of reinforced concrete dam where the water may be allowed a free fall or where no water passes over the dam. It consists of separate concrete buttresses spaced about 8 feet apart, supporting an inclined floor of reinforced concrete. As regards strength the method of design is evident.‡ In such structures large factors of safety should be employed and at all points subject to impact the dimensions will usually need to be much greater than called for by the static load in order to provide sufficient weight and mass.

Fig. 108c illustrates the dam at Schuylerville, N. Y. This has a down-stream floor or apron and is designed to act as a spillway. To avoid any

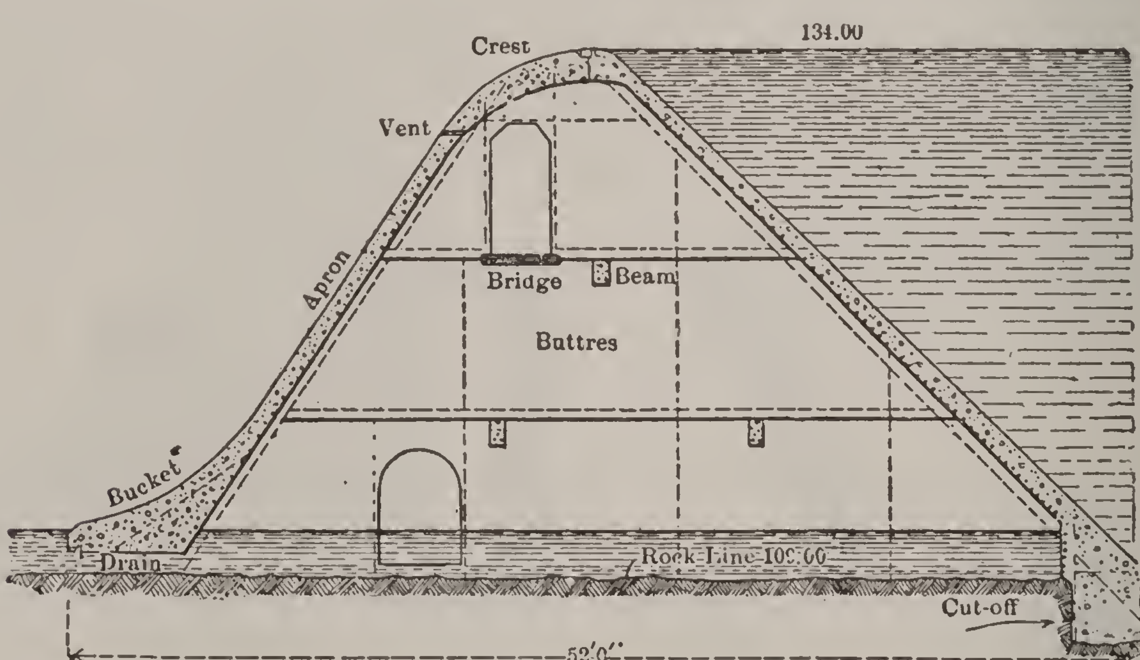

Fig. 108c.— Dam at Schuylerville, N. Y.

internal pressure due to seepage through the up-stream face, drain openings are provided in the down-stream face.§

Where constructed upon earth foundation, a continuous floor of reinforced concrete from buttress to buttress may be used to spread the load and to prevent scour. The entire structure thus becomes a monolith and is exceedingly strong and rigid. Thus built it may be safely constructed

* Trans. Am. Soc. C. E. 1897, XXXVIII. p. 291.

† *Eng. News*, 1898, LX. p. 148.

‡ *Ibid.* 1904, LII. p. 255.

§ *Ibid.* 1905, LIII. p. 448.

on pile foundations, care being taken to cut off seepage at the up-stream toe by means of sheet piling well connected to the concrete.

449. **Cost.** — The cost of constructing masonry dams will vary greatly with the local conditions. If these are reasonably favorable as to transportation and ease of securing stone, the range of prices for the principal items will be about as follows: Earth excavation 25 to 50 cents per cubic yard; rock excavation $1.00 to $2.00; rubble masonry, natural cement, $4.00 to $6.00; concrete masonry, natural cement, $4.00 to $6.00; for masonry laid in Portland cement add about $1.00 per cubic yard; reinforced concrete $8.00 to $12.00, including steel; rock-faced ashlar masonry $10.00 to $15.00; dimension-stone masonry for gate-houses, etc., $15.00 to $30.00; paving $2.00 to $3.00; riprap $1.50 to $2.00.

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CHAPTER XVIII.

TIMBER DAMS; LOOSE-ROCK DAMS; STEEL DAMS.

TIMBER DAMS.

450. **Use of Timber Dams.**—Where a weir is constantly submerged, a timber structure is of a permanent nature, and will need repairs only on account of the wear of the apron. A timber dam may also be advisable in certain circumstances even when its life will be short, as, for example, where a temporary supply may be furnished pending the construction of more permanent works, or where the expense of permanent and costly structures is for the present prohibitory. Such dams are, however, used mostly for diversion purposes or for water-power, and seldom for the storage of large volumes of water.

Timber dams may be constructed on any kind of a foundation, but are usually built on rock or on a gravelly bed. They consist of cribs or frames built of logs or squared timber, filled with stone and clay, and planked over to render them water-tight. They may be built as separate cribs in sections, each section consisting of perhaps 3 to 4 cribs, or as one continuous framework. The former method is especially useful in dealing with large flows and irregular foundations, the stream being gradually closed as the sections are constructed. The cribs may also be filled and sunk separately so as to form piers on which a continuous structure may be built.

The foundation of a crib dam, if soft, is prepared by dumping stone over the area to be built upon. In the framed dam the foundation must be more carefully prepared. Where it is soft the dam is supported on piling, and sheet-piling is used to prevent underflow. If the dam is built on a rock bottom, it must be bolted thereto. The framework is usually built with a sloping upper face and a series of stepped aprons below, or a single free fall to a water-cushion. Rock and gravel, or puddle is used for filling.

451. **Examples of Timber Dams.**—*Sewall Falls Dam* (Fig. 109).—This dam across the Merrimack is a crib dam 497 feet long, constructed on a hard-pan foundation. It was built in sections by means of coffer-dams, sluiceways

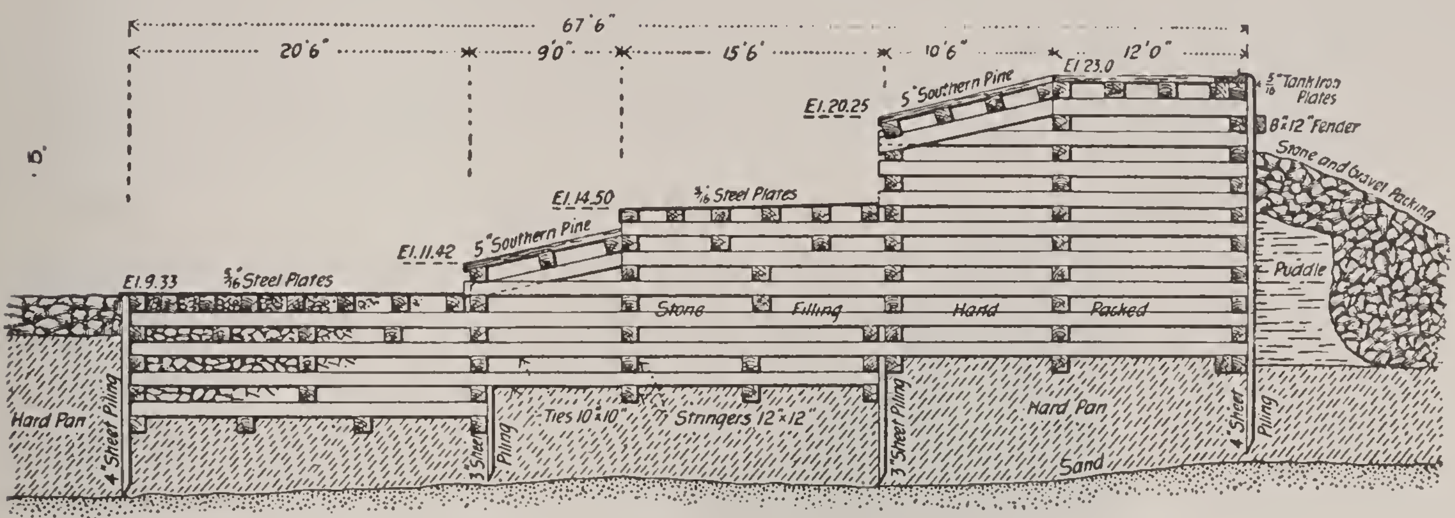

FIG. 109.—SEWALL FALLS DAM.
(From *Engineering News*, vol. XXXI.)

being left in the completed portion to carry the water during the construction of the last sections. The longitudinal pieces are 12 × 12-inch hemlock and Southern pine, and the ties 10 × 10-inch hemlock, all fastened together by drift-bolts. The spaces were hand-packed with stone. The aprons are made of steel on account of heavy ice. The figure shows the construction clearly. The life of the structure is estimated by the engineers at fifty to sixty years. The cost was about 60 per cent that of a stone dam, the contract price being $120,000.*

452. *Bear River Weir.*—Fig. 110 is a section of a timber weir across the Bear River, Utah, built to divert water for irrigating purposes. The foundation is solid rock into which the timbers are bolted. All timbers are 10 × 12-

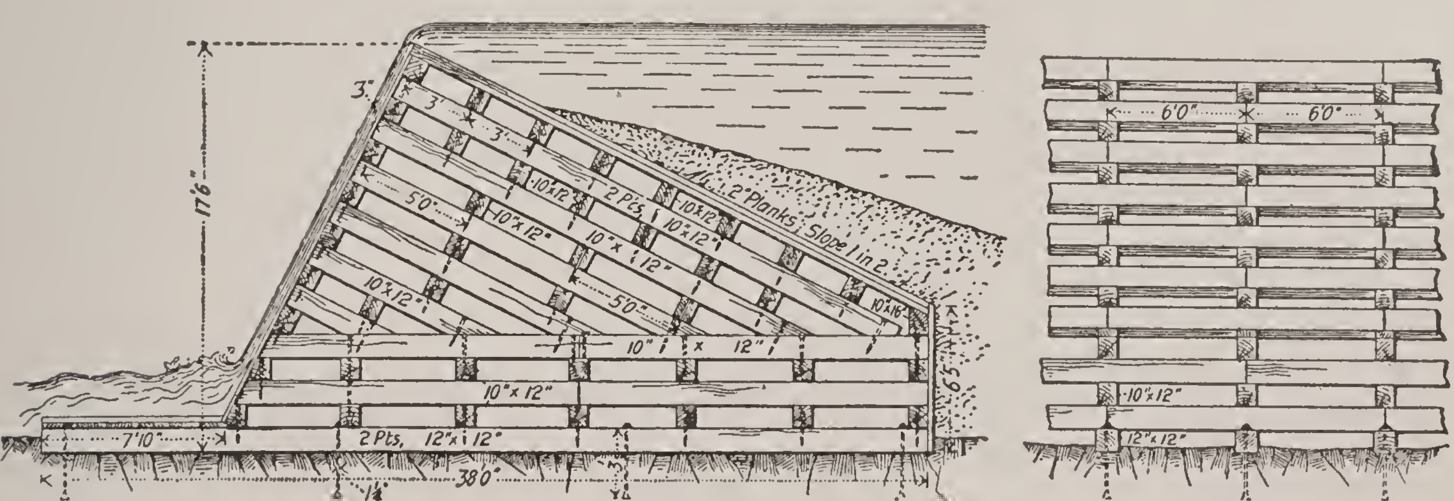

FIG. 110.—BEAR RIVER DAM.
(From *Engineering News*, vol. XXXV.)

inch. The interior is filled with stone, and a heavy layer of earth is placed at the back to prevent percolation. In the middle of the stream the apron consists of 10 × 12-inch timbers, instead of the second layer of 3-inch plank as shown. A portion of the dam founded on gravel and boulders was badly underscoured in 1891. This part was afterwards protected by two rows of

* *Eng. News*, 1894, XXXI. p. 326.

sheet-piling 4 feet apart, driven at the back side, the space between being filled with concrete. The whole was then covered with earth and boulders.*

453. *Butte, Mont., Crib Dam.* — In Fig. 111 is illustrated a crib built at Butte, Mont., and notable for its great height. The dotted portion shows the section of the spillway. The height from low water to crest is 56 feet. It is founded on a bed of stiff clay and boulders 12 to 35 feet below the surface. A concrete wall 4 feet thick extends from the foundation to a point

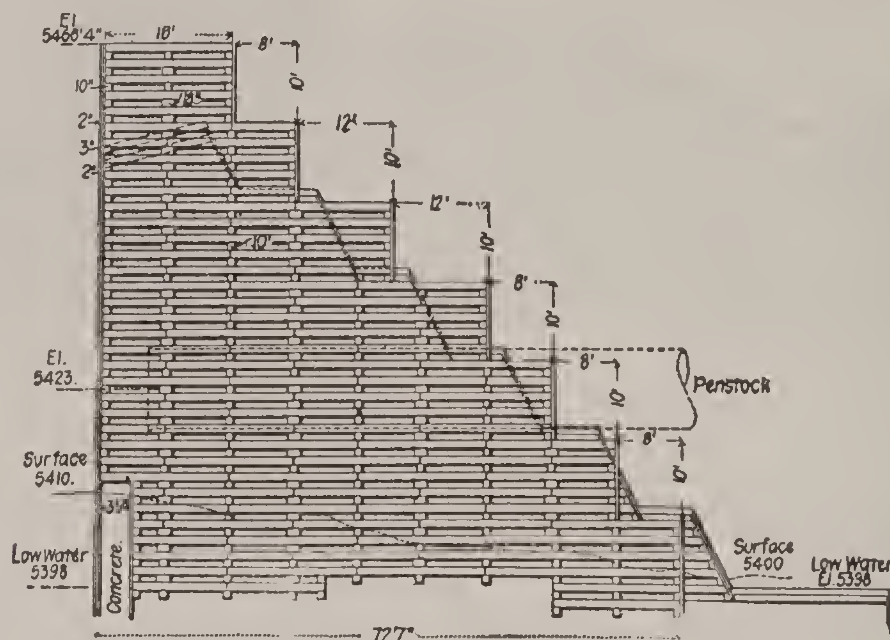

FIG. 111.— TIMBER DAM AT BUTTE, MONT.
(From *Engineering Record*, vol. XXXVII.)

about 6 feet above the original surface, as shown in the figure. The remainder of the excavation is filled with clay puddle, well rammed. The dam is made of 10 × 12-inch pine timbers and filled with granite packed in layers in the crib-work. Soon after completion this structure partially failed under a heavy flood. The pressure of the water caused the highest portion to settle or cant over (the top moving some 7 or 8 feet down-stream), and the entire structure to settle vertically. At one place twenty-seven 12-inch timbers were compressed to a thickness of 24 feet 10 inches. The failure was due to lack of resistance to shearing forces, and to the compression of the timbers at the joints. The filling was not sufficiently compact to render the structure rigid, and no diagonal bracing was used.†

LOOSE-ROCK DAMS.

454. **Loose-rock Dams.** — Dams composed largely of loose rock have been used to a considerable extent in the West, and in some respects present considerable advantages as to stability. Another advantage is that they can be constructed in running water, but the finished dam is not suited to act as a waste-weir.

The body of the dam is made of loose rock placed with more or less care, and rendered comparatively impervious by a sheathing of plank, or by a facing of earth or fine material on the upper face, or, as in one case, by a core of steel. As regards stability the principle of

* *Eng. News*, 1896, XXXV. p. 84. † *Eng. Record*, 1898, XXXVII. p. 301, XXXVIII. p. 203.

construction is of the best. Since considerable percolation is likely to take place, such a dam cannot be founded on a material liable to scour; and if the dam is high, the foundation should be solid rock. The lower slope is usually 1 to 1, while the upper slope may be made $\frac{1}{3}$ or $\frac{1}{2}$ to 1; but to secure these steep slopes it is necessary to lay the stone for a considerable thickness as a dry wall. Above this wall the facing of timber or earth is placed. The former material is objectionable on account of its perishable nature.

Rock-fill dams have been constructed where a stratum of loose material of considerable thickness has overlaid the solid rock. In such a case, as the dam is built up the loose material gradually scours out and the loose rock settles into place. On such a foundation both slopes must be made quite flat and no reliance can be placed on retaining-walls of any sort.

A disadvantage of rock-fill dams is in the relatively large loss of water which occurs, an important consideration in the case of storage-reservoirs. The cost of such dams has in some cases been very low, in one instance as low as 45 cents per cubic yard.

455. Examples. — Fig. 112 shows the section of the dam at Pecos, N. Mex. The facing here is of earth.

A rock-fill dam with timber facing is shown in Fig. 113, the Escondido Dam in California. The upper portion of the dam is laid as a dry wall with a thickness of from 5 to 15 feet. The height is 76 feet. The outlet is a 24-inch cast-iron pipe laid in concrete and having a valve at the upper end.*

An interesting example of a rock-fill dam is illustrated in Fig. 114, which represents the lower Otay Dam in southern California, already referred to on page 351 as having a steel core. A masonry dam had been considered for this place, but owing to the great depth to bed-rock the plans were changed. The construction is clearly shown in the figures. The lower figure shows to an enlarged scale the method of joining the steel and masonry core to the foundation at the ends of the dam. The rock forming the dam was placed by dumping from a cableway. The leakage is very slight. Another very notable example of the rock-fill type is the dam at Laguna, Ariz., across the Colorado River.†

STEEL DAMS.

456. Steel Cores. — The use of steel cores and facings for concrete, loose rock and earthen dams has been noted in Arts. 455, 448a, and 391. In such cases the steel is employed to furnish or insure the desired impervious face; the supporting element is furnished by other material.

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* *Eng. News*, 1898, XXXVIII. p. 63.
† *Ibid.*, 1908, LIX, p. 213.

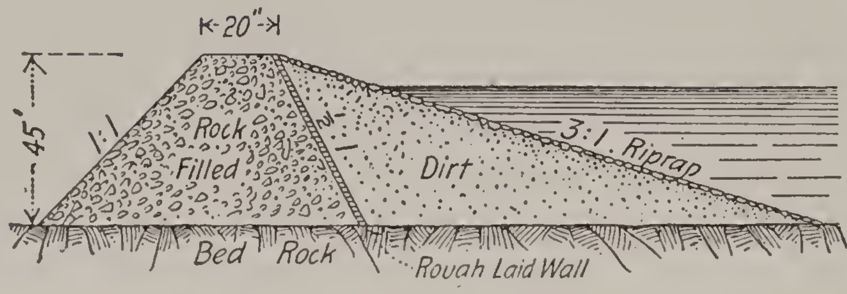

FIG. 112.— PECOS DAM.
(From *Engineering News*, vol. XXXVI.)

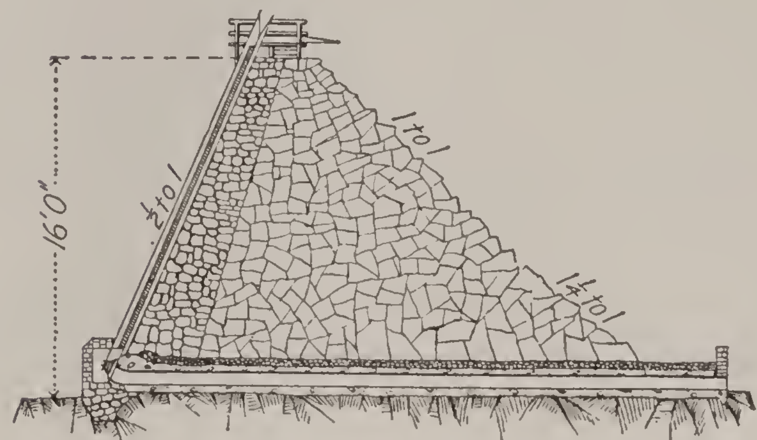

FIG. 113.— ESCONDIDO DAM.

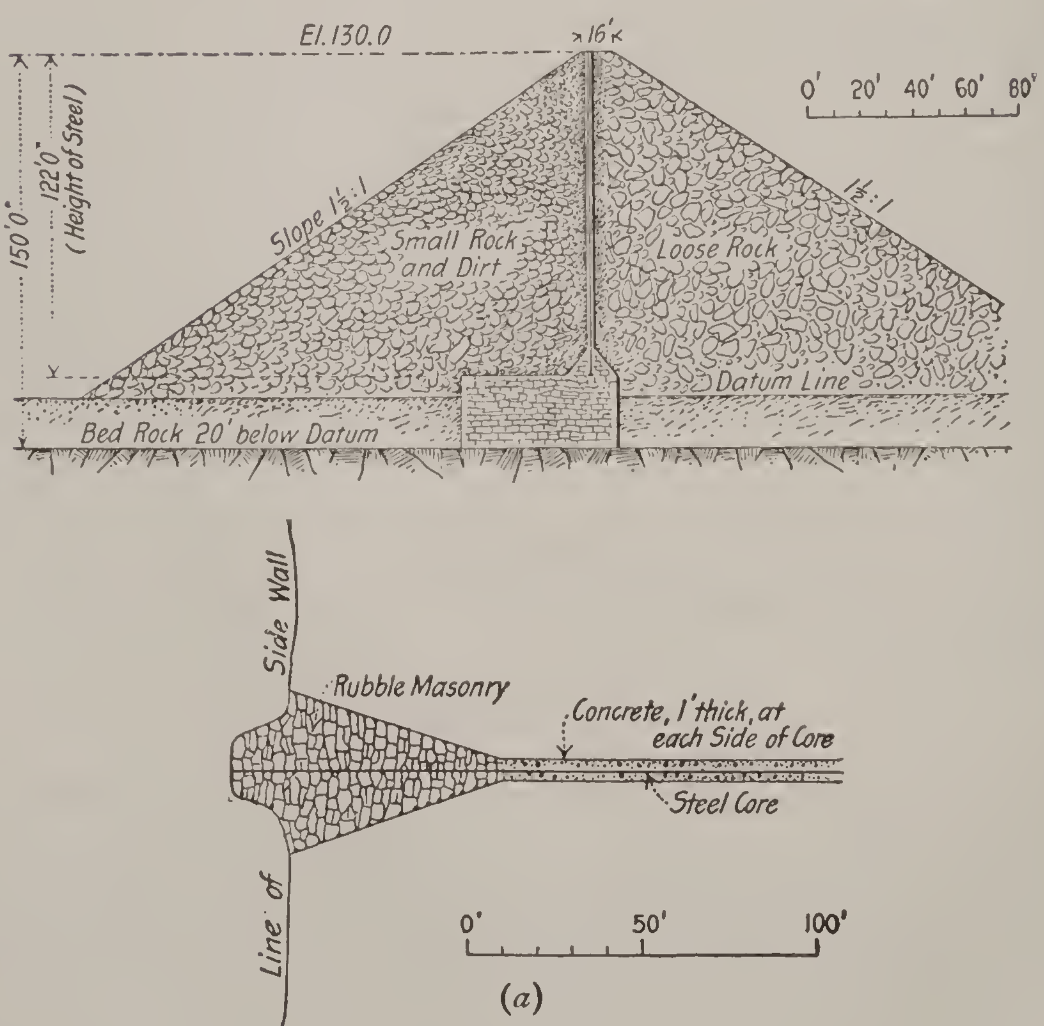

FIG. 114.— LOWER OTAY DAM.
(From *Engineering News*, vol. XXXIX.)

457. Dams built wholly of Steel. — A dam entirely of steel has been built in Arizona, at Ash Fork. The face consists of curved plates $\frac{3}{8}$ inch thick imbedded at the bottom in concrete. The greatest height is 46 feet. The plates are riveted to a system of inclined struts resting on a rock foundation. Expansion is taken up by a slight bending in the curved plates. Such a form as this possesses a great advantage in the definiteness with which the stresses can be calculated and provided for, and the fact that the stability of the structure is independent of the imperviousness of the face. Its chief disadvantages lie in the cost for maintenance and, probably, in its lack of durability, a point not yet well determined. Fig. 115 shows the form of bracing at the highest portion.* Another notable steel dam is that across the Missouri River, near Helena, Mont.

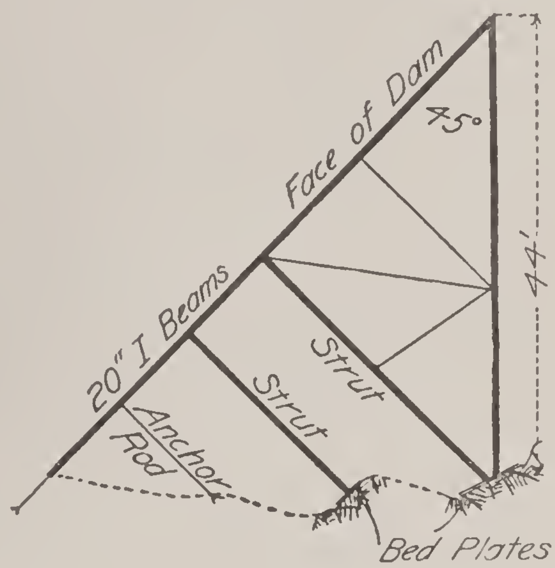

Fig. 115. — Steel Dam, Ash Fork, Arizona.

A comparative estimate for a 60-foot dam, made in connection with the Ogden Dam described above, gave for a steel dam of the form shown in Fig. 115 a weight of 7000 pounds per lineal foot; for a cantilever design for a steel dam 8050 pounds per lineal foot; and for an ordinary masonry dam 48 cubic yards per lineal foot.†

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* *Eng. News*, 1898, XXXIX. p. 299.

† Trans. Am. Soc. C. E., 1897, XXXVIII. p. 305.

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B. WORKS FOR THE PURIFICATION OF WATER

CHAPTER XIX.

OBJECTS AND METHODS OF PURIFICATION.

458. **Purification of Water for Manufacturing Purposes.**—In the purification of water-supplies, reference is generally made to the treatment of water designed for domestic use, but the subject may also be considered as applied to water intended for manufacturing purposes. Generally speaking, in technical industries and for manufacturing purposes a soft water is desired, also one that is free from organic impurities. In such industries as brewing, distilling, and sugar and starch manufacturing, the question of germ content is more important, as water containing some kinds of micro-organisms is apt to produce abnormal fermentations that injure the product. Iron-containing waters are particularly detrimental in the manufacture of paper and pulp, also in dye-works. These technical industries, however, demand a special examination in selecting a proper source of water-supplies, and generally do not pertain to the ordinary work of a water-works engineer.

For general manufacturing purposes it is desired that a water should not readily form boiler-scale. The precipitation of certain inorganic salts, particularly those of calcium and magnesium, interferes much with the economic action of boilers as steam-generators. The accumulation of this incrustation to a thickness of one-sixteenth inch involves a loss variously stated at from 12 to 20 per cent of the energy of the coal used. The methods of purifying water so as to remove the mineral ingredients capable of forming boiler-scale deserve, therefore, careful consideration.

459. **Purification of Water for Domestic Purposes.** — In ordinary household use the quality of water is of considerable import. Not only is a water that is rich in alkaline earths not well adapted for cooking and similar purposes, but on account of its action upon soap it is very

undesirable for general household use. In a hard water, soap is decomposed and the fatty acids unite with calcium and magnesium salts, forming insoluble compounds under such circumstances. To secure the cleansing action, it becomes necessary to use a much larger amount of soap. The removal of the hardening impurities of a water constitutes, therefore, an important feature of water purification. Its economic value is well illustrated by the case of Glasgow already mentioned in Art. 150.

In purifying a drinking-water there may be two objects in view. That which is the most important is the treatment of the water in a way so as to remove any danger from pathogenic organisms; in addition, however, waters may be purified so as to improve their physical appearance. This latter object, while it ought to be subordinated to the former, often is not, and in the eyes of the consumer an unsavory water will often cause more complaint than a pure sparkling water that may be polluted with disease organisms. Not all waters destined to be used as drinking-water supplies need artificial purification. Ground- or spring-waters rarely need to be artificially treated, as they have already been purified by the operation of natural forces (Chapter IX). They sometimes need treatment for the removal of iron, but generally speaking, so far as deleterious bacteria are concerned, they are comparatively safe if they are normal ground-waters.

The waters that need artificial purification most are those that remain in contact with the surface of the soil. Not infrequently it is possible to secure a surface supply that is perfectly wholesome, but the opportunity for pollution is too often present, and the only regions in which unpolluted waters are likely to be found are those that are sparsely settled. With the increasing density of population, surface waters are in general becoming more and more dangerous, until in many sections it has become impossible to furnish a supply that is safe without the use of some method of artificial purification. This condition is seen in the steady increase of the typhoid death-rates in many of the cities that are supplied with waters from surface sources.

460. Outline of Methods of Purification Employed.—Numerous processes of purification have been devised and tested experimentally to a greater or less extent, but in actual practice only a few have been found feasible. While many of the methods have been the outgrowth of empirical testing, others have been devised as the result of a thorough study of the principles that have been found to underlie these processes as they occur under natural conditions or where artificially controlled.

The various processes of purification may be divided into two general groups: (1) those for the removal of suspended impurities, and (2) those for the removal of dissolved impurities. Of the first class there are two general processes, sedimentation and filtration, both of which may be called natural processes. By sedimentation, water may be more or less freed of its suspended matters, including the bacteria, the efficiency of the treatment depending much upon the element of time. The process is carried out artificially in large storage-reservoirs or in small special settling-basins. It is often aided by the introduction of some chemical that will produce a precipitate which will readily settle and carry down with it the more finely divided matter in suspension. Variations in the method of operation of settling-basins and in the introduction of the chemical give rise to various modifications of the general process.

Filtration is accomplished in different ways. The most common is by means of the artificial sand-filter bed, either as contained in masonry basins of large size, or confined in small tanks as in the mechanical filters. Special forms of filtering media have also been devised, such as the Fischer tile filter, also filters made of asbestos, and the various forms of small filters for domestic purposes. The chief object is in all cases the removal of the suspended matters, and in most public supplies particular attention is paid to the removal of bacteria. In many instances chemical changes occur in filters, but they are not often of any great importance.

The processes for the removal of dissolved impurities include the softening process, in which lime and magnesia are removed by chemical precipitation, and the process for the removal of iron in a similar manner. Such methods usually involve subsequent sedimentation or filtration for the removal of the precipitate. In the iron treatment the filtration itself sometimes plays an important part also in the chemical changes involved. Aeration consists in bringing air into intimate contact with all parts of the water. This acts both to supply deficient oxygen and also to drive out objectionable dissolved gases. It often constitutes an important part of other processes.

Besides the above-mentioned processes there should be mentioned the method of purification by distillation, in which practically all impurities are removed, and the various methods of sterilization, in which the bacteria are simply killed.

From the preceding statements it will be seen that each problem in water purification demands individual treatment; and that the best method to adopt in any case will depend upon the character of the water and the use to which it will be put, both of which elements are subject to many variations. No one process is universally applicable; furthermore, of two processes for removing the same kind of impurity, the most efficient may not in all cases be the best. The highest efficiency is not always necessary, and in such cases economy may properly be secured by the adoption of a system of less efficiency but of lower cost.

In 1902 there were reported the following number of cities of more than 3000 population using the various methods of purification:—*

Slow sand filters	21
Rapid or mechanical filters	141
Sedimentation basins	53
Filter galleries	14
Filter cribs	11
Softening plants	2
Aërating plants	5

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CHAPTER XX.

SEDIMENTATION AND COAGULATION.

461. In the case of many surface supplies the water contains at various times large quantities of suspended matter, either with or without more serious polluting substances; and a considerable part of the work of purification consists in the removal of this suspended matter so as to improve the physical appearance of the water.

462. **The Character of the Suspended Matter.** — In streams such as would be considered suitable as sources of supply the sediment is principally of an inorganic nature, consisting of particles of sand and clay of various sizes. There is also usually a small amount of organic matter, and, in addition, varying numbers of bacteria, which, although too minute to render the water turbid, yet are of the greatest importance on account of their possible relation to disease. During seasons of high turbidity, the bacterial content is usually very high, owing to the large numbers derived from the surface drainage of the soil. Varying numbers may also be derived from sewage pollution, but the bacteria from this source are usually more numerous during the seasons of low water when the turbidity is at a minimum. The amount and character of the sediment varies greatly from time to time, as pointed out in Chapter IX; it depends largely upon the stage of water in the different tributaries, and upon the geological character of the various parts of the drainage-area. Thus Fuller found that the amount of sediment in the Ohio River water at Louisville varied from 1 to 5000 parts per million, ranging ordinarily from 100 to 1000; and that the bacteria varied from a few hundred per c.c. to as high as 50,000.*

The size of the suspended particles varies greatly. In some waters the finer particles of clay are less than 0.00001 inch in diameter, which is smaller even than bacteria. This great variation in amount and kind of sediment constitutes one of the most troublesome factors in connection with purification works for river supplies. For example, at New Orleans the water is much more difficult to treat than at St. Louis, although containing a lower percentage of sediment.

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* Water Purification at Louisville, 1898, p. 15.

The average amount of sediment carried by various river waters used as public water-supplies is reported as follows: *

City	River.	Suspended Matter.	
		Parts per Million.	Tons per Million Gallons.
Lawrence	Merrimac	10	0.042
Albany	Hudson	15	0.062
Pittsburgh	Allegheny	50	0.208
Washington	Potomac	80	0.333
Cincinnati	Ohio	230	0.957
Louisville	Ohio	350	1.480
New Orleans	Mississippi	650	2.70
St. Louis	Mississippi	1000	4.16

463. Limitations of Artificial Sedimentation. — In Chapter IX the marked effect of natural sedimentation upon the character of water in rivers and lakes was pointed out, — such effect, for example, as may be observed in any natural lake or pond fed by sediment-carrying streams. Where the body of quiescent water is sufficiently large, and the period of repose sufficiently long, this action of sedimentation becomes practically perfect, and a clear and greatly improved water is the result. Artificially, such high efficiency is often obtained where the water is collected in large impounding-reservoirs holding several months' supply. Where, however, the supply is taken directly from a large sediment-bearing stream, very large reservoirs are usually impracticable on account of the great cost; and the period of time during which sedimentation can be operative must therefore be limited to a few days or even to a few hours. Such a limited amount of sedimentation is, however, of much value.

In general the longer the time of storage within practicable limits the better the result; but the value of large reservoirs lies not only in the length of time allowed for settlement, but also in the opportunity thus afforded for shutting off the river supply at times of great turbidity. This is an especially valuable feature in the case of streams of moderate size where the high-water stage lasts but a few days. Its value in water purification has been long recognized in England. In cities where an elevated location can be found for a storage-reservoir so that it may also act as a distributing-reservoir, the advantages above noted, together with those pertaining to the matter of distribution, would

* Weston. Report on Filtration at New Orleans, 1903, p. 171.

properly lead to the adoption of relatively large sizes. (See Chapter XXVII for further discussion of distributing-reservoirs.)

Where a water contains little that is objectionable besides the inorganic sediment, a degree of purification can often be obtained by mere sedimentation which will render the water fairly acceptable. In many instances, however, a satisfactory water cannot be obtained without subsequent filtration; but in this case the process of sedimentation constitutes a very valuable and almost indispensable prerequisite to the final treatment. For a sewage-polluted water, sedimentation alone is an inadequate treatment, as the bacteria are not eliminated in sufficient numbers to insure safety.

464. **Methods of Sedimentation.** — There are two methods to be considered: (1) Plain sedimentation; (2) Sedimentation with the addition of a coagulant.

PLAIN SEDIMENTATION.

465. **Action of Subsidence.** — The particles of sand and clay have a specific gravity of about 2.6; they are therefore held in suspension only by virtue of the currents maintained in the water. When these currents become retarded the suspended matter is gradually deposited, the rate of settling varying with the size and form of the particles. The weights of similar particles are proportional to the cubes of their diameters, while the surface areas are proportional to their squares; consequently the relative resistance to sedimentation is much greater with fine particles than with coarse. Very weak currents may be sufficient to hold fine particles in suspension, while the coarser material readily settles. To cause the deposition of the finer sediment it is therefore necessary for the water to be brought as nearly as possible to a state of rest. In the case of the Missouri and Mississippi River waters, and those of similar clay-carrying streams, complete clarification by simple sedimentation is impossible at certain seasons of the year, owing to the extremely attenuated and colloidal character of the clay particles. This condition of the clay is considered by some as approaching the condition of solution, — requiring at least some agglomeration or coagulation before sedimentation can take place. Water from the Covington, Ky., reservoir which had settled about 30 days was found by Fuller to contain as high as 50 parts per million of clay.

466. **Time Required for Subsidence.** — The time required for satisfactory sedimentation is very different for different waters, and to determine this period recourse must be had to actual experiments. For some waters it requires weeks and even months to remove all the tur-

bidity, while for others a settlement of a day or two accomplishes fairly good results. If the amount of suspended matter is measured by weight, a large proportion will settle in one or two days; but the reduction in turbidity is not correspondingly great, as it is the finer portions which exert the greatest influence upon the appearance of a water. When the purpose of plain sedimentation is to prepare the water for further treatment, a high degree of clarification is not needed, it being more economical to perfect the process by other means. The best period of sedimentation will thus depend upon the character of the raw water and the relation of the sedimentation to the operation of the entire plant.

For plain sedimentation a period of 24 hours' subsidence is about the minimum limit adopted, although in some cases a still shorter period may be advisable. At Cincinnati it is planned to allow about three days, the treatment being intended as a preparation for filtration. The rate of improvement at Cincinnati is indicated by the results of some experiments on small settling-tanks. The average removal of suspended matter was as follows :*

Time of Subsidence.	Amount of Suspended Matter Removed.
24 hours	62 per cent.
48 "	68 "
72 "	72 "
96 "	76 "

The percentage of removal was greatest when the amount of suspended matter was greatest.

At Louisville, Fuller concludes that the economical limit of plain subsidence is about 24 hours, during which time 75 per cent of the suspended matter is removed. Further preparation is there deemed necessary for filtration. At Kansas City about 83 per cent of the suspended matter is removed by 24 hours' subsidence.

At New Orleans the sediment is unusually fine, average results obtained by the experiments of Weston being as follows : †

Period of Subsidence hours.	Suspended Matter.	
	Parts per Million.	Per cent Removed.
0	650	0
12	435	33
24	360	45
48	300	54
72	265	59

* Report on Water Purification at Cincinnati, p. 126.

† Report on Water and Sewage, 1903, p. 101.

In this report it was proposed to use rapid filters with alum as a coagulant and it was estimated that for such purpose the economical period of plain sedimentation would be from 12 to 24 hours. The plans adopted, however, employ sulfate of iron and lime, partly in order to effect a softening of the water. The period of plain sedimentation provided for is only about one hour.

467. **Bacterial Efficiency of Sedimentation.** — In discussing the bacterial efficiency of plain sedimentation, it must be remembered that any data gathered under ordinary conditions may possibly be misleading, as in the case of natural sedimentation in lakes, because of the operation of other factors, such as light, etc., that may also act as more or less effective agents in purification. The effect of sedimentation alone can be most accurately determined by considering the phenomenon as occurring in covered reservoirs where the direct disinfecting action of light and its indirect effect as modifying the development of algæ might be excluded. This condition precludes the study of the question on a large scale, as it is impracticable to cover reservoirs that are large enough to permit of storage for a considerable period of time. In lieu of any data under these conditions reference must needs be made to studies on sedimentation in open reservoirs. The conclusions drawn from such studies are strictly applicable only to reservoirs under like conditions.

The monthly results obtained by the Chelsea Water Company of London in 1896 are given in Table No. 61.* (Time of storage twelve days.)

TABLE NO. 61.

BACTERIAL EFFICIENCY OF STORAGE AND FILTRATION, CHELSEA WATER CO., LONDON.

Month.	Bacteria per Cubic Centimeter.		
	River Thames.	After 12 days' Storage.	After Filtration.
January	11,560	1360	20
February	26,800	460	44
March	18,000	240	28
April	7,520	...	4
May	2,060	140	24
June	6,760	1150	178
July	2,220	420	20
August	1,740	200	18
September	4,300	140	2
October	39,760	340	8
November	8,560	280	12
December	160,000	854	55
Average	24,107	508	34

Average percentage reduction by subsidence, 97.85.

Average percentage reduction by subsidence and filtration, 99.86.

Results obtained by other London works ranged from 49 to 85 per cent average reduction for periods of subsidence varying from 3.3 to 15 days.

* Hill. Public Water-supplies, 1898, p. 144.

The effect of the time factor in sedimentation is more clearly seen by reference to the data given in Table No. 62 relating to experiments made at the St. Louis settling-basins.*

TABLE NO. 62 — BACTERIAL RESULTS OF STORAGE AT ST. LOUIS.

Time of Standing.	No. of Bacteria per c. c. at Different Depths.		
	2.5 Feet.	5 Feet.	7.5 Feet.
0	6510	1970	2960
1 day	6290	2990	880
2 days	230	320	200
3 days	200	200	200

The effect of long subsidence is shown by the following typical figures relating to the number of bacteria in the Ohio River water as supplied to Cincinnati, Ohio, and to Covington, Kentucky. In Cincinnati the water had little or no sedimentation, while at Covington the large reservoir furnished about 30 days' subsidence.†

TABLE NO. 63.— BACTERIAL RESULTS OF STORAGE AT CINCINNATI AND COVINGTON.

Date.	Bacteria per Cubic Centimeter.		
	Cincinnati.	Covington.	Reduced by Sedimentation, Per cent.
January 23, 1896	1599	194	87.87
February 4, 1897	1656	53	96.80
February 17, 1897	684	20	97.08
February 26, 1897	1436	102	92.90

An experiment on the efficiency of plain sedimentation was carried out by Mr. J. W. Hill upon one of the divisions of the Fairmount Park Reservoir, Philadelphia, having a capacity of 3,346,000 gallons. Raw water from the Schuylkill River was pumped in and allowed to rest for three to four weeks. Results of two such experiments were as follows: ‡

Days' Subsidence.	Test No. 2.		Test No. 3	
	Turbidity, Parts per Million.	Bacteria, No. per c. c.	Turbidity, Parts per Million.	Bacteria, No. per c. c.
Raw water.	90	24,000	12	5700
5	35	18,500	10	1450
8	25	2650	10	500
11	30	400	10	145
14	25	400	12	35
17	30	415	9	45
20	35	250	9	60
21	...	...	15	63
24	35	975	...	...

* New York State Board of Health Report, 1893, p. 711.

† Report of Engineer Commission of Cincinnati Water-works, 1896, p. 15.

‡ Jour. Assn. Eng. Soc. 1903, xxx. p. 246.

At Cincinnati Fuller found in his experiments that about 75 per cent of the bacteria were removed by three days' subsidence.* On the other hand the experiments at New Orleans showed very little effect upon the bacterial content of Mississippi River water after from 12 to 72 hours' subsidence.

Notwithstanding there is a marked degree of purification from long periods of subsidence, yet it should be kept in mind that such a method of purification is extremely hazardous, especially where the water-supply is subject to any sewage-pollution. This is strikingly shown in the case of the Lawrence reservoirs that used to hold from 10 to 14 days' supply and in which there was a reduction of about 90 per cent of the bacteria present in the river-water; still such purification was insufficient to protect the supply, as is evidenced by the fact that the typhoid death-rates of this town were exceptionally high for many years.

468. **Bacterial Content of Reservoir Sediment.** — It must be remembered that while subsidence removes bacteria from the water, it is only to accumulate them in relatively larger quantities in the mud and ooze at the bottom of the reservoir (Art. 180). Not only do they accumulate here from deposition from superincumbent waters but actual growth occurs in abundance in the rich organic matter of lake bottoms. Russell † found the germ content in the water and mud of the bay of Naples to be as given in Table No. 64:

About one-half of the species present in the mud were indigenous to this habitat, while the remainder were common to both water and mud. He finds that the same principle also obtains in waters of fresh-water lakes. In the layer of ooze taken from the reservoirs of the Altona Water-works,‡ which is supplied with Elbe River water, there were found 17,000,000 bacteria per c.c., while the water just over this slime had upwards of 1,000,000 for same volume.§

TABLE NO. 64.

BACTERIA IN WATER AND MUD OF BAY OF NAPLES.

Depth of Water. Meters.	No. of Bacteria per Cubic Centimeter.	
	In Water.	In Mud.
50	121	245,000
100	10	200,000
200	59	70,800
300	5	24,000
400	30	22,000
500	22	12,500
1100	...	24,000

* Fuller. Cincinnati Report, p. 128. † *Zeit. f. Hyg.*, 1891, XI. p. 177.

‡ *Cent. f. Bakt.*, 1898, XVI. p. 881.

§ See also Lortet, Pathogenic Bacteria in Mud of Geneva Lake. *Cent. f. Bakt.*, 1891, IX. p. 709.

469. Experimental Data on the Action of Finely Divided Matter in Water. — Abundant experience has demonstrated the value of copious amounts of suspended matter in purifying waters by sedimentation. In the light of this fact, the following experiments made by Frankland * are of interest in showing the value of different solids in purifying water by agitation and subsequent subsidence. These trials were confined for the most part to laboratory conditions, but they illustrate the principle involved in this type of purification. Water was shaken up for a definite length of time with finely divided sterilized material of uniform size and then allowed to clarify itself by sedimentation. The clear supernatant water was then examined bacteriologically with the results given in Table No. 65.

TABLE NO. 65.

PURIFICATION OF WATER BY AGITATION WITH FINELY DIVIDED SOLID MATTER AND SUBSEQUENT SUBSIDENCE.

	Spongy Iron.	Chalk.	Animal Charcoal.	Vegetable Charcoal.	Coke.
Amt. of suspended matter (by wgt.)	1:10	1:50	1:50	1:50	1:50
No. of bacteria per c.c. before treatment	609	8000	8000	3000	very large
No. of bacteria per c.c. after treatment for 15 minutes	63	270	60	120	0
Percentage reduction	90	97	99	96	100

SEDIMENTATION WITH COAGULATION.

470. The Use of Coagulants. — Various chemicals when added to water will combine with certain substances ordinarily present, forming precipitates which are more or less gelatinous in character. These act as coagulants to collect the finely divided suspended matter into relatively large masses which are thus much more readily removed by sedimentation or filtration. Color may also frequently be removed to a large extent by this treatment. The use of coagulants in water purification was at first almost entirely confined to their employment in connection with rapid filters (see Chapter XXII), but the great advantage of their use in connection with the subsidence of turbid waters makes them of value whatever the subsequent process may be. In some notable instances sedimentation thus aided has been found to be sufficient without further treatment. Where waters are very turbid it will usually be more economical to allow the coarser sediment to settle

* Proc. Roy. Soc., 1885. Proc. Inst. C. E. 1886, LXXXV. p. 197.

before the application of a coagulant, as in this way the amount of chemical required is much reduced. Occasionally, also, double sedimentation with the use of coagulants in both cases may be advisable.

471. **The Action of Various Coagulants.** — *Sulfate of Alumina.* — Several substances can be used as coagulants. That most commonly employed is sulfate of alumina. When this substance is introduced into water containing carbonates and bicarbonates of lime and magnesia, it is decomposed, the sulfuric acid forming sulfates with the lime and magnesia, while the carbonic acid is set free, and the alumina unites with water to form a bulky gelatinous hydrate which constitutes the coagulating agent. According to Fuller, part of this hydrate may be absorbed by the clay particles before much coagulating action takes place, the amount absorbed depending upon the amount and character of the sediment. If more sulfate is used than can combine with the quantity of carbonates present, it will remain dissolved in the water, a result which is necessary to avoid on account of the possible injurious effect of the alum. If the water does not naturally contain a sufficient amount of alkalinity to decompose the necessary amount of coagulant, lime should be previously added to the water. Theoretically, one grain of sulfate will decompose about 8 parts per million of $CaCO_3$ or its equivalent, but owing to the absorptive action previously mentioned the actual reduction of alkalinity is likely to be less. Experiments at Louisville and at New Orleans indicate a reduction of alkalinity of from 65 to 90 per cent of the theoretical amount. It was also shown that much more coagulant was required with fine sediment than with coarse.

Accompanying the reduction of the carbonates is an equal increase in the sulfates of lime and magnesia. As these compounds form the objectionable incrusting constituents or the permanent hardness of a water, this change is detrimental. With the ordinary quantities of coagulant used, such as 1 to 2 grains per gallon, this increase in hardness would amount to from 9 to 18 parts per million, not relatively a very important matter, and probably much overbalanced by the gain in clearness of the water. This objectionable increase in the permanent hardness may be avoided by the use of sodium carbonate instead of lime (Art. 558).

The amount of carbonic acid set free is equal to 44 per cent of the decrease in carbonates. This acid remains absorbed in the water and increases its corrosive action on unprotected iron plates, which is, however, not a serious matter. If the carbonate is all, or nearly all reduced, there is more danger of solvent action on lead pipes.

Iron. — Since about 1903 the use of iron as a coagulant has been rapidly developed. Ferric hydrate has long been known to be an effective coagulant, acting in a manner similar to the aluminum hydrate. Ferric hydrate can readily be produced by the use of ferric sulfate, but this is impracticable on account of the expense involved. Another way of obtaining this hydrate is by the use of metallic iron, as in the Anderson process described later (Chapter XXIII). In this case the metallic iron forms ferrous carbonate with the carbonic acid present, which in turn oxidizes to the ferric hydrate from the oxygen dissolved in the water.

More recently the iron solution has been furnished by the direct absorption of fumes of burning sulphur by water containing scrap-iron. This process, patented by the Jewell Filter Company, has been successfully used in several plants in the Middle West. A still more promising method and one now (1908) in successful use in several places, notably at St. Louis, is the use of ferrous sulfate and caustic lime in the form of milk of lime. In this process, as in the alum process, the sulfuric acid unites with the lime and magnesia present forming soluble sulfates. Without the addition of caustic lime the iron would form a carbonate which would change to the hydrate but slowly. The lime unites with the free CO_2 present thus greatly hastening the process, and at the same time precipitating part of the lime present, $CaCO_3$, in same manner as in the lime softening process. (See Chapter XXIII.) Very soft waters require a more exact proportioning of chemicals than waters somewhat hard, as in the latter case any excess of lime serves only to partially soften the water. Waters containing vegetable coloring matter are likely to give trouble by retaining the iron in solution in the same manner as certain ground-waters which contain iron. (See Art. 563.) In the case of hard waters the element of softening may become an important feature and the amount of lime increased to attain this object. This needs to be done with caution as the resulting precipitate of $CaCO_3$ is likely to be troublesome to deal with because of its tendency to clog pipes and channels.

The ferric hydrate seems to be quite as efficient a coagulating agency as aluminum hydrate, and as its cost is considerably less, the iron and lime process is likely to be more economical in those waters where experiments show that it can be used with success. On the other hand, sulfate of aluminum appears to be of more general applicability for waters of all kinds.

Other Coagulating Agencies. — Both the aluminum and the ferric hydrate can be produced electrolytically from the metals. The expense of metallic aluminum as compared to the sulfate precludes the use of that metal, but it is possible that in some cases the ferric hydrate might be economically produced in this way. (For further discussion see Chapter XXIII.)

Lime is another substance that may be used as a coagulant. When used in the ordinary Clark process for softening water the effect is considerable, but still greater effects can be obtained by using lime in moderate excess. Naturally the pulverulent precipitate of lime carbonate is generally not nearly as effective as the gelatinous alumina precipitate. Experiments involving this process at Cincinnati, Ohio, showed the following average results : *

	River Water.	Effluent.	Per cent Removed.
Suspended matter (parts per 1,000,000)	273	35	87.2
Bacteria (per c.c.).	23,800	1300	94.5

The average period of subsidence was about 14 hours, and the average amount of lime used was 4.7 grains per gallon, of which 3.1 grains was in excess of the amount required to combine with the bicarbonates. This excess of lime involves a further process for its removal, which may consist in the addition of carbonic acid.

In some cases the coagulating agent, if added in large quantities, produces not only the mechanical effect of sedimentation due to the settling of the precipitate, but the excess of chemical used may act as a direct germicide on the bacteria present. Such treatment, however, is inapplicable for water-purification, although it is sometimes used in the treatment of sewage.

472. **The Amount of Chemical Required.** — This depends upon the amount and character of the sediment, upon the degree of purification desired, and upon the time of settlement. It varies in practice from about $\frac{3}{4}$ grain to 3 or 4 grains of sulfate per gallon. The proper amount can only be determined by experiment. Some idea of the amount required can be had from the data of Table No. 66. This gives the approximate amount of chemical required for the Ohio River water at Cincinnati,† the Allegheny River water at Pittsburg,‡ and the Mississippi River water at New Orleans, § as a preparation for filtration. In general the more chemical used the greater the effect, and by using a sufficient quantity and allowing enough time for sedimentation a

* Cincinnati Report, p. 483.

† Cincinnati Report, pp. 290, 341.

‡ Report of the Pittsburg Filtration Commission, 1899.

§ Report on Water and Sewerage, 1903, p. 130.

clear water can be secured. But the question of economy will usually limit the efficiency obtained, and where the process is but a preliminary treatment a high degree of efficiency is not necessary. At New Orleans it is estimated by Weston that, with a preliminary period of sedimentation of twelve hours, an amount of coagulant should be used sufficient to reduce the suspended matter to about forty-five parts, requiring a maximum of about twelve hours further sedimentation. This treatment is supposed to be followed by rapid filtration.

TABLE NO. 66.

ESTIMATED AVERAGE AMOUNTS OF REQUIRED CHEMICAL FOR DIFFERENT GRADES OF WATER.

Suspended Matter, Parts per Million.	Sulfate of Alumina Required, Grains per Gallon.					
	Raw Water for Sand Filters, Cincinnati.	Subsided Water for Sand Filters, Cincinnati.	Subsided Water for Rapid Filters, Cincinnati.	Minimum for Raw Water for Rapid Filters, Pittsburg.	Subsided Water for Rapid Filters, New Orleans.	Unsubsided Water for Rapid Filters, New Orleans.
10	0	0	0.75	0.40	...	...
25	0	0	1.25	0.50	...	...
50	0	0	1.50	0.70	1.70	...
75	0	1.30	1.95	0.90	1.85	...
100	1.50	1.60	2.20	1.00	2.10	...
125	1.60	1.80	2.45	1.15	2.30	...
150	1.70	2.00	2.65	1.30	2.45	3.00
175	1.80	2.10	2.85	1.40	2.60	3.15
200	1.95	2.20	3.00	1.60	2.70	3.30
300	2.25	2.45	3.80	2.00	3.40	3.95
400	2.50	2.75	4.40	2.50	4.10	4.65
500	2.80	...	...	...	...	5.65
600	3.05	...	...	...	...	6.30
750	3.40	...	...	...	...	7.45
1000	4.00	...	...	...	...	10.15
1200	4.75	...	...	...	...	....

The amount required for the Missouri River water at Kansas City, where sedimentation is the only purification employed, is given by Kiersted as follows:

AMOUNT OF CHEMICAL REQUIRED FOR THE CLARIFICATION OF THE MISSOURI RIVER WATER

Suspended Matter after 24 hours Natural Subsidence, Parts per Million.	Sulfate of Alumina Required for Clarification, Grains per Gallon.	Suspended Matter after 24 hours Natural Subsidence, Parts per Million.	Sulfate of Alumina Required for Clarification, Grains per Gallon.
50	0.0	350	2.9
100	0.5	400	3.4
150	1.0	450	3.8
200	1.5	500	4.3
250	1.9	550	4.8
300	2.4	600	5.3

* Waterworks Management and Maintenance, p. 148.

The water is subjected to preliminary natural sedimentation for 24 hours. For less than 50 parts suspended matter per million no further clarification is required.

At St. Louis iron and lime are used without preliminary sedimentation. The average amounts used in 1906–7 were 2.13 grains sulfate of iron and 7.39 grains of lime. The average amount of suspended matter is about 1200 parts per million.

The following is an estimate by Ellms of the amounts of iron and lime required for the Ohio River water at Cincinnati as compared to alum.*

AMOUNT OF CHEMICALS REQUIRED FOR THE PURIFICATION OF SUBSIDED OHIO RIVER WATER.

Turbidity, Parts per Million.	Sulfate of Alumina, Grains per Gallon.	Sulfate of Iron and Lime.	
		Sulfate of Iron, Grains per Gallon.	Caustic Lime, Grains per Gallon.
10	0.75	1.00	0.75
25	1.25	1.25	0.90
50	1.50	1.40	1.00
75	1.95	1.50	1.10
100	2.20	1.60	1.20
125	2.45	1.75	1.30
150	2.65	1.90	1.40
175	2.85	2.10	1.50
200	3.00	2.25	1.70
300	3.80	2.50	1.90
400	4.40	3.00	2.00

473. **Time of Subsidence.** — The rate of sedimentation depends greatly upon the amount of coagulant employed. It takes place much more quickly than where no coagulant is used, so that a large part of the action will occur in a few hours. Where the process is preliminary to rapid filtration the period allowed is usually from two to six hours. In this case it is not desired that perfect clarification shall be secured, as better results will be obtained from the filters if a small amount of the flocculent coagulant be carried over to the filters ; but too large an amount of sediment increases the cost of filtration more than the decrease in cost of sedimentation. At New Orleans, where the sediment is very fine, a period of twelve hours is estimated by Weston to be somewhat more economical considering the entire process than six hours, but the difference is not great. The new plant provides for about six hours using iron and lime. At St. Louis, where the process is final, the total capacity of the series of basins is about two days'

* *Eng. Record*, 1906, LIV. p. 441.

supply. This is now (1908) being increased by 50 per cent by the construction of two additional basins. The question of amount of chemical needed, time of subsidence and degree of purification desired are intimately related, and the best and most economical arrangement must be worked out for each case individually.

474. **Efficiency of Sedimentation with Coagulation.** — As previously stated, the efficiency is a function of the time, amount of coagulant, and character of the sediment. The bacterial efficiency follows in a general way the efficiency with respect to the suspended matter. Where used as a preliminary process there is usually no difficulty in securing a sufficient degree of clarification in a few hours, either with or without preliminary natural sedimentation, the only question being that of amount of coagulant and cost of operation. As already stated, the amount of suspended matter which may economically be left to be taken care of by the filters is estimated at 30 to 45 parts per million at New Orleans; at Cincinnati Mr. Fuller considered it advisable to apply further preparatory treatment than plain sedimentation where the amount of sediment exceeded 40 or 50 parts per million. Where the process is final the absolute efficiency, both with respect to suspended matter and bacteria, is of great importance. Ordinarily, with waters containing clay, it will be difficult to reduce the suspended matter below 20 parts per million in a reasonable time. Probably the most successful plant in this respect is that at St. Louis. Here the water is treated with iron and lime and flows through a series of six basins originally operated for plain sedimentation. The average results for a year are as follows in parts per million:

	River.	Treated Water.
Solids in suspension	1188	3.8
Color	43	10.7
Alkalinity	135	49
Calcium	42.3	22.8
Magnesium	13.1	4.5

A relatively large amount of lime was used, thus causing a considerable softening effect. The average results for March, 1907, were as follows:

	River.	Weir 1.	Weir 3.	Weir 5.	Tap.
Suspended matter	1444	14.2	8.35	5.8	2.56
Color	45.3	13.6	13	11.6	10.8
Alkalinity	133	51.1	47	45	41
Bacteria per c.c.	57029	933	501	106	42

The several "weirs" are the effluent weirs of the several basins operated in series. The very high results obtained with reference both to the suspended matter and bacteria are noteworthy. Further data show an average percentage removal of bacteria for the three months, January, February and March, 1907, of 98.87, 98.8 and 99.88 respectively, from raw water to tap. This result is quite comparable with the best filtration.* These results as to bacteria are better than are generally secured, but a very large degree of purification is obtained at all times.

In the case of the usual sedimentation of two to six hours secured in connection with rapid filtration the reduction in bacterial content will usually range from 50 per cent to as high as 90 or 95 per cent. The latter figure is unusual and occurs only where the absolute number in the raw water is high. Bacterial examinations of water from the settling-tanks connected with rapid filters at Louisville showed a removal in the very short time there allowed for sedimentation (less than one hour) of ordinarily from 40 to 75 per cent of the bacteria. The removal of other suspended matter was scarcely more than this. When allowed to stand overnight, or over Sunday, the removal of bacteria and suspended matter was practically always over 90 per cent. With large amounts of coagulant, such as 5 or 6 grains per gallon, very high efficiencies may be reached.

In general it may be said that the results of sedimentation with coagulation are not sufficiently good to make this a safe process to apply, without further treatment, to a sewage polluted stream, although the work at St. Louis indicates that under certain circumstances very satisfactory results may be secured. Many waters can be satisfactorily clarified in this way, but in the case of some waters perfectly satisfactory results cannot readily be secured without filtration.

The removal of color depends much upon the nature of the water. Usually from 70 to 90 per cent of the color of ordinary waters may be removed by suitable quantities of chemical, but some waters, especially those having a high color, cannot readily be decolorized in this way. At Superior, Wis., the use of four grains of sulfate per gallon had no appreciable effect on a water having a color of about 2 on the platinum scale.

SETTLING-BASINS.

476. Settling-basins are constructed in accordance with the same general principles as other reservoirs; in fact, in many cases, distributing-reservoirs or storage-reservoirs act also as settling-basins. Where,

* See valuable paper by Edward E. Wall on "Water Purification at St. Louis, Mo.," in Proc. Am. Soc. C. E., Sept. 1907, p. 758. Also *Eng. Record*, 1907, LVI. p. 98.

however, but a short time is allowed for settling, and reservoirs are intended for that special purpose, there are differences in detail which should be considered. Settling-basins are usually supplied with water by means of low-service pumps, and from the basins the water flows into an equalizing clear-water reservoir, or to a pump-well, or to filters, as the case may be.

477. **Methods of Operation.** — There are two general methods of operating settling-basins: (1) the continuous-flow method, and (2) the intermittent or fill-and-draw method. In the former the water is allowed to flow at a very slow velocity through one or more reservoirs, during which time the settling takes place. In the latter, the water is let into a basin and allowed to remain quiescent during the period of subsidence. It is then drawn off to as low a level as efficient clarification has taken place, and the basin refilled. The method of fill-and-draw, formerly used at St. Louis for plain sedimentation, has been changed to the continuous-flow method with coagulation. At Cincinnati, Ohio, the fill-and-draw method is used, but it is stated that this is on account of matters pertaining to the form of the basins which are purely local in character. In the fill-and-draw method no settlement of fine particles can commence until the operation of filling is completed, which condition materially reduces the time of subsidence. On the other hand the water becomes more quiet than in the other process, and this operates to its advantage.

Independent of the question of clarification, a disadvantage of the intermittent-flow method is in the loss of head occasioned by its use. Thus the highest level of the water in the clear-water basin or in the filters must be as low as the lowest point at which the water is drawn off. This not only increases the expense of pumping, but is an arrangement not always easy to make. In the continuous-flow system practically no head need be lost in the settling-basins. It should be noted, however, that basins on the fill-and-draw method can be utilized more or less as storage-reservoirs, which cannot be done with the others. This gives more elasticity to the system and admits of a freer operation of the supply-pumps.

Generally speaking the continuous-flow system is the more advantageous and is the system now almost universally employed where the water is given a relatively brief period of sedimentation with the aid of a coagulant.

478. **Number and Size of Basins.** — If the basins are operated on the continuous system, a single basin can be made to suffice, an arrangement quite suitable for a relatively clear water where sedimentation is a

secondary matter, or merely a preparation for filtration. If there is much sediment, at least two basins are needed, in order that one may be cleaned without interrupting the supply. It is found also that generally better results can be obtained by the use of two or three basins in series than by the use of a single one of the same total capacity. While this effect can be secured by inexpensive partitions in a single basin, yet convenience in the removal of sediment makes it desirable to have at least two and often three independent basins. Where a coagulant is used after partial sedimentation at least three would be necessary for convenient operation.

With the fill-and-draw method, the number becomes a question of economical construction and operation. The basin being filled is not effective, and that being drawn from may be counted as one-half effective, so that if q is the capacity of each, A the volume of consumption for the selected time of settlement, and n the number of basins, then

$$n = \frac{A}{q} + 1\tfrac{1}{2};$$

that is, the required number is equal to the fixed volume A divided by the capacity q, plus $1\frac{1}{2}$. The larger the value of q, the lower will be the cost of the $\frac{A}{q}$ basins, but the higher will be the cost of the extra $1\frac{1}{2}$ basins. The best capacity and number can readily be determined by trial estimates. At St. Louis, the best number was found to be from 6 to 8.

479. **Form of Basin.**— For a single rectangular basin of given area the square is the most economical form. For a number of basins the best proportions may be determined by trial estimates, but the following analysis will be of some assistance in arriving at an approximate solution:

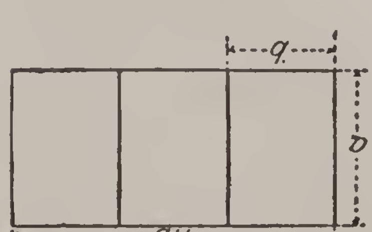

FIG. 117.

Let n = number of basins, each of which has a width b and length a (Fig. 117). Let c_e = cost per lineal foot of exterior wall or embankment, and c_i = cost of interior wall or embankment. Then if C = total cost of embankments, we have

$$C = (2nb + 2a)c_e + (n - 1)ac_i. \quad . \quad . \quad . \quad . \quad (1)$$

Let $ab = A$, a constant quantity. Then $b = \frac{A}{a}$. Substituting this value in the above equation, we have

$$C = \left(2n\frac{A}{a} + 2a\right)c_e + (n - 1)ac_i \quad . \quad . \quad . \quad . \quad (2)$$

Differentiating with respect to a, equating to zero, etc., we find that for a minimum value of C the value of a is

$$a = \frac{2\,nbc_e}{2\,c_e + (n-1)c_i}, \text{ whence } \frac{b}{a} = \frac{2 + (n-1)\dfrac{c_i}{c_e}}{2\,n}. \quad . \quad . \quad (3)$$

If $c_e = c_i$, then $\frac{b}{a} = \frac{n+1}{2\,n}$; which gives for $n = 2$, $b = \frac{3}{4}\,a$; for $n = 3$, $b = \frac{2}{3}\,a$, etc.

The above results are seen to be independent of the area, and hence are true for basins or reservoirs of any size, arranged in the manner shown.

In general, settling-basins, where large, are built similar to ordinary reservoirs, partly in excavation and partly by embankment, so as to secure the greatest economy. Earthern slopes will usually be cheaper than masonry walls, but with the fill-and-draw method the former have the disadvantage of exposing the mud at each period of emptying. They are, however, more often used. Where built for use as coagulating basins in connection with filters, they are, in the case of small plants, frequently built as part of a structural unit, being made with masonry or concrete walls and possibly floored over.

The depth of basins is made about such as to give the most economical construction, very shallow basins being avoided. The time of settlement is found not to be materially affected by depth.

480. Arrangement of Pipes, Continuous-flow System. — The object to be attained in this system is the distribution of the water on entering as evenly as may be across one side or one end so that it shall enter with as little disturbance as possible; then to draw it off in a similar manner from the opposite side, and from the stratum of clearest water. As far as possible all parts of the water should remain in the basin equal lengths of time, and all strong currents should be avoided. A common form of inlet consists in a single large pipe laid through the embankment, or a single sluice-gate in a gate-chamber built in the walls.

A much better distribution of the water is obtained by means of numerous inlets, or numerous branches from a single inlet conduit, and several of the later works have been arranged in this way. For this purpose a concrete conduit may be used, built within the reservoir, or just back of the face, and provided with numerous openings. The maximum uniformity of flow will usually be secured if the water is admitted near the bottom. This is especially the case in summer when the entering water is apt to be cooler than the surface water in the reservoir.

The withdrawal of water in this system should take place from near the surface. Broad weirs formed in the wall, or made of iron troughs, are frequently used. Instead of weirs, a series of vertical pipes open at the upper end may be used, as in the Albany settling-basin described on page 467. At Denver the water flows off over a very large number of circular weirs fitted to vertical effluent pipes. Outlet conduits of concrete, arranged as above described for inlet conduits, constitute a convenient arrangement. (See Art. 484, for example.)

If a perfectly uniform movement can be secured, a single large basin will be as efficient as any other arrangement. On account, however, of the effect of wind, temperature changes, and variation in flow, in causing irregular currents, and hence a more or less mixing of the entire contents of a single reservoir, there are some advantages in separating the process into parts so as to prevent the more turbid water from mixing with the less turbid. This can be done by using two or more reservoirs in series, or, less perfectly, by placing baffles or light wooden partitions in a single reservoir, or by constructing a single reservoir very long and narrow. The general effect of such arrangements is to increase the average velocity of flow, but up to a certain point the effect of this increase is more than balanced by the beneficial effects of separation above mentioned. Undoubtedly a certain amount of subdivision by means of baffles is often advantageous where the water enters or leaves at a single point, as otherwise there is certain to be much inequality in the velocities. Where baffles or a series of reservoirs are used the operation of each division should be arranged according to the same principles as apply to the single reservoir. Inlets near the bottom and outlets near the top are preferable, but baffles are quite commonly arranged merely to guide the water in a circuitous path through the basin at full depth at all points, thus making in effect a long and narrow but tortuous channel.

Where a series of compartments is thus used the first one may economically be operated at much higher velocities than the following. Theoretically the maximum efficiency would be secured when the velocity of the water is progressively less as it moves forward in the series, since the sediment remaining becomes progressively finer. As to the number of such basins in series, it will seldom be economical to use more than two or three. At St. Louis, where six basins were available when the continuous process was adopted, the results obtained at the successive weir outlets is represented by the following averages for March, 1907:*

SUSPENDED SOLIDS, PARTS PER MILLION.

River.	Weir 1.	Weir 2.	Weir 3.	Weir 4.	Weir 5.	Weir 6.
1444	14.2	12.1	8.35	7.1	5.8	5.46

The reduction beyond the third basin is very small.

481. **Arrangement of Pipes, Intermittent System.** — In this system, since the water may enter rapidly, the inlet is arranged in the simplest way, as in an ordinary reservoir. The position of the outlet is of more importance. If but a single one is used, it will need to be at the lowest point of outflow, and so will not draw from the clearest stratum except near the end of the operation. The difference in clearness at different depths after 24 hours' subsidence or more is, however, not very great. At St. Louis, observation showed that there was very little difference in clearness from top to bottom, and but a single outlet was there provided. Experiments at Cincinnati † showed that the upper 6 inches was considerably clearer than the water lower down, but that below this there was little change. The results of the experiments are given in Table No. 67. The time of settlement was 72 hours.

TABLE NO. 67.

EXPERIMENTS ON SEDIMENTATION AT CINCINNATI.

Depth of Sample, Feet.	Suspended Matter, Parts per Million.	Percentage Removal.
0.25	137	78.8
3.00	190	70.3
8.00	195	69.5
13.00	197	69.2
23.00	206	67.8
28.00	200	68.7
30.00	215	66.4
31.00	200	68.7
32.00	206	67.8
33.75	641	00.0

Unless the water can be drawn from very near the surface little advantage is gained in ordinary shallow basins by constructing an outlet near the top. With a depth such as at Cincinnati, however,

* *Eng. Record*, 1907, LVI. p. 98.
† Report on Purification, p. 121.

there would be some advantage in two outlets instead of one. To enable water to be drawn always from near the surface, the adjustable outlet pipe described in Chapter XXVII is used to advantage in many reservoirs, and among these the new settling-reservoirs at Cincinnati.

482. **Drain-pipes.** — To enable the sediment to be removed, the bottom of the basin should be made slightly sloping (1 to 2 per cent grade) towards a central drain leading to an outlet-gate or to a drain-pipe. The mud is removed by flushing it into the drain by means of a hose-stream, supplied from a high-pressure main. The cleaning is done at intervals depending entirely upon the local conditions, and may be every month or so, or only at intervals of years. The longer the mud is allowed to remain the more compact it becomes and the more difficult to remove, but the change in compactness takes place quite slowly.

483. **Clear-water Well.** — Where the basins are operated on the continuous-flow system and the water passes from them directly to the pumps, it is necessary to interpolate a small clear-water or pump well to avoid the necessity of too frequent adjustment of the rate of supply to the basins. It is not necessary that the operation be perfectly uniform, for the loss in efficiency due to a more rapid motion through the basins part of the time is largely compensated by a reduced rate of flow at other times.

483a. **Preparation and Control of Coagulant.** — In using a coagulant it is of the utmost importance that the introduction of the proper amount at all times be certain. This is especially true where a coagulant is depended upon in rapid filtration of sewage polluted water where the interruption of the process would endanger the health of the community. This element has been a strong argument against the use of the rapid filter for such waters. This feature of operation has, however, been so well perfected in the more recent plants that the objection has lost much of its force.

A common method of supplying a known quantity of coagulant is first to prepare the solution of known strength in independent mixing tanks, a duplicate set of these tanks being used. Then from one of these tanks the prepared solution is pumped or conveyed to a smaller orifice or dosing tank in which the liquid is maintained at a constant level, usually by applying an excess and permitting the surplus to overflow over a weir and return to the mixing tank. From this orifice tank the solution is fed through an orifice of known capacity. The head on the orifice is thus constant and the rate of flow is regulated to any desired quantity by regulating the size of orifice by hand wheel with suitable indicator. The liquid should be permitted to pass this orifice into free air and not directly into a closed pipe, as the latter arrangement would give rise to uncertainty as to head. Such apparatus must be occasionally checked to guard against the effect of corrosion or clogging from accretions of chemical. Accurate regulation requires a knowledge of the rate of flow of the water-supply as well as that of the coagulant. This is obtained from the pump counters, if pumps are used, or may be conveniently got by the use of venturi meters. While being used the contents of a mixing tank must be of uniform strength. This is accomplished by stirring with paddles, or by agitation with air, or by other mechanical means. Such agitation also aids greatly in the preparation of the solution. Lime may be used either as milk of lime or lime water, the latter requiring a relatively large amount of water in its preparation, but giving more uniform and reliable results.

In large plants where the quantities handled are large the preparation of the chemical may be more economically carried out by the continuous method, no storage of prepared solutions being required. This involves accurate and convenient means for measuring out and introducing into the mixing tanks any desired quantity of chemical and at very frequent intervals.*

The amount of coagulant needed is determined by frequent analyses of the water and by direct observation of results secured. In the use of alum it is very essential that there be sufficient lime present in the water to decompose all of the sulfate of alumina used.

In the construction of the apparatus for the preparation of the coagulant great care should be exercised to secure substantial and durable work. Bronze and rubber fittings must be used in machinery for handling alum, and pipes should be of brass, bronze or lead. Tanks and large conduits are advantageously made of reinforced concrete.

A coagulant is introduced into the water in various ways. A common and satisfactory method is to introduce the solution into the water in a conduit or channel by means of a series of perforated tubes distributed over the entire section; or by such perforated tubes placed along a weir over which the water passes. It may also be introduced just previous to where the water passes through pumps, but this method is not desirable where lime is used, as this tends to cause accretions on the machinery.

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* See Proc. Am. Soc. C. E., Sept. 1907, for description of the large plant at St. Louis.

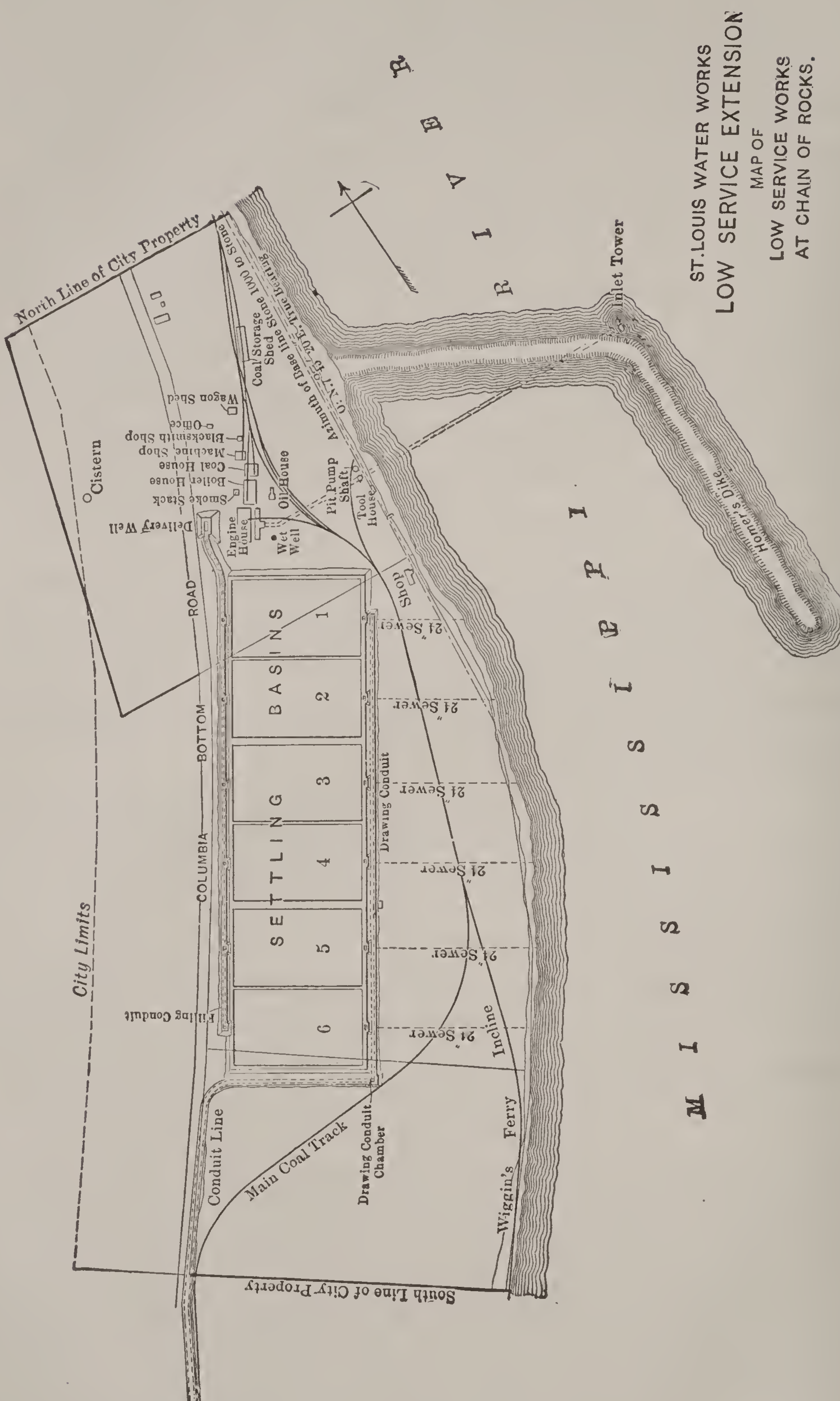

FIG. 118. — ST. LOUIS SETTLING-BASINS.
(From Proc. Am. Soc. C. E., Sept., 1907.)

484. Examples of Settling-basins. — The St. Louis settling-basins constitute the largest plant of its kind ever built. The general arrangement of intake-pumps, basins, and filling and drawing conduits is shown in Fig. 118. The basins are of 22,000,000 gallons drawing capacity each. They are built with masonry side and partition walls, and linings of concrete, on about 18 inches of puddle. Through the center runs a ditch having a slope of 1 per cent, and leading to a 24-inch drain-pipe at the east end. The floor also slopes towards this ditch from both sides. Formerly these basins were operated on the fill-and-draw system, the filling being done through a masonry conduit on the west side and the drawing through a similar conduit on the east. They are now operated as coagulating basins on the continuous system, the end basins being used alternately as supply basins, communication from basin to basin being effected by means of long weirs in the division walls.

A compact design for a settling-basin is that for the city of St. Joseph, Mo., illustrated in Fig. 119, Mr. Wynkoop Kiersted, Mem. Am. Soc. C. E.,

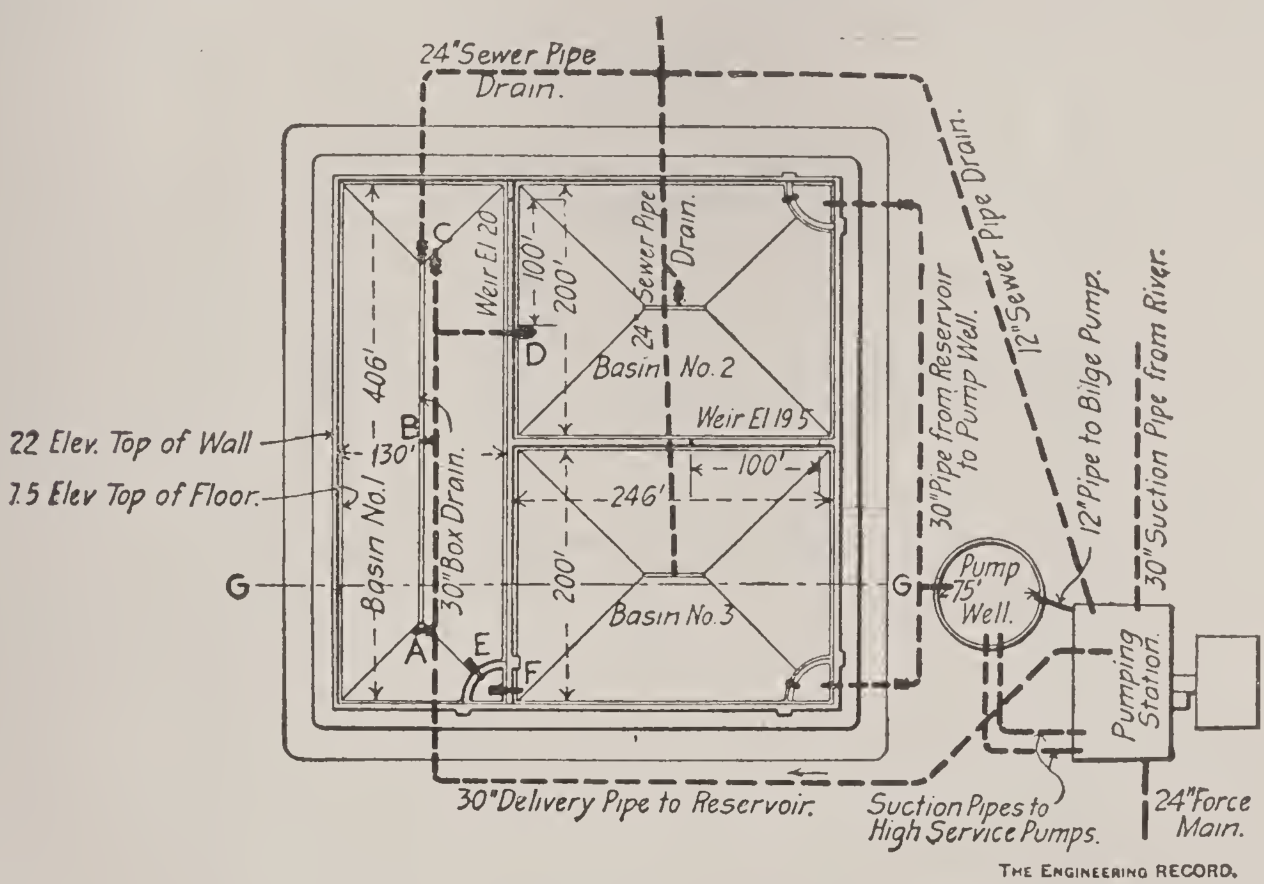

FIG. 119. — SETTLING-BASIN AT ST. JOSEPH, MO.
(From *Engineering Record*, vol. XL.)

engineer. The following description is from an article by Mr. Kiersted in the *Engineering Record*, 1889, vol. XL. p. 506.

"The water delivered by the low-service pumps enters the basin No. 1 at the points *A* and *B* either when all three basins are in operation, or when basins 1 and 2 are in operation and No. 3 is empty for cleaning; at point *C* when basins 1 and 3 are in operation, and at *D* when No. 1 is empty for cleaning. The continuous method of sedimentation is recommended; consequently communication between the basins is made by weirs." When basin No. 2 is being cleaned, water enters basin No. 1 at *C* and overflows the arch at *E* and thence passes through pipe *F* into the bottom of No. 3. It is proposed to introduce a coagulant into the water as it passes the weirs, through a line of small pipe provided with suitable openings. The bottoms of the basins slope in each case towards a central gutter from which the sewer drain-pipes lead.

At Cincinnati the arrangement of basins for secondary sedimentation with coagulation is shown in Fig. 119a. Usually each of the three basins is operated independently, the water passing through but a single basin. In all cases the water is admitted to the basin through numerous openings in an inlet conduit placed at one end and near the bottom. It is taken out through a similar conduit at the opposite end placed near the top.* The small basin No. 3 may be used when necessary for a second treatment with coagulant. The period of sedimentation may be varied from 0.4 hour to 4.7 hours.

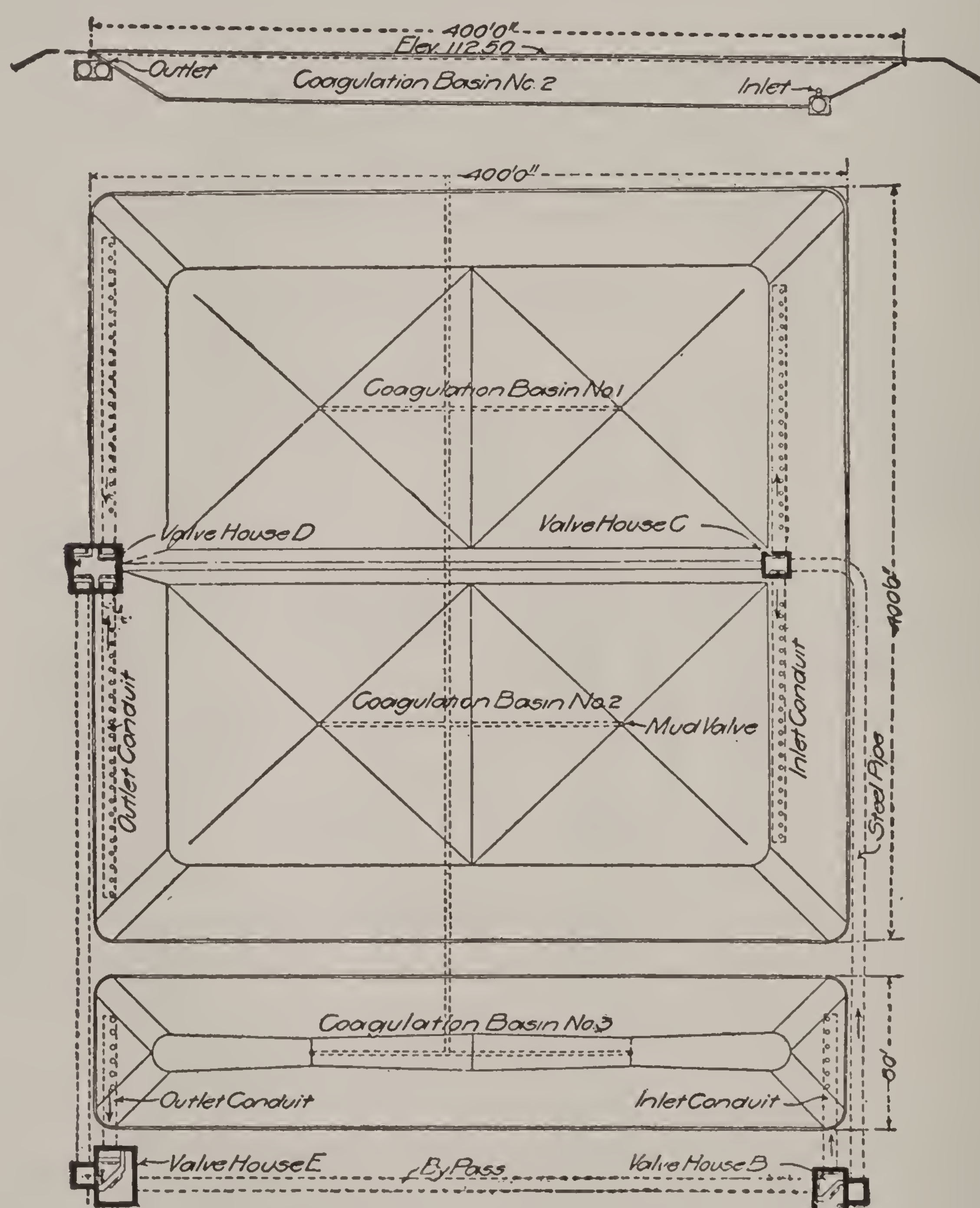

FIG. 119a. — CINCINNATI COAGULATION BASINS.
(From *Eng. Record*, vol. LV.)

At Pittsburg three basins are provided, a central receiving basin of relatively small size and two larger basins on either side. The water enters the receiving basin through numerous openings in a large conduit in the center of the basin. The velocity of entrance is low and sedimentation of the coarser particles promptly begins. From the central basin the partially settled water passes to the larger basins, likewise through a perforated con-

* *Eng. Record*, 1907, LV. p. 431.

duit extending entirely across the end of each basin. Settled water is drawn off at the opposite ends from a series of openings arranged as weirs and leading to the outlet conduit.* See also Chapter XXII for examples of coagulating basins in connection with rapid filters.

The settling-basin at Albany, N. Y., used in connection with the filter-plant, possesses several features worthy of notice. (For illustration see page 467.) The capacity is 14,600,000 gallons, or about $1\frac{1}{2}$ days' supply. The operation is continuous, water being admitted through eleven inlets along one side and flowing out through an equal number of overflow-pipes on the opposite side. The inlet-pipes rise 4 feet above the water-line and are perforated, this causing aeration of the water as it enters. An overflow is provided through a manhole as shown on the plan (Fig. 120). A waste or blow-off pipe leads from a sump near the center, towards which point the bottom slopes from all directions. As the Hudson River water is clear during a large portion of the year, the basin can be readily thrown out of service for cleaning.†

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(See also references of Chap. XIX.)

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13. The Settling Reservoirs of the New Cincinnati Water-works. *Eng. Record*, 1907, LV. p. 672.
14. Wall. Water Purification at St. Louis, Mo. Proc. Am. Soc. C. E., Sept. 1907, p. 758.

See also references 10, 11, 14, 15, 19, 22, 27 of Chapter XXII.

* *Eng. Record*, 1906, LIV. p. 622.

† For full description see Trans. Am. Soc. C. E., 1900, XLIII. p. 256.

CHAPTER XXI.

SLOW SAND FILTRATION.

485. **Historical.**—The first filter of which we have any record was established by Mr. James Simpson in 1829 for the Chelsea Water Company of London. The chief object of this filter was to remove turbidity, and in this it was a success. Its value in improving the water from a hygienic standpoint was also appreciated, although the principles underlying its action were not understood until some years later. As a consequence of the good results obtained from this filter, the filtration of all river-water supplies of London was made compulsory in 1855. Similar filter-plants were also soon established at several places on the Continent.

When efficient chemical methods of water analysis were devised about 1870 and applied to the subject of filtration, it was found that but little purification, chemically, was effected by the process. The result was disappointing, as the organic matter itself was at that time considered to be a chief cause of disease. After the establishment of the germ theory of disease and the application of modern bacteriological methods to water filtration by Prof. P. F. Frankland in 1885, the subject was put upon an entirely new and substantial basis; for it was found, fortunately, that the sand filter, although showing imperfect results from a chemical standpoint, was an excellent medium for removing bacteria. It is thus interesting and valuable to note that this process, which was developed empirically, really had a scientific foundation.

Within the last fifteen or twenty years the use of sand filters has become almost universal abroad wherever surface-waters are used. In Germany it is compulsory. It is estimated that at least 30,000,000 people are now (1907) supplied with filtered water. In the United States it is only very recently that this subject has received the attention that it merits. In view of these facts it is interesting to note that as long ago as 1869, Mr. J. P. Kirkwood wrote a most valuable report on filtration, describing therein many foreign works and recommending the adoption of the system in St. Louis. The recommendations, however, were not adopted, but in 1872 a filter was constructed at Poughkeepsie, N. Y., under Mr. Kirkwood's direction, which is still in operation. A similar one was built at Hudson, N. Y., in 1874, but no others until recently. An important step in the development was marked by the completion in 1899 of a fifteen-million-gallon plant at Albany, N. Y., the largest yet constructed at that time. Since this time progress has been rapid and some very large plants are now (1907) under construction or have recently been completed, notably for the cities of Philadelphia, Pittsburg, Washington, Cincinnati, Louisville and New Orleans. The growth in the use of filters in the United States is shown by the following statistics from Hazen.*

TABLE SHOWING USE OF FILTERS IN THE UNITED STATES.

Year.	Total Urban Population in the United States (Towns above 2,500).	Population Supplied with Filtered Water.			Percentage of Urban Population supplied with filtered Water.
		Slow Sand Filters.	Rapid or Mechanical Filters.	Total.	
1870	...	None.	None.	None.	0
1880	13,300,000	30,000	...	30,000	0.23
1890	21,400,000	35,000	275,000	310,000	1.45
1900	29,500,000	360,000	1,500,000	1,860,000	6.3
1904	32,700,000	560,000	2,600,000	3,160,000	9.7

486. Types of Sand Filters. — Sand filters are of two general types, the slow filter and the rapid filter. The former is operated at a rate of from 2,000,000 to 6,000,000 gallons per acre per day, while the latter is generally operated at a rate of from 100,000,000 to 125,000,000 gallons per acre per day. These very great differences in rate of filtration necessitate important differences in construction and methods of operation in order to secure satisfactory and economical results, but the rate of filtration is the essential point of difference between the two types.

In the slow sand filter the sand-bed is constructed in large water-tight reservoirs, either open or covered, each having usually an area of from $\frac{1}{2}$ to $1\frac{1}{2}$ acres. On the bottom of the reservoir is first laid a system of drains, then above this are placed successive layers of broken stone and gravel of decreasing size, and finally the bed of from 2 to 5 feet of sand which forms the true filter. The water flows by gravity, or is pumped,

* Trans. Am. Soc. C. E. 1905, LIV. D. p. 145.

upon the filter, passes through the underdrains to a collecting-well, and thence to the consumer. As the water filters through the sand, the friction causes some loss of head, which gradually increases as the filter becomes clogged with foreign matter. The rate of filtration is, however, maintained nearly uniform by suitable regulating devices which vary the head according to the resistance. When the working head has reached a certain fixed limit of a few feet, the water is shut off, the filter drained, and the surface cleaned by removing a thin layer of clogged sand. The operation is then resumed. Before the thickness of the sand layer becomes too greatly reduced, clean sand is added sufficient to restore the filter to its original depth. The chief features to consider in this form of filter are the proper construction of sand-bed and drains, the rate of filtration and its regulation, the loss of head, cleaning of beds, washing of sand, and the control of the operation by bacteriological tests.

The rapid filter differs from the slow filter in many of its details. It is built in much smaller units, and the drainage system and operating devices are widely different. Furthermore, in its operation it is dependent upon the use of a coagulant for efficient results. Further discussion of this type of filter is given in the next chapter.

THEORY AND EFFICIENCY OF FILTRATION.

487. **General Results of Filtration.** — In filtering water through a sand filter the main improvement to be noted is in the removal of the suspended matter. Even the color of a peaty water may be somewhat lessened, but that portion of the color due to matter in solution is not so readily removed by filtration. With respect to the elimination of bacteria and other micro-organisms, the results are so startling that it was a question for a long time how to explain them.

488. **Bacterial Results.** — When bacterial cultures are made from the raw water and from the effluent of a properly operated sand filter, a very great reduction in the number of bacteria is to be noted. This is well illustrated by the following data from examinations made on the Lawrence City filter, which uses the polluted Merrimack River water.

NO. OF BACTERIA PER C.C.

	1894.	1895.	1896.	1897.	1898.
Raw water	10,417	11,111	7,108	10,360	4,850
Filtered water	176	121	91	61	46
Efficiency of purification (per cent).	98.31	98.91	98.72	99.41	98.95

Typical bacterial results obtained with a water comparatively low in germ content are the following from the operations of the Washington filters for nine months from October, 1905 to June, 1906. The raw water is thoroughly settled in large reservoirs.*

RESULTS OF FILTRATION AT WASHINGTON, D. C.

Month.	Bacteria per c.c.		Month.	Bacteria per c.c.	
	Water from Reservoir.	Filtered Water.		Water from Reservoir.	Filtered Water.
1905.			1906.		
October. . . .	198	78	February . .	562	16
November. . .	153	27	March . . .	654	19
December . .	3750	60	April	399	22
1906.			May.	66	17
January . . .	1520	39	June	224	17

489. **Chemical Results.** — Usually the amount of organic matter of an unstable or objectionable character present in a raw water is not so large that the question of nitrification of organic matter, or the chemical purification of the water, is of great importance. Ordinary sand filtration does, however, effect a very considerable purification in this respect, especially in the case of a badly polluted water, such as the Merrimack water at Lawrence. The average results obtained at the Lawrence filters for six years were as follows (see also Art. 539).

	Raw Water.	Effluent.	Per cent Removed.
Color	0.43	0.38	11.6
Albuminoid ammonia	0.199	0.094	52.8
Oxygen consumed.	4.2	2.8	33

490. **Theory of Filtration.** — When working under favorable conditions, a sand filter will remove very nearly all of the bacteria originally present in the water. At first glance, it might be thought that this filtration process was merely a mechanical one, a straining out of the suspended particles by the sand layers.

491. *Inadequacy of Mechanical Explanations.* — There are various reasons why such an explanation is not wholly satisfactory, although undoubtedly the mechanical theory is effective in part. Particles too large to pass into the pores of the filters are of course removed by simple straining action. This action is, however, relatively unimportant.

* Trans. Am. Soc. C. E. 1907, LVI. p. 358.

The chief effect produced that may be considered mechanical in principle is, doubtless, the action of the sand bed as numerous minute sedimentation chambers, which, owing to their small size and the low velocity of flow, are quite efficient in the removal of the finer suspended particles including bacteria. In this way particles much smaller than the pore spaces in the sand are removed to a very considerable extent by purely mechanical means. If, however, the process was purely mechanical, the filtered water should be as good at one time as another, but such is not usually the case. When a sand filter is first installed, the filtrate is much richer in germ life than it is later. As it increases in age, it becomes more efficient, showing that some other factor than purely mechanical removal, functions in the process. The character of the applied water has also much to do with the quality of the effluent independent of its bacterial content. If the mechanical theory were correct, a variation in the fineness of the sand would in a measure affect the efficiency of the process, but, within the limits ordinarily employed, the difference in results due to variation in size of sand grain is very slight. The lack of relation between the number of bacteria in the affluent and effluent is also against a mechanical explanation.

492. *Inadequacy of Chemical Explanations.* — The chemical changes that are to be noted in filtration are of such a nature that it is hardly conceivable that a satisfactory explanation of the phenomena of filtration will rest upon a chemical basis. Generally there is some oxidation of the organic matter, as is shown by the reduction in "oxygen consumed." Most of the improvement, however, in the chemical condition of a water is occasioned not by purely chemical changes, but is due to the action of the living organisms present in the filter.

493. *Biological Explanation.* — As previously noted, a filter improves in efficiency as it grows older until it finally reaches a point where the flow of water through the same is so small as to necessitate cleaning, a process technically known as scraping. But even after cleaning, the results obtained are better with filters long established than with new ones. With the improvement in the bacterial content of the effluent, a marked change occurs in the character of the sand, particularly in the upper layers.

Naturally the suspended matter in the water, apparent turbidity as well as bacteria, is intercepted for the most part at the surface of the filter. Where there is an appreciable amount of these substances held in suspension in the water, a layer is quickly formed on the surface that quite changes the nature of the sand. Generally the coating is slimy and gelatinous, and to it has been ascribed the filtering power of a sand filter. This layer also forms, although more slowly, in waters that are relatively deficient in suspended matter, at least where particles are not sufficiently numerous to cause turbidity. When critically examined it is found to contain inorganic matter, as silt of all kinds, organic substances, as bacteria, algæ, diatoms, and amorphous material.

While this jelly-like deposit is forming at the surface there is also an appreciable action of a similar nature going on in the depth of the filter. In this case the formation of this substance is produced by living causes, organic instead of inorganic matter, therefore, predominating in its composition. This is due to the growth of the bacteria derived from the sand and water, the slimy matter being formed by the cells themselves and the exudation from the same. This organic matter accumulates more readily in summer than in winter, because of the more favorable growth conditions.

While a casual examination of the sand layers will show in a general way the distribution of the organic matter, a bacterial study demonstrates the presence of the organisms in the body of the filter, but they are accumulated in much larger numbers at or near the surface, as is evident from the following data gathered from the results of examinations of ten filters at Lawrence.*

TABLE NO. 68.

EXAMINATION OF TEN FILTERS AT LAWRENCE AS TO ORGANIC CONTENT AND BACTERIA AT DIFFERENT DEPTHS OF THE SAND LAYER.

Depth. Inches.	Organic Nitrogen. Parts per 100,000 by Weight.	Bacteria. Per Gram.
0–¼	20	6,600,000
1	9.5	1,940,000
3	6.4	720,000
6	4.7	300,000
12	4.0	90,000
24	2.3	47,000
36	1.6	35,000
48	1.2	29,000
60	1.2	26,000

494. Importance of Sediment Layer.—The accumulation at the surface of the filter has led to the view generally accepted by the German school that this surface sediment layer (Schmutzdecke of the Germans) is the chief agent in effective filtration. From the experiments conducted at various places in this country it is quite evident that too

* Report Mass. Bd. of Health, 1894, p. 635.

much emphasis has been put upon the filtering power of this layer,* as is shown from the following facts:

Waters so free from suspended matter as to show no turbidity form by bacterial growth, in a brief time, an organic slimy deposit in and on the sand, by the aid of which good effluents are produced. Waters containing much inorganic sediment may not develop enough organic slime to bind the mineral matter into a layer and so hold it on the surface. In such a case the inorganic solids are forced into the body of the filter, to the detriment of the efficiency of the same. Again, filters having a well-developed superficial sediment layer may have the continuity of the same broken if the surface of the filter-sand be exposed to the air. This peeling of the slimy surface-coat ought to disturb the efficiency of the filtrate if this layer was the sole effective agent in filtration, as has been generally claimed. But such is not the case as shown by the Lawrence tests.

Still again, the removal of the upper layer of the sand that has become clogged through deposition of suspended matter from the water ought to invariably impair the efficiency of the filtration until a new layer is produced. As a matter of fact such results do not necessarily follow, although it should be said that this is the most critical period in the condition of the filter. The Massachusetts experiments often show as good an effluent immediately after cleaning as before. Under the same auspices it has also been noted that filters often become more effective with age and long service. This has been shown particularly with medium coarse or coarse sands. The efficiency increased in one case (Filter 18*a*) as follows: †

Year.	Rate. Gallons.	Bacterial Efficiency. Per cent.
1893	2,000,000	96.75
1894	4,500,000	98.97
1895	4,500,000	99.57

Notwithstanding the increase in rate and the diminution in depth of sand from 5 to 3 feet (due to scraping) in two years, the character of the effluent steadily improved.

* Reinsch (*Cent. f. Bakt.*, 1894, XVI. p. 881) also emphasizes in the case of the Altona filters the importance of the thickness of the sand layer in comparison with the sediment layer.

† Mass. Bd. Health, 1895, p. 511.

These facts cannot well be explained on the theory that the effective agent in filtration is merely the surface layer. They are readily explainable, however, if one considers that the bacterial growth in the body of the filter exerts a strong effect on the filtration process. The value of the denser surface layer should not, however, be entirely neglected, but the relative merits of the organic slime in the inner layers of the filter should not be overlooked.

To recapitulate, the effective agent in filtration in sand filters is the organic slime in the filter-bed and the accumulated surface sediment layer, which is made up of both inorganic and organic constituents. A filter is therefore something more than a mere mechanical strainer, inasmuch as its efficiency rests largely upon biological causes. The sand itself acts as a mechanical support for these gelatinous films, holding them intact; for this reason a certain depth of sand is necessary to steady the action of the filter and prevent disturbance of this organic slime.

495. **Bacteria in the Effluent.**—Where a slow sand filter is doing satisfactory work, the number of bacteria found in the effluent is on the average small, either when expressed absolutely or compared with the number originally present in the unfiltered water. A good deal of variation, however, exists even in the same supply when a continuous study is made for considerable periods of time. The following data from the weekly results obtained in the Belmont filtration works of Philadelphia during August and September, 1903, illustrate this point: *

Filter No.	Bacteria per c. c. in filtered water.								
1	19	9	6	13	8	21	22	26	55
2	37	13	10	12	8	33	21	170	17
5	10	10	8	12	21	18		99	7
6	14	27	11	10	10	59	220	21	6

496. **Origin of Bacteria in Effluent.** — It is of considerable importance to determine the origin of the bacteria appearing in the effluent, especially as this figure is used in interpreting results of efficiency of filtration. When bacteriological methods were first introduced, it was assumed that the difference between the number in the water before and after filtration represented the number removed. This assumption is now known to be false. If a specific micro-organism, such as *B. prodigiosus*, which does not possess the quality of developing in the body of the filter and under-drains, is applied in sterile water to the filter,

* *Jour. Frank. Inst.*, 1904, CLVII. p. 193.

it is possible to determine with much greater accuracy the exact origin of the respective bacteria in the effluent. More recently* the colon organism, *B. coli communis*, has been utilized in this work. The value of this organism lies in the fact that it is generally a regular accompaniment of polluted water, and therefore if it should appear in the effluent its presence is indicative of danger to some extent.

The relative proportion which actually pass through a filter and those which develop in the under-drains will vary at different seasons of the year and under different conditions as to filtration. During the colder months, when the water is low, the number developing in the filter will be at a minimum. Increased rate of filtration will diminish the number per c.c. in the drains by more rapid flushing, while the higher velocity will tend to force a slightly larger number through the filter. Of the two classes of bacteria appearing in the effluent, those that develop in the drains and body of the filter are the least important. They are generally the distinctive water organisms that naturally grow in such a habitat.

If the sand of a filter is sterilized by steam or by the addition of a chemical disinfectant, and the water allowed to filter through the same, it has been observed that the effluent often contains more bacteria than the unfiltered water. This is due to the rapid development of bacteria in the sterilized sand, there being enough organisms derived from the percolating water to seed the filter. In the "cooked" filter the conditions seem to favor very rapid growth. The high number in the effluent then in a case like this is not due to filtration through the filter as was formerly supposed.

497. Efficiency of Filtration.—Since the introduction of bacteriological methods it has been customary to consider the ratio of the difference between the number of bacteria in the raw water and in the effluent, to the number of bacteria in the raw water, as an index of the efficiency of operation, and this number is frequently referred to as the *bacterial efficiency*.† Inasmuch as the bacteria in the effluent includes those organisms of post-filtration origin as well as those that have found their way through the filter, this bacterial efficiency evidently does not represent accurately the number of bacteria removed from the water. To determine this factor, which has been called the *bacterial purification*, it is necessary to add some special form to the applied water, as *B. prodigiosus* or *B. coli communis*, and then determine its frequency in the filtered water. The *hygienic efficiency* is the percentage removed

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* Clark & Gage. *Science*, Mch. 23, 1900.

† Mass. Bd. of Health, 1894, p. 592.

by filtration of applied bacteria capable of producing disease. It does not necessarily follow because one organism is able to pass through a filter that all others will, so the hygienic efficiency and the bacterial purification may not correspond closely. These relations are brought out strikingly in the tests made on the Lawrence City filter,* which were as follows:

	Number of Cases of Typhoid in City.	Bacterial Efficiency. Per cent.	Percentage of Cases in which *B. coli* was found in 1 c.c. of Water.
December, 1898	12	92.2	72
January, 1899	59	98.31	54
February, 1899	12	98.17	62
March, 1899	9	99.89	8

Under ordinary working conditions, the germ content of the effluent should be reduced to the lowest possible terms. The German standard calls for an effluent with not to exceed 100 bacteria per c.c. when cultures are grown in gelatin for two days, but as a matter of fact this number is frequently exceeded even in good working filters, although the average number is usually below this limit. Generally speaking, the efficiency when expressed in percentage of number present in raw water ranges from 98 to 99 per cent, or above. In the case of filters using quiescent waters as sources of supply, where the number of bacteria in the applied water is low, the percentage of bacterial efficiency is of course reduced. Where the source of supply is from running streams, the bacterial content of the raw water is much higher, and consequently the percentage removed is much greater. The efficiency of filtration is also much affected by variation in working conditions, as by a fluctuation in rate or by scraping the surface of filter. Formation of ice on uncovered filters is also detrimental.

498. **Passage of Bacteria Confirmed by Disease Outbreaks.**—The preceding experimentally controlled work can also be substantiated by observations made on the practical working of filters in relation to disease production. Between the years 1886 and 1893 several outbreaks of typhoid fever occurred in Altona, and in 1891 a marked epidemic in Berlin. The distribution of disease in these two cases was such that it was evident that the same had been disseminated by the water-supply. In Altona these outbreaks invariably followed similar epidemics in Hamburg. In Berlin the case was strikingly emphasized,

* Clark & Gage. *Science*, Mch. 23, 1900.

because the outbreak was confined to the region supplied by the Stralau (open) filters. At that time no search was made for the typhoid germ, but later, in 1894, Lösener * found the typhoid organism in the tap-water in this district, a discovery that was confirmed by Elsner.† During the cholera scourge in Germany in 1892–3 the cholera organism appeared in the filtered water of at least four cities. ‡ It should, however, be noted that in these cases the filters were not in proper working conditions, on account of the presence of ice. Still again, at Rotterdam in 1904, typhoid fever was transmitted through the agency of imperfectly filtered water due to the effect of winter weather and relatively coarse filters.§

Evidence such as the above indicates that it is possible for even disease bacteria to find their way through filters that are under unfavorable working conditions. Under normal operating conditions, the passage of disease-producing bacteria is very rare.

499. Death-rates as Measures of Efficiency. — While the common method of expressing the efficiency of any filter is to measure it by the bacteria appearing in the effluent, either expressed absolutely or in terms of percentage apparently removed, still, after all, the effect on the death-rate or the case-rate of water-borne diseases is the crucial test of efficiency. Where statistics are comparable, they invariably show a diminution in death-rate that is sometimes so marked as to be astonishing. The following table from Hazen || exhibits clearly the effect of filtration upon the typhoid death-rate.

TABLE SHOWING EFFECT OF THE ADOPTION OF FILTRATION UPON THE TYPHOID DEATH-RATE.

Place.	Date of Change.	Typhoid Death Rate per 100,000.		
		Five Years before Change.	Five Years after Change.	Percentage of Reduction.
Zurich, Switzerland	1885	76	10	87
Hamburg, Germany	1892–93	47	7	85
Lawrence, Mass.	1893	121	26	79
Albany, N. Y.	1899	104	28*	73

* Four years.

Since the introduction of sand filtration into Lawrence, Mass., a city that formerly used the polluted Merrimack River water, the

* Arbeit. a. d. k. Gesundheitsamte, XI. p. 240.

† *Zeit. f. Hyg.*, XXI. p. 30.

‡ Fränkel, C. *Hyg. Rund.*, 1896, VI. p. 3.

§ Trans. Am. Soc. C. E., 1905, LIV. D., p. 164.

|| Trans. Am. Soc. C. E., 1905, LIV. D., p. 151.

typhoid rates have been reduced nearly 80 per cent. In Hamburg the death-rate from typhoid was diminished by the installation of the filter over 70 per cent. The most striking instance on record is the classic example of the protection afforded to the city of Altona in the summer of 1892, while its sister city, Hamburg, was striken with cholera (216). Numerous European cities that use this system are thus able to utilize surface-waters of doubtful purity, and by treating them in this way to insure a safe supply.

CONSTRUCTION AND OPERATION.

500. Rate of Filtration.—In the design of a filter-plant the first question to be settled is the rate of filtration which shall be adopted. The higher the rate the less the area required and hence the less will be the first cost; but the cost of operation is not greatly affected by the rate, so that the economy of high rates is not as great as it might appear at first sight.

Rates of filtration are in this country usually stated in terms of gallons per acre per day or per hour, and on the Continent in meters depth of water per day or per hour.*

501. *Rates Used in European Practice.*—The experience of European works has resulted in the adoption of a rate, for most places, of between 2 and 3 million gallons per acre per day. This is materially less than the rate of 3.9 millions mentioned by Kirkwood as being the average in 1866, and denotes a marked change in practice since that time.

The Hamburg works, completed in 1893, were designed on a basis of 1.6 millions. At Berlin the standard rate is about 2.6 millions. At London the average rate of all filters is about 1.8 million gallons per acre per day, but some companies use as high as 2.5 millions. Rates considerably higher than these are used in a few places, notably at Zurich, where a rate of nearly 8 millions is used with satisfactory results, but here the unfiltered water is very clear and contains but a few hundred bacteria per c.c.

As a general statement 100 mm. per hour (equivalent to 2.57 million gallons per acre per day) is considered by German authorities as a standard maximum rate. English engineers favor a slightly greater rate of 8 to 12 feet per day, or 2.6 to 3.9 million gallons per acre.

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* One meter per day is equal to 1.07 million gallons per acre per day; one foot per day equals 0.326 million gallons per acre per day.

502. *Effect of Rate on Efficiency.*—The long experience of European works, resulting in the adoption of the rates given above, is very strong evidence that higher rates are undesirable. The decreased efficiency with increased rate has been directly shown in important experimental studies by Piefke* at Berlin. From these trials he recommended as low as 1.28 million gallons per acre daily as a safe maximum rate, but more recently he has used as high as 2.57 million gallons.

It is undoubtedly true that high rates of filtration will give less efficiency than low rates when the difference is very great, but much evidence is available which indicates that, under certain conditions, no perceptible decrease in efficiency will result from a considerable increase in rate beyond the standard rates mentioned above. Experiments by Kümmel at Altona on Elbe water, with rates of 4, 8, and 16 feet per day (1.3, 2.6, and 5.2 million gallons per acre), showed equally good results.† At Zurich, rates of from 4.4 to 21.5 million gallons per acre per day likewise gave equally good results. The latter experiments are not, however, especially significant for normal conditions on account of the extremely clear water and very low germ content.‡

The most important experiments in this direction are those which have been carried out by the Massachusetts Board of Health at Lawrence, Mass., on the Merrimack River water. This water is not often very turbid, but is badly polluted by sewage. It contains on the average about 0.2 parts per million of albuminoid ammonia. In 1892 the results obtained on filters, the majority of which had been in operation but six or eight months, indicated that the efficiency § decreased slightly with increase in rate, even for rates as low as 0.5 to 3 million gallons per acre daily. In 1893, with older filters, the influence of rate was not so apparent, but high rates were not as yet used. In 1894, rates of 5 to 10 million gallons and over were used with several of the filters, with satisfactory results "from those filters which had been in operation for a considerable period." The bacterial efficiency in these cases was fully 99 per cent. With such high rates, however, the effect of scraping was more marked than with low rates. Regarding rates in practice, the following statement is made: ‖ "Experience during the past ten years with ten different filters which have been in operation at rates of 5 million gallons or more per acre daily leads us to the con-

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* *Jour. f. Gasbel. u. Wasservers.*, 1891, pp. 208, 228.

† Trans. Am. Soc. C. E., 1893, XXX. p. 333.

‡ Proc. Inst. C. E., CXL. p. 280.

§ Determined by the percentage of *B. prodigiosus* that passed the filters.

‖ Mass. Bd. Health, 1894, p. 606.

clusion that, with conditions substantially equivalent to those at Lawrence, the above-mentioned rate may be safely adopted in practice and yield an effluent of satisfactory quality after the first or second month of operation." These conclusions are further substantiated in the report for 1895, entirely satisfactory results having been obtained with rates of 5 to 7 million gallons.

503. *Rate to be Adopted.* — In view of the results obtained above and the statements of the Board based on many years of experimentation, it would appear that rates somewhat higher than those used abroad could be adopted with safety. In the Albany plant Mr. Hazen assumed a rate of 3 million gallons, and in most of the important plants constructed since, a rate of about 3 million has been adopted as the standard. Considerably higher rates have been used in some cases for the filtration of relatively clear waters. Where a preliminary treatment is employed, of greater efficiency than ordinary sedimentation, such as the use of a coagulant with sedimentation, rapid mechanical filters, or "scrubbers," the rate of filtration may be materially increased, a rate of 6 to 8 million gallons being then quite practicable.* (See Art. 534.)

A conservative course should unquestionably be followed in using higher rates than those established by past experience, and probably 3 or 4 million gallons is as high as it would in general be advisable to go in the design of a new plant. If subsequent operation shows that a higher rate can be adopted with efficiency and economy, the fact can be taken advantage of as the demand for water increases. Local conditions are apt to vary widely so that any general rule must be applied with caution. Each case demands independent consideration in order that the best and most economical solution may be arrived at.

504. *Uniform Rate Desirable.* — Sudden changes of rate are apt to produce disturbances in the filter and to give a reduced efficiency. The Lawrence experiments on this point show that for a moderate increase in rate of 10 or 20 per cent the effect is inappreciable, but that a large reduction in efficiency is caused by an increase in rate of 50 per cent. Marked reductions in rate followed by a return to the normal had little effect. It was also found that filters most sensitive to such changes were those of shallow depth, those of coarse sand, and those that had been but a short time in operation. In practice, absolute uniformity of operation is unnecessary, but sudden changes in rate should be avoided, and especially any large increase above the normal.

* For additional data see references 42 and 43 at end of chapter.

505. Capacity.—The standard rate having been determined, the required net working capacity will be equal to the maximum rate of delivery divided by the assumed rate of filtration. To economize area and to avoid rapid changes in rate, a clear-water reservoir should be provided. The best size for this will depend on local conditions, but it will usually be desirable to have it of sufficient capacity to equalize the demand throughout the day. It will then be necessary to vary the rate of filtration only to accord with the daily variations in consumption (see Art. 527). In Chapter II it was shown that the maximum daily rate of consumption is likely to be about 175 per cent of the average, and with a clear-water reservoir of the capacity mentioned above the filters must be designed to deliver at this maximum daily rate. If the reservoir has a less capacity, then the maximum rate of delivery of the filters will be correspondingly increased. Occasional high rates of consumption and extraordinary demands may be provided for by the use of a rate of filtration somewhat higher than the standard adopted.

In addition to the area as above found, a reserve area for cleaning must be provided. For small works this will be one bed; for works containing several beds it will be necessary to allow one bed for each 5 to 10 beds, depending on the frequency of scraping and the time required for putting a filter into operation after cleaning.

506. Number and Size of Beds.—The proper size of beds is chiefly a question of economical construction. The larger the beds the less the cost per acre, but the greater will be the area out of service in the one or more reserve beds. Ordinarily the size for a considerable number of beds is from 1 to 1.5 acres for open beds, and from .4 to .8 acres for covered beds. For small total areas of .5 to 1 acre three beds would ordinarily be used, and for still smaller areas two beds. The economical number can in any case be determined by comparative estimates, but some assistance may be had by the following mathematical analysis.

The cost of a filter may roughly be estimated as made up of two items: (1) a portion proportional to the area, which would include cost of bottom, filling, small drains, cover, and the end walls, we will say (basins assumed rectangular and placed side by side); and (2) a portion nearly independent of the size, such as cost of piping, valves, valve-chamber, division walls, etc. Let c = amount of the first portion per acre, and C = the latter portion per filter. If q = area of one filter, n = number of filters, and A = total net area required, then if one filter is to be held in reserve,

$$n = \frac{A}{q} + 1. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

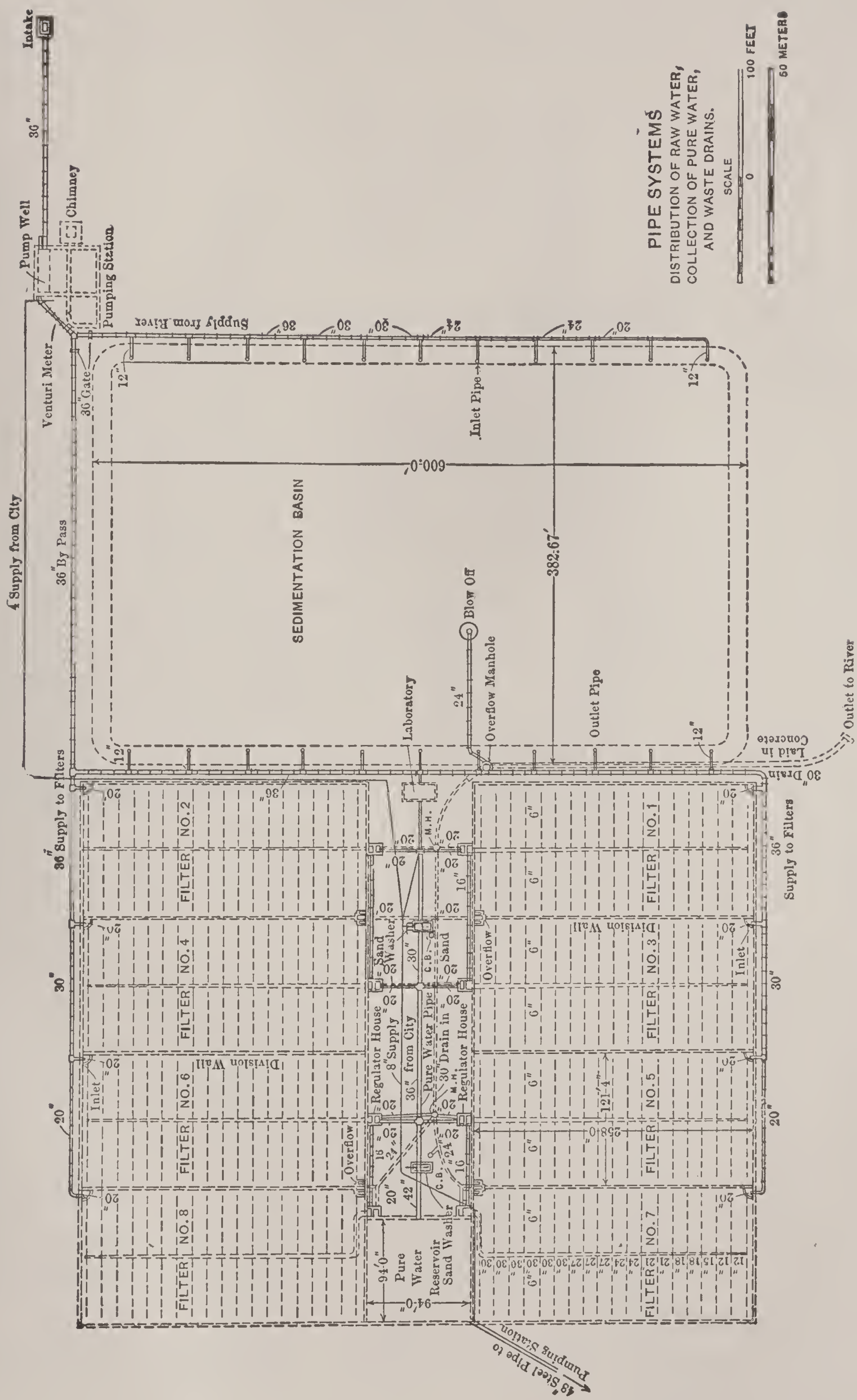


FIG. 120.—ALBANY FILTER-BEDS AND SEDIMENTATION-BASIN.
(From Trans. Am. Soc. C. E., vol. XLIII.)

The total cost is

$$K = Cn + cnq \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

$$= C\left(\frac{A}{q} + 1\right) + cq\left(\frac{A}{q} + 1\right),$$

$$= \frac{CA}{q} + C + cA + cq, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

We then have $\quad \dfrac{dK}{dq} = -\dfrac{CA}{q^2} + C,$

whence for a minimum cost

$$q = \sqrt{\frac{C}{c}A}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

that is, the economical area is proportional to $\sqrt{A}$ and to $\sqrt{\dfrac{C}{c}}$. The larger the value of c the smaller is q, and hence for covered beds q will be smaller than for open beds. The values of $\dfrac{C}{c}$ will hardly be larger than $\frac{1}{9}$ or less than $\frac{1}{16}$, giving a value of $q = \frac{1}{4}\sqrt{A}$ to $\frac{1}{3}\sqrt{A}$. Thus when $A = 1$ acre, the capacity would be $\frac{1}{4}$ to $\frac{1}{3}$ acre, giving 3 or 4 beds; for $A = 4$ acres, $q = \frac{1}{2}$ to $\frac{2}{3}$, giving 6 to 8 beds; and for $A = 9$ acres, $q = \frac{3}{4}$ to 1 acre, giving 9 to 12 beds, etc. When the number becomes so large as to require two beds to be held in reserve the size will no longer increase with the area. Sizes considerably larger than 1 acre have been used, such as 1.9 acres at Hamburg, but they will hardly be economical. Such large beds are also undesirable on account of the increased difficulty of securing uniform operation.

507. **General Construction.** — Filter-beds are usually rectangular in form and arranged side by side in one or two rows according to the number. The shape of the area available often determines this point, but otherwise a convenient arrangement is to place them in two rows with a space between for sand-washing, etc., and to have valve-chambers facing this central passage-way, as illustrated by the Albany plant (Fig. 120). A single row would be more economical of masonry, but would require more piping.

A large number of basins may be divided into groups and arranged in the above manner.

The economical proportions for rectangular beds arranged side by side is approximately given by the formula derived in Art. 479 for settling-basins. It is $\frac{b}{a} = \frac{n + 1}{2n}$, where b = width, a = length, and n = number of beds in a row. This assumes the cost per lineal foot of

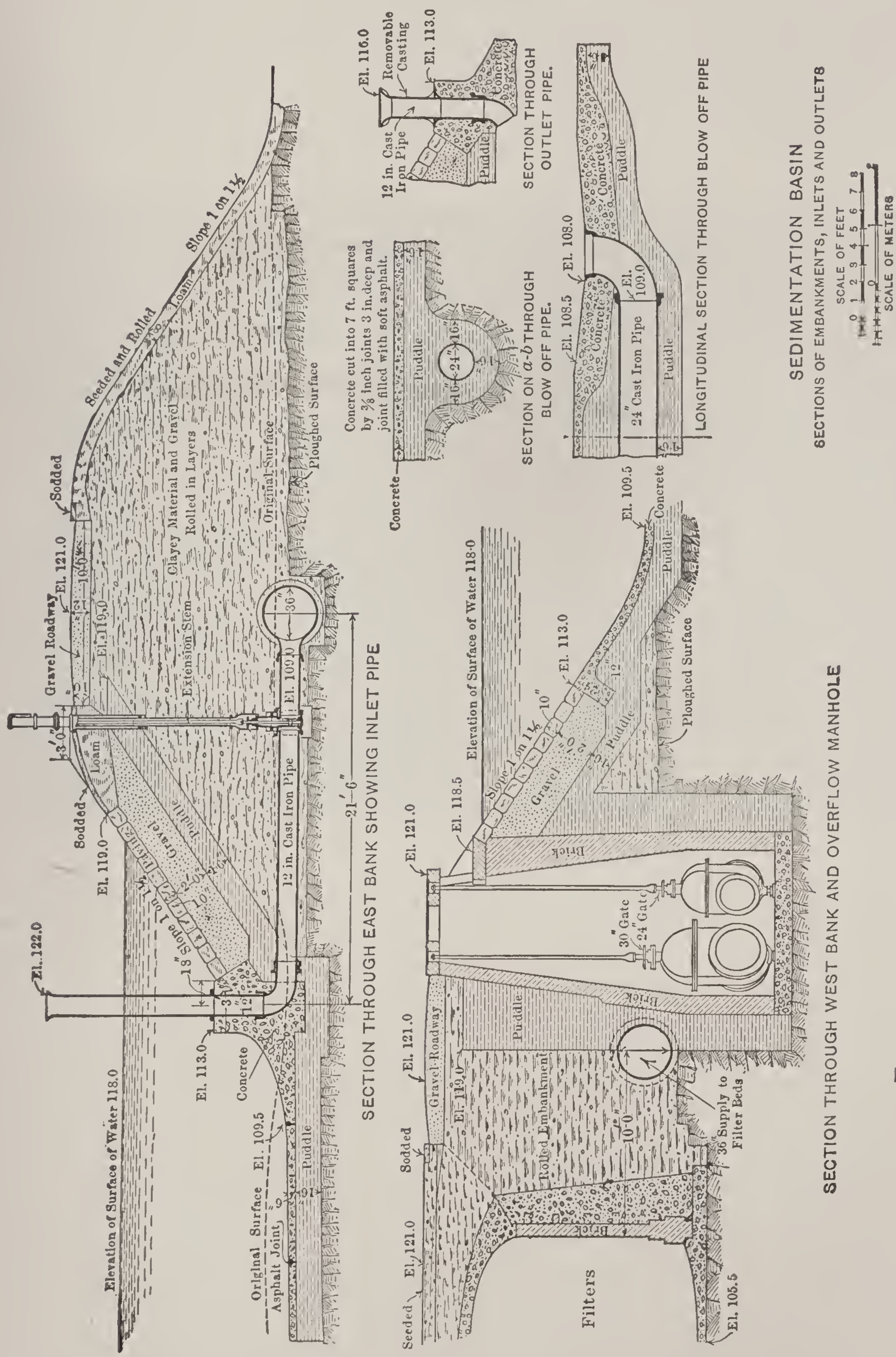


FIG. 121. — ALBANY FILTER-PLANT; DETAILS OF SEDIMENTATION-BASIN. (From Trans. Am. Soc. C. E., vol. XLIII.)

interior and exterior walls equal, which is approximately true. A larger cost of interior walls will tend to increase b and *vice versa*.

The cost of the large central drain running lengthwise of the bed will also tend to increase *b*, while the expense of exterior piping will tend to reduce it.

In general construction a filter-basin is built in a way similar to small distributing-reservoirs. (See Chapter XXVII.) Earth embankments for the sides are cheaper than masonry walls, but require more ground. If the filters are covered, masonry walls are usually employed. Particular care must be taken to render the basin water-tight, both on

FIG. 122. — INTERIOR VIEW; WASHINGTON FILTERS.
(From Trans. Am. Soc. C. E., vol. LVII.)

the bottom and at the sides. Cracks in division walls are likely to admit unfiltered water to the under-drains and should be especially guarded against.

Concrete, well reinforced, is a very satisfactory material for filter construction, especially with respect to the walls. If any cracks occur they are likely to be very minute. In the construction of water-tight bottoms good results have been secured by placing concrete in sections in two layers, so arranged that the sections in the different layers will thoroughly break joints. Covers for filters are constructed in the same general manner as described in Chapter XXVII. Reinforced or plain concrete vaulting is usually employed, although wood has been used; but the latter does not afford as good a protection from freezing or from summer heat. Admission for workmen is provided by a gangway leading from an opening at a point where the vaulting is raised; or the entire cover may be made of the necessary height to give ready access at any point. This method of construction offers opportunity for lighting and ventilation by means of windows in the outside walls. Walls and piers should be built with small offsets near the bottom in order to insure good filtration at that point. The covered filter at Washington is illustrated in Fig. 122. Groined arches of concrete were there used. Fig. 123 shows a section through the gangway of a filter and also the

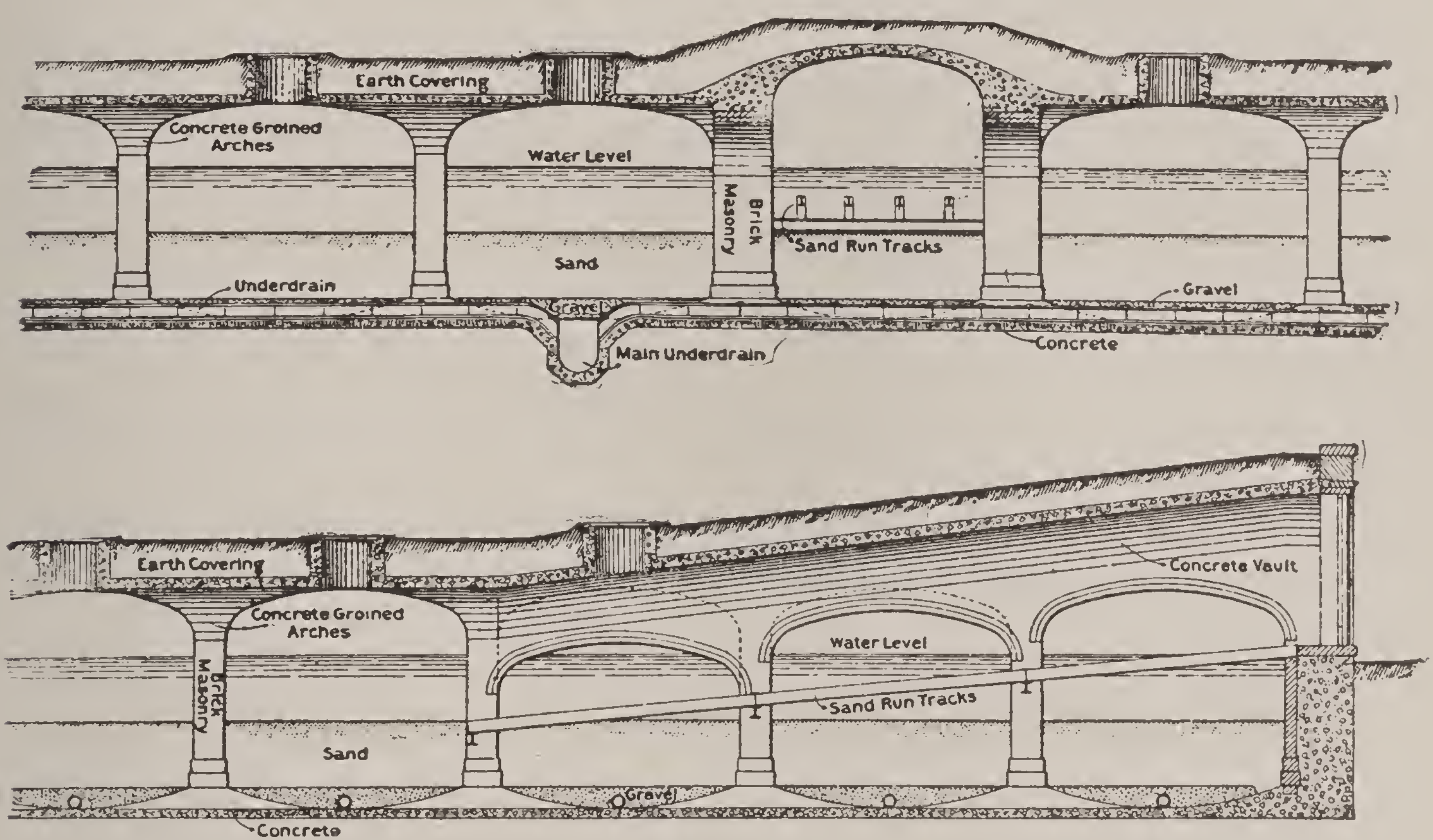


FIG. 123. — GENERAL CONSTRUCTION OF A COVERED FILTER.
(From Report on the Water-supply of Philadelphia, 1899.)

general method of construction. In some of the most recent plants sand-run tracks are dispensed with, the sand being moved through pipes.

508. Necessity for Covering Filters. — Since the cost of covers amounts to about one-third of the total cost of filters, the question of open *versus* closed filters is a very important one. The principal reason for covering filters is to avoid the difficulties connected with the operation of open filters in winter. To clean filters when covered with ice is a troublesome and expensive operation, while if the filters are drained for cleaning, trouble arises from the freezing of the sand. Winter operation is thus likely to show a decreased effective area and a lowered efficiency.

At Berlin all beds are now covered. At Hamburg open filters are used. Ice forms there, however, to a thickness of 10 or 12 inches and causes considerable trouble in cleaning. In England filters are not covered and little trouble is experienced, but the winters are quite mild, the mean January temperature at London being about 38°. At Poughkeepsie, N. Y., and at Lawrence, Mass., where original filters were built open, it was found that a large expense was involved in the removal of ice. The Poughkeepsie filters have since been covered, and covers have been used in additional new filters at Lawrence. In general the increased convenience and regularity resulting from the use of covers tends to encourage their use even when not necessary for good efficiency.

Mr. Hazen * has proposed as a general rule that covers should be used where the average January temperature is below 32°. This includes the area north of a line passing through St. Louis, Cincinnati, Pittsburg, and Philadelphia. In the large plants at Pittsburg, Washington and Philadelphia, constructed since 1900, it has, however, been found desirable to use covers. On the other hand, open filters have been built at Providence and Denver. Whether covers should be used depends upon the extent to which ice will form, the frequency of the occurrence of thaws which will enable a filter to be properly cleaned, and the length of time between cleanings as determined by the character of the water.

Another very considerable advantage of covered filters in some places is in the prevention of the growth of algæ, and thereby reducing the frequency of cleaning. At Zurich both open and closed filters were for a time in use. The number of days between scrapings was on the average, for 1891 and 1892, as follows: †

	1891.	1892.
Covered filters	37	36
Open filters	28	23

At Poughkeepsie much trouble was also experienced from the growth of algæ in the open filters there used.

509. *Effect of Cold Weather on Efficiency of Filtration.* — The reduced efficiency of open filters in winter is shown by the results of bacteriological analyses, and is further substantiated by a considerable number of disease epidemics that have broken out in the winter in cities supplied with filtered water. Freezing weather is especially apt to have a detrimental effect in connection with the cleaning of the filter. (See Art. 531.)

In 1889 the effluent of the Stralau (uncovered) filters in Berlin contained on the average less than 100 bacteria per c.c., but in March

* Filtration of Public Water-supplies, p. 12.

† Report Zurich Water-works, 1892, p. 27.

that year, at a time when the filter operations were interfered with through the action of cold weather, the number rose to 3 or 4 thousand. Coincident with this change occurred a typhoid epidemic, and also one of dysentery, that were limited to the Stralau district, while that portion of the city supplied from the Tegel filters remained free from both diseases. The history of the open filters at Altona is also similar. Outbreaks of typhoid have occurred explosively at Altona in the winters of 1886, 1887, 1888, 1891, and 1892, and Reincke has traced these to the imperfect operation of the filters during cold weather. It is significant that in almost every case these outbreaks were preceded by similar epidemics in Hamburg, and furthermore that they only occurred in Altona during the winter, when the action of the filters was impaired by frost. In the typhoid outbreak that occurred in the early part of 1891, Wallichs * had noted a sudden increase from a normal of less than 100 bacteria per c.c. to 2615. The small winter outbreak of cholera that occurred in Altona in 1893 Koch was able to trace to the imperfect operation of a single filter.

510. **The Filtering Sand.** — In selecting a sand for filtering purposes the important properties are its size and uniformity of grain, the presence or absence of fine material and organic matter, and its chemical composition.

511. *Mechanical Analysis.* — The particles of any given sand vary much in size, but as regards the size of the interstices and the percolation of water, it is obvious that the size of the finer particles rather than the coarser determines its effective size. In Art. 85 the term "effective size" as used in sand analysis was defined, as also the measure of uniformity known as the "uniformity coefficient."

Methods of analysis of size are fully described by Mr. Hazen in the Massachusetts Report for 1892, page 541,† and in his work on "Filtration of Water-supplies." Gravel is separated by hand-picking into several sizes, and the average size of each is determined by weighing. Sand is separated by sets of sieves with meshes ranging from 2 to 200 per inch. The proportion of sand or gravel finer than each particular size is then plotted and the effective size, or the size corresponding to the 10 per cent proportion, is readily found. Care must be taken to have the sand thoroughly dry before sifting.

The separation size of any particular sieve is found by Mr. Hazen by determining the average diameter of the very last particles to pass the sieve. To compute this, the weight and specific gravity of a known

* Deutsche med. Wochenschrift, 1891, No. 25.

† See also *Eng. Record*, 1897, XXXV. p. 163.

number of such particles is determined and the grains calculated as spheres. The actual size of mesh is irregular, and the number of meshes per inch is not to be relied upon as a measure of size.

For particles finer than 0.1 mm. (corresponding to a sieve with about 200 meshes per inch) the method of elutriation is used. In this process, 3 grams of sand are placed in a beaker 90 mm. high and holding about 230 c.c., and the beaker is then filled with distilled water at 20° C. (68° F.) The water and sand are thoroughly mixed and allowed to stand 15 seconds, and the water is then decanted. This is repeated twice and the sand is then weighed. Experiments show that this sand can be considered as greater than 0.08 mm. in size. The decanted sand is then treated in a similar way, with one minute for settling, and the sand which settles calculated as greater than 0.04 mm. sand. The amount of the portion below 0.04 mm. is estimated by difference.

512. *Selection of Sand.*—Experiments show that very fine sand is considerably more efficient in removing bacteria than ordinary or coarse sand, but within the ordinary limits of size (0.2 to 0.4 mm.) the Lawrence experiments indicate but little difference in efficiency. The finer sands, however, cause a steadier action and prevent disturbances due to scraping; they also cause a greater loss of head in the filter, and so make the action more uniform over the filter area. On the other hand, fine sand becomes clogged sooner than coarse and involves therefore more expense in cleaning. For waters containing very fine sediment, coarse filters are likely to become clogged to a considerable depth, requiring the removal of too thick a surface layer.

In practice the size of sand used varies from about 0.2 mm. for some of the Holland dune sands to about 0.4 mm., averaging about 0.35.* It is desirable that a sand be fairly uniform in grain. If the particles vary greatly in size, it will be difficult to wash, and in fact will have much of the finer particles removed in the process, thus increasing the effective size. It is especially important that the sand should be of the same grade in all parts of the same filter in order that the frictional resistance, and therefore the rate of filtration, shall be uniform. Frequent analyses should be made as the sand is delivered at the works.

Regarding other requirements, the sand should be free from clay, and if necessary it should be washed. The chemical composition is also important, as a sand containing a considerable amount of lime will increase the hardness of the water. It has also been found that the presence of aluminous and calcareous material increases very materially the resistance to the flow of water.†

* See analyses of sand from many filters in Hazen's "Filtration of Water-supplies."

† Mass. Report, 1894, p. 757.

The specifications for the sand for the Albany filter-plant, Allen Hazen, Mem. Am. Soc. C. E., engineer, were as follows:

"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 per cent by weight less than 0.13 mm., nor more than 10 per cent less than 0.27 mm.; at least 10 per cent by weight shall be less than 0.36 mm., and at least 70 per cent by weight shall be less than 1 mm., and no particles shall be more than 5 mm. in diameter. The diameters of the sand grains will be computed as the diameters of spheres of equal volume. The sand shall not contain more than 2 per cent, by weight, of lime and magnesia taken together and calculated as carbonates."

Where it is necessary to wash the sand a standard for this work must be adopted. At Washington a turbidity standard was required equivalent to about 0.2 per cent of clay. At Pittsburg the specifications required that 100 grains of sand shaken in one liter of water should not cause a turbidity greater than 200 parts per million, silica standard.

513. Friction in the Sand Layer. — From Art. 85 the rate of filtration through sand is

$$V = cd^2\frac{h}{l}\left(\frac{t + 10}{60}\right), \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where V = velocity of water in meters daily in a solid column;
c = a coefficient, equal to 400 to 1000;
d = effective size of sand;
h = head causing flow;
l = depth of sand layer; and
t = temperature in degrees Fahrenheit.

Using a value of c of 800 the following table of losses of head, or values of h, has been calculated for a filter 1 foot thick

TABLE NO. 69.

FRICTIONAL HEAD IN FEET, IN COMPACTED SAND ONE FOOT THICK, AT A TEMPERATURE OF 50° F.

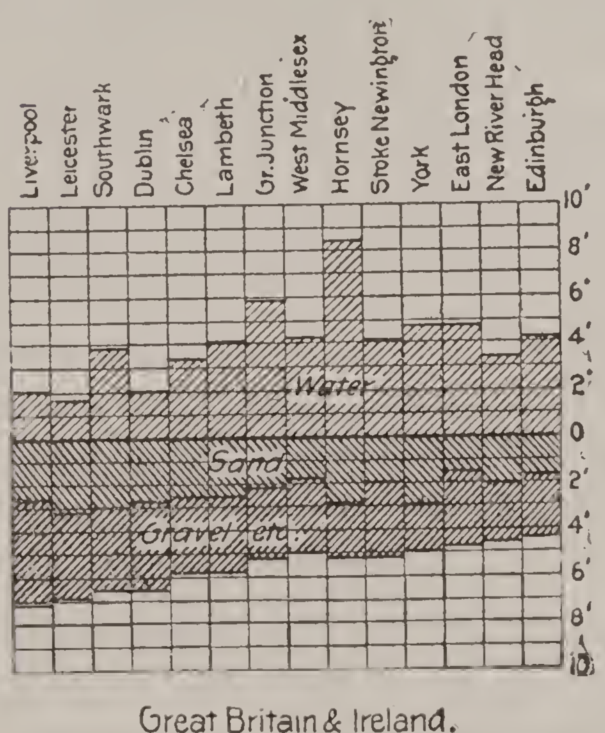
Size of Sand Millimeters.	Rate of Filtration, Millions of Gallons per Acre per Day.								
	1	1½	2	2½	3	3½	4	4½	5
.15	.052	.078	.104	.130	.156	.182	.208	.234	.260
.20	.030	.045	.060	.075	.090	.105	.120	.135	.150
.25	.019	.028	.038	.047	.056	.065	.075	.084	.094
.30	.013	.020	.026	.033	.039	.046	.052	.058	.065
.35	.010	.014	.019	.024	.029	.034	.034	.043	.048
.40	.007	.011	.014	.018	.022	.026	.029	.033	.037

The effect of temperature on the resistance is very marked, the loss of head at 40° being 20 per cent higher, at 60° about 14 per cent lower, and at 70° 25 per cent lower than the above figures.

The loss of head in a freshly cleaned filter composed of a 0.30-mm. sand, 4 feet deep, and filtering at a rate of 3 million gallons per acre per day, will be, according to this table, approximately. 039×4=.156 feet, or about 2 inches. In the winter it will be more and in the summer less. After a filter has been in use for some time after cleaning, the effect of clogging is of course to cause a loss of head many times greater than these figures.

514. Thickness of Sand Bed.—In the older filters great variations exist in the thickness of the several layers of sand and gravel, and in the depth of water on the filter. Fig. 124 shows the make-up of many filters abroad and illustrates this lack of uniformity. It will be seen that the thickness of sand is usually from 2 to 3 feet, the gravel layer about the same, and the depth of water about 3 or 4 feet. In some filters a layer of fine sand is underlain by a thick layer of coarse.

In designing a filter it should be noted that the sand forms the filtering medium; the gravel serves simply to collect the filtered water with little resistance to flow. There is no object in having the main body of sand of different sizes unless it happens that



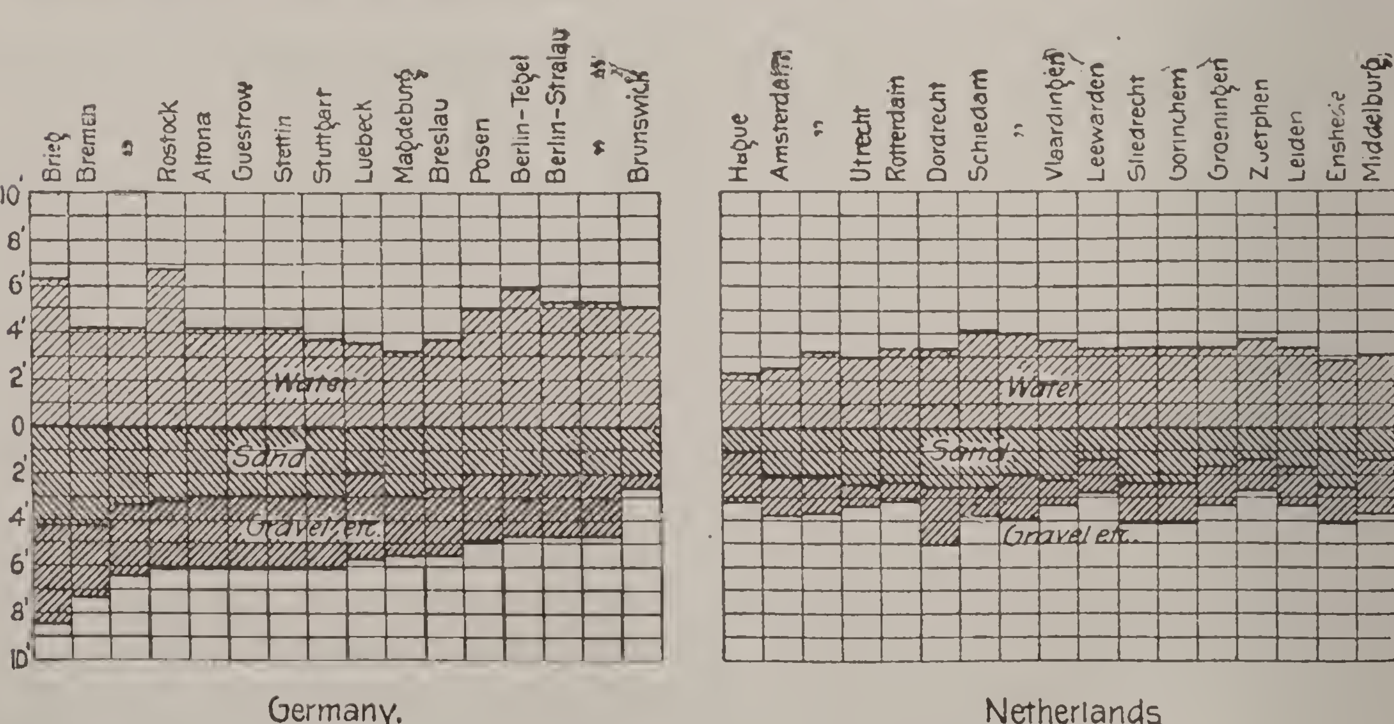


FIG. 124.—MAKE-UP OF FOREIGN FILTERS.
(From *Engineering News*, vol. XXVIII.)

a sand of the fineness desired for the upper portion of the bed is expensive, in which case a coarser sand may be used for a considerable thickness next to the gravel. A fine sand should never be placed below a coarser one, as this will cause subsurface clogging.

The original depth of sand must be sufficient to form an effective filter and, besides, to allow of several scrapings without the renewal of the sand. Inasmuch as the bacterial efficiency depends in part on the action which takes place in the body of the filter (Art. 494), and not exclusively at the surface, an increase in depth within certain limits will tend to increase the efficiency of the filter. The Imperial German Board of Health requires as a minimum at least 12 inches, but in actual practice the beds are considerably thicker. The effect of deep beds is similar to that of fine sand in steadying the action of a filter, and it has been clearly shown by the Lawrence experiments that the operation of beds 4 to 5 feet thick is not so much affected as that of beds 1 to 2 feet thick by such disturbances as variations in rate, scraping of beds, etc., although the results with perfectly uniform conditions are not materially different. The effect of depth is also very important in causing a more uniform action over the entire bed of a freshly cleaned filter by minimizing the effect of frictional resistance in the under-drains.

For the foregoing reasons it would seem desirable to adopt a minimum thickness of at least 2 feet, and to make the bed originally 3 feet thick. In several filters recently constructed the original depth of sand is 4 feet.

515. **The Depth of Water on the Filter** should be sufficient to enable the desired maximum head to be used without reducing the pressure in the filter below atmospheric; and as the resistance is nearly all at the surface of the sand, the depth must be about equal to the maximum head used. (Art. 520.) Certain experiments have shown that "negative heads" are likely to cause the liberation in the filter of some of the air dissolved in the water and so cause disturbances. The depth must also be greater than the thickest ice likely to form. Beyond these limiting depths any increase serves only to increase the expense of construction.

516. **Drainage Systems.** — To collect the filtered water a system of under-drains is necessary. The important points to be considered in its design are durability and freedom from derangement, and that the loss of head therein shall be small. The system of drains usually consists of a large central drain running the length of the filter, and branch drains at right angles thereto placed at regular intervals, usually of 8 to 12 feet. The central drain may be made either of large vitrified

pipe or of concrete; the branch drains are usually of 4- to 8-inch round or special tile, laid with open joints.

To avoid using a very large amount of gravel filling in order to form a level surface for the sand bed, the main drain should be sunk into the floor of the filter so that its top is no higher than the laterals. For the same reason the floor of the filter is sometimes made wavy in section and the laterals are placed in the depressions so formed. Various arrangements are illustrated in Figs. 125 to 128.

To conduct the water to the lateral drains, coarse gravel an inch or two in diameter is filled about the drains and spread in a layer of 6 inches or more in depth evenly over the floor of the filter, or, if the bottom of the filter is irregular, it may be arranged as shown in Fig. 127. Above this coarse gravel are then placed three or four layers of finer gravel, each successive layer being finer in size, but not so fine as to settle into the previously laid layer. The last layer is made fine enough to support the sand. The thickness of these layers need be only 2 or 3 inches if carefully laid, or just sufficient to insure that the next layer below is well covered. In many of the old filters as much as 3 or 4 feet of gravel was used, with very large sizes at the bottom, but as it has little or no duty except to act as a drain, any depth above what is needed for this purpose only adds to the expense of construction. It will be seen that the frictional resistance in gravel only 1 or 2 inches in diameter is very small at the velocities which obtain.

The gravel used should be carefully screened and, if dirty, washed. It is readily sized by revolving or fixed screens, using for this purpose three or four different sizes. The smallest should have about a $\frac{3}{16}$-inch mesh, and each larger size about double the size of mesh of the next preceding. At Albany the sieves used were $\frac{3}{16}$, $\frac{3}{8}$, 1, and 3 inches respectively, all of the gravel being required to pass through the 3-inch sieve.

517. *Special Arrangements.*—In some cases, in place of lateral drains covered by a deep layer of gravel, a cellular floor is used. This may be made by laying brick flatwise, with narrow open joints, upon other brick placed at right angles thereto, an arrangement which requires but little gravel and occupies but little space in the filter. In other cases, drain-tile has been laid at right angles to the main drain so as to cover the entire bottom. Still other arrangements have been employed and various special tile used. The only office of the drainage system is to furnish a channel for the flow of the water with a certain minimum loss of head, and that arrangement should be used which will accomplish this in the most economical manner and leave a level bed for the sand layer.

518. *Examples.* — Fig. 125 illustrates the drainage arrangement of the Hamburg beds. The filters have an area of 1.89 acres each. The central drain is 22 by 32 inches, with brick sides and masonry cover. The laterals are 6 inches wide and $7\frac{1}{2}$ inches high, and are spaced about 30 feet apart. The gravel layer is 2 feet thick. *

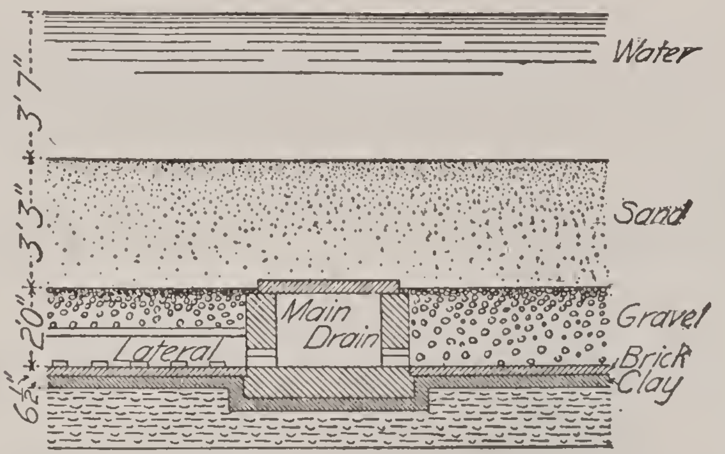
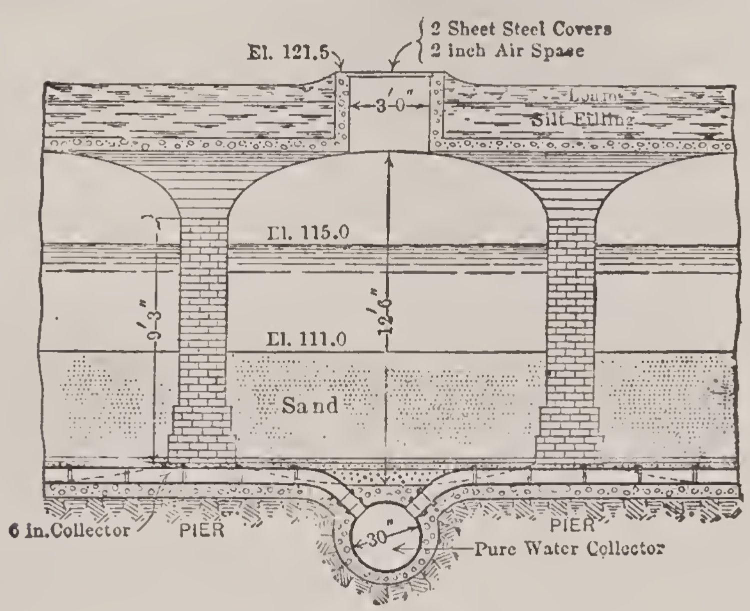
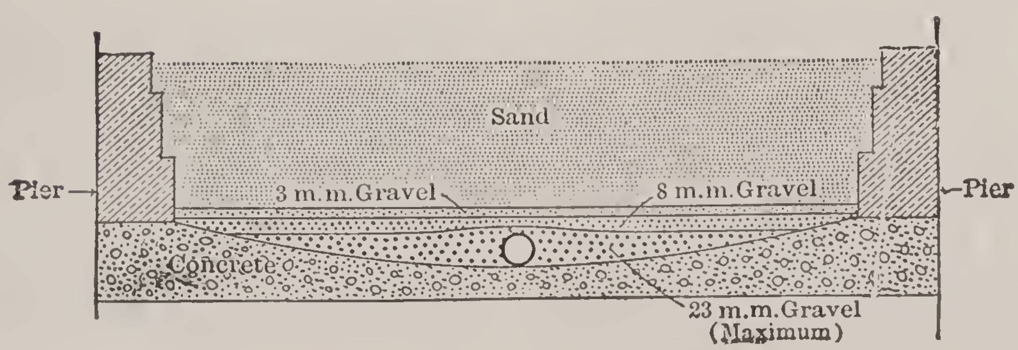


FIG. 125.— SECTION OF HAMBURG FILTER.

The arrangement of drains in the Albany plant is shown in Fig. 120, page 465, and sections through a main drain and laterals in Fig. 126. The filter



Section through Main Drain.



Section through Lateral.

FIG. 126.— DETAILS OF DRAINS, ALBANY FILTER-BEDS.
(From Trans. Am. Soc. C. E., vol. XLIII.)

is covered, and a 6-inch lateral is laid in each space between the rows of piers. The drains are of vitrified pipe, the laterals being laid with open joints.

Fig. 128 is a plan of bed and a section through the central drain of one of

* Meyer. Das Wasserwerks Hamburg, p. 19.

the Zurich filters. The main drain is of concrete, and the laterals are of tile laid over the entire floor.*

Fig. 127 shows the general design and drainage system of one of the large Philadelphia plants. The arrangement is quite similar to that at Albany, but the more general use of concrete should be noted.

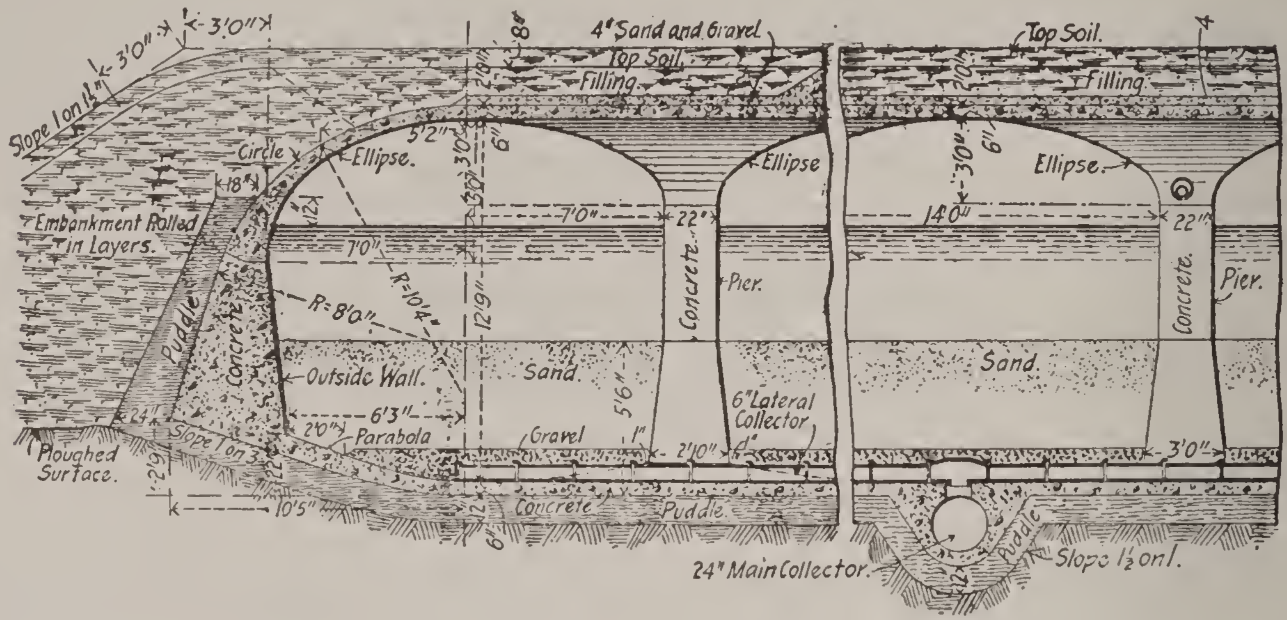


FIG. 127.—DRAINAGE SYSTEM LOWER ROXBOROUGH PLANT, PHILADELPHIA.
(From *Engineering Record*, vol. XLII.)

519. Loss of Head in the Drainage System. — The total loss of head in the filter is equal to the loss of head in the sand plus that in the under-drains. That in the sand is uniform throughout the filter, but in

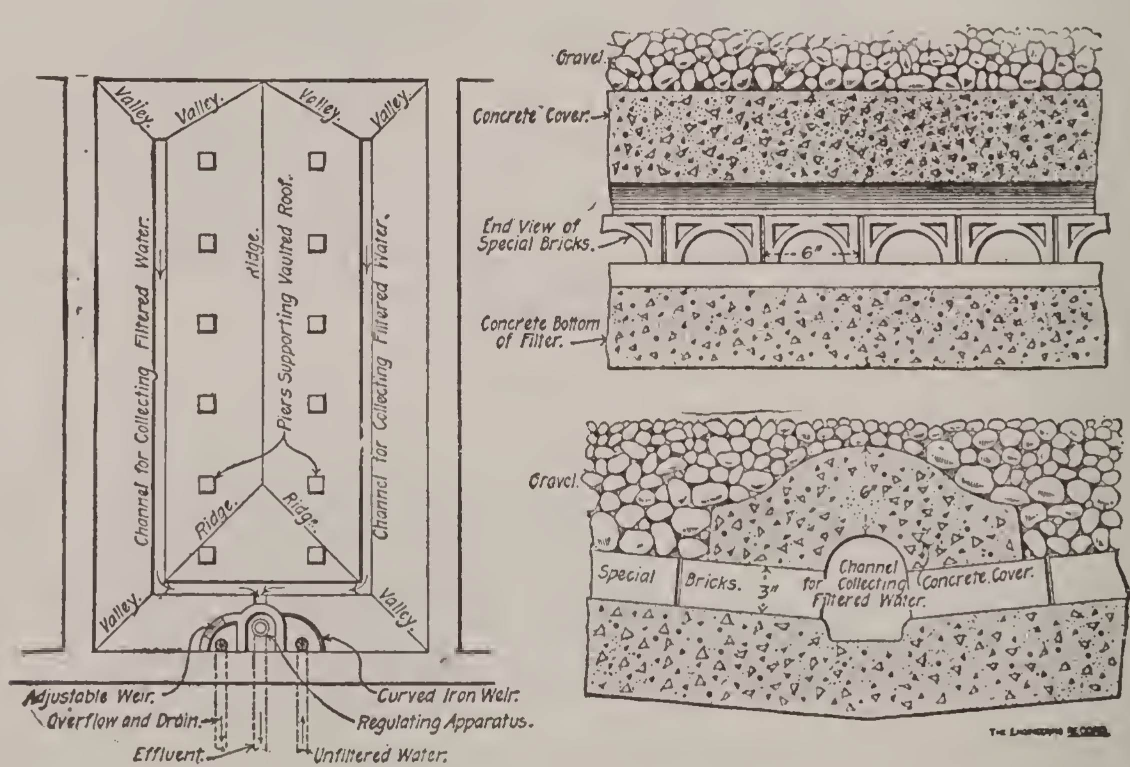


FIG. 128.—FILTER-BED AND DRAIN DETAILS, ZURICH.
(From *Engineering Record*, vol. XXXIX.)

the under-drains it varies from zero near the outlet to a maximum for the most remote point. The rate of filtration will be proportional to the total head and therefore will vary in different parts of the bed.

* *Eng. Record*, 1899, XXXIX. p. 472.

The loss of head in the drains should be kept so low that with a clean filter the variation in the rate of filtration in different parts of the bed will not be excessive. A variation of 20 to 25 per cent would not be a serious matter, as the excess above the average would then be only 10 or 12 per cent. Furthermore, this difference would occur for only a short time after cleaning, for as a filter becomes clogged the relative difference in heads is much less.

If we take, for example, a filter composed of .30 mm. sand, depth 4 feet, rate of filtration 3 million gallons per acre per day, the loss of head due to the sand alone when the filter is clean will be about $.039 \times 4 = .156$ foot. If we allow a maximum loss of, say, one-fifth of this for the drains, or .031 foot, the total head will then vary from .156 to .187, and the rate of filtration will vary about 10 per cent above and 10 per cent below the average. To keep the loss of head in the drains to this low limit requires the use of low velocities and relatively large pipes.

The loss of head in drains according to Kutter's formula is given in Table No. 70. The loss of head in gravel per foot of distance is approximately given in Table No. 71.*

TABLE NO. 70.

FRICTIONAL HEAD IN DRAINS, IN FEET PER 100 FEET OF DRAIN.

Discharge. Gallons per Day.	Velocity. Feet per Sec.	Diameter of Drain in Inches.									
		4	6	8	10	12	15	18	20	24	30
700×(diam.)^2	.2	.012	.006	.004							
1400 " "	.4	.050	.025	.016	.011	.009	.006	.005	.004	.003	.002
2100 " "	.6	.113	.057	.036	.025	.019	.014	.011	.009	.007	.005
2800 " "	.8	.202	.101	.064	.045	.035	.025	.019	.016	.012	.009
3500 " "	1.0	.315	.158	.100	.070	.054	.039	.030	.025	.019	.014

TABLE NO. 71.

FRICTIONAL HEAD IN GRAVEL PER FOOT OF DISTANCE.

Rate of Flow. Gals. per Day per Square Foot of Cross-section	Effective Size of Gravel in Millimeters.			
	10	20	30	40
500	.00035	.00012		
1000	.0007	.00025		
2000	.0014	.0005	.00025	
3000	.0022	.0008	.00037	.00025

* Based on results of experiments of Mass. Bd. Health, Report for 1892, p. 555.

The loss of head in the gravel can be kept low either by means of a thick layer, or by putting the drains close together. Wide spacing requires fewer drains, but larger sizes and more gravel. When the cost of drains and gravel is known, the most economical arrangement for a given loss of head can be determined by a few trials.

Thus with a rate of 3 million gallons per acre per day (equal to about 75 gallons per square foot per day), and drains 20 feet apart, the total flow through each foot of width of gravel will be $10 \times 75 = 750$ gallons. With 6 inches of 20 mm. gravel the average flow per square foot will be $2 \times 750 = 1500$ gallons, and by the above table the loss of head is seen to be about .00037 foot per foot. The average distance travelled is 5 feet, hence the total loss of head in the gravel will be .0018 foot. This is a very small loss and would usually be much smaller than necessary. A still thinner layer of gravel might therefore be used, or the drains placed farther apart.

The maximum length of drain (main and lateral) for beds of one acre in area will be about 350 feet. If the total loss is to be kept down to, say, .03 foot, this will allow but about .008 foot per hundred feet in the drains. Inspection of Table No. 70 will show that it will be necessary to use velocities of .2 to .3 foot per second in laterals, and .6 to .8 foot in main drains. The necessary size for any given capacity is readily computed. The size of main drain should increase towards the outlet. In the above example, 6-inch laterals 20 feet apart would themselves consume about .023 foot of head, an amount too large where the total allowable loss is only .03 foot. Eight-inch drains 20 feet apart would use only .0035 foot, leaving about .026 foot for the main drain. This can then be made up of sizes varying from 12 to 30 inches. With thin beds of coarse sand the difficulty of maintaining uniform rates is evidently much increased.

In the Washington filters the loss of head in the various parts of the filter is equalized by the use of brass orifices of different sizes inserted at the points of connection between main and lateral drains. This arrangement permits the use of a smaller main drain than would otherwise be necessary.*

520. **Maximum Total Loss of Head.** — As a filter becomes clogged the head necessary to cause filtration at the assumed rate increases. By allowing the head to increase to a high figure the filter can be operated longer without scraping and so a saving in operation effected. On the other hand high losses of head require more pumping, a

* Trans. Am. Soc. C. E. 1906, LVII. p. 325.

greater depth of filter, and have a detrimental effect in compacting the sand. The efficiency of the filter is little affected. Experiments of the Massachusetts Board show that heads of 70 inches, constantly used there, give substantially the same bacterial efficiency as lower heads. Many filters in use also operate under heads of 4 or 5 feet with good results, and there appears to be no good reason for using less than this. Much higher heads would probably not be economical. Results of operation and experiment show in some cases an increase in time between scraping proportional to the maximum head used, and in other cases the gain in time is much less proportionally than the increase in maximum loss of head.

521. Inlet-pipes. — Water is admitted to the filter through a single branch main at about the level of the surface of the sand. The flow is usually controlled by a balanced valve operated by a float, so as to maintain the water in the filter at a constant level. A gate-valve is provided in addition, to enable the water to be completely shut off at any time. Fig. 129 illustrates the balanced float-valve and details at one of the large Philadelphia plants, while Fig. 130 shows in detail a somewhat different form designed by Mr. D. W. Mead for the filters at Rock Island, Ill. To avoid disturbing the sand as much as possible the water should flow upon the bed at a low velocity, and a common arrangement is to provide a broad weir, as shown in Figs. 128 and 129, over which the water passes. On filling the filter after cleaning, it is necessary then to fill from below only slightly above the surface of the sand before turning on the unfiltered water.

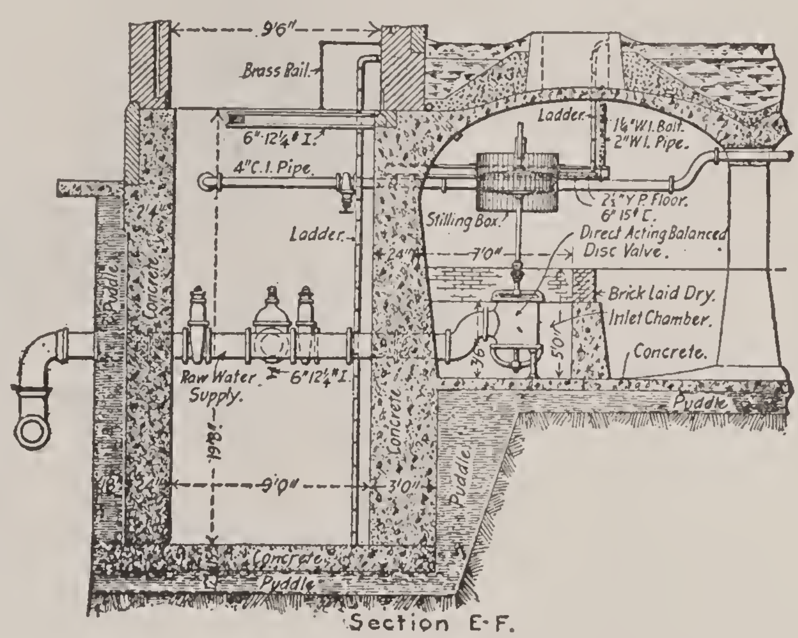


Fig. 129. — Inlet Regulator used at Philadelphia.
(From *Engineering Record*, vol. XLII.)

In place of providing a regulating-valve for each filter the influent pipes may all lead from a central regulating-well in which the water-level is maintained constant. Such an arrangement is suited to a compact group of small filters.

522. Outlet-pipes and Apparatus for Regulating the Head. — If the water-level on the filter is kept constant, the rate of filtration must be regulated, as the filter becomes clogged, by lowering the water-level or reducing the pressure at the outlet. In the older filters no arrangement was provided for regulating each filter independently, but each was connected to the clear-water well by a short pipe fitted with an ordinary valve. The head on all filters was consequently always the same, except as it might be controlled by throttling at the valves. The effect of unequal heads on the rate of filtration, where some of the filters might be freshly cleaned and others badly clogged, can readily be imagined. Independence of action, especially as respects maximum rate, is greatly to be desired and is now the general practice.

The regulation of head requires, first, some form of measuring device, such as a weir, orifice, or Venturi meter by which the rate of

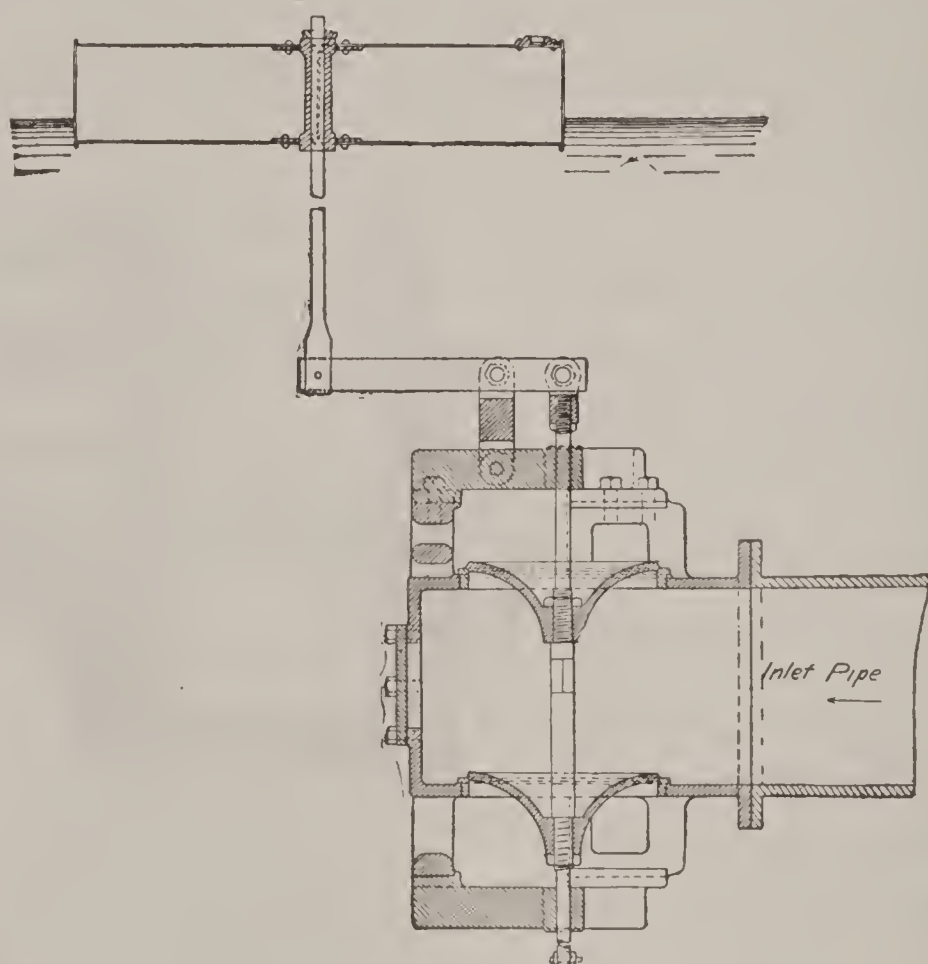


Fig. 130.—Regulating-valve, Rock Island Filters.

filtration can be ascertained at any time by floats and indicators; and, second, the controlling of the rate of flow either by hand or automatically. Floats are also required to show the level on the filter and the head in the main drain, the difference of which is the working head on the filter. The apparatus for regulation is placed in one or more chambers with which the main drain of the filter connects.

523. *Hand Regulation.*—If a weir or orifice is used the rate of flow may be regulated by lowering the weir or orifice itself as the beds become clogged, or by varying the opening in a valve connecting the main drain with the weir chamber. In either case the object sought is to maintain a constant head on the weir or orifice.

The first plan is that followed at Hamburg, the regulating arrangements for which are shown in Fig. 131. The head on the filter at any time is the difference in level between the water in the filter and that in the main drain and chamber connecting therewith; it is indicated by suitable floats. The head on the weir, or the rate of filtration, is also indicated by floats, and is kept constant by moving the weir from time to time as the filter becomes clogged. Instead of a weir like that shown in Fig. 131, a telescopic tube has been used in some places, similar to the form shown in Fig. 134, but adjusted by hand.

In the Albany plant and the plant at Yonkers the second method is adopted, a fixed orifice being used. The design at Albany is illustrated

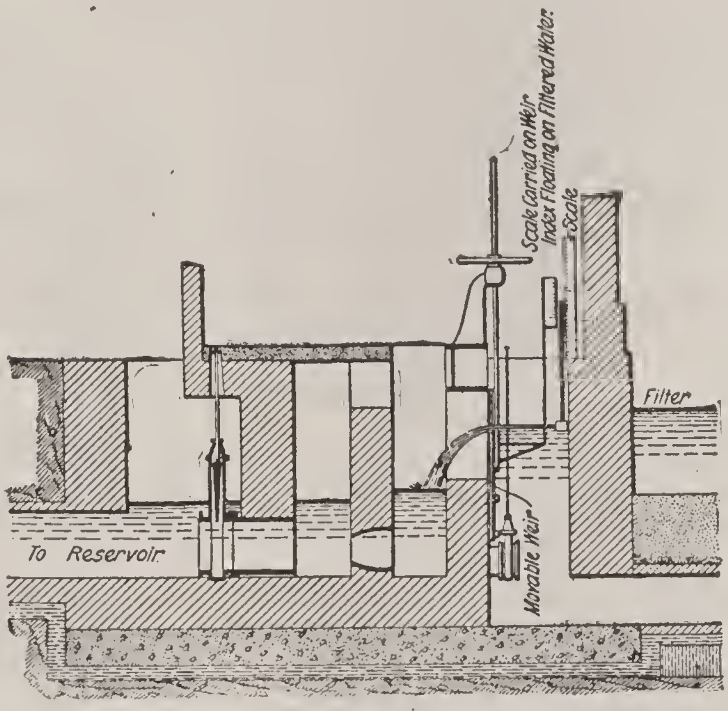


FIG. 131.— REGULATING-APPARATUS AT HAMBURG.

in Fig. 132. The measuring is done by means of an orifice in a wooden partition, a head of 1 foot on this orifice being necessary to pass the standard quantity of water. This head is varied by means of the gate-valve admitting water from the under-drains. The actual head on the filter is measured by the difference between the water-level in the filter and the pressure-head in the pipe just back of this valve. Small float-chambers are provided, connecting with the different points, and suitable floats indicate the loss of head and the rate of filtration. When the rate of filtration exceeds the demand, the level of the water in the clear-water well gradually rises above the orifice, thus decreasing the rate to correspond to the reduced demand. This arrangement was adopted on account of the small size of the clear-water reservoir which local conditions made necessary. Only so much capacity was provided as was necessary to give a reasonable time for the filters to respond to the variations in the demand. The advantage of this arrangement is a smaller loss of head in the system, a smaller clear-water reservoir, and a partial automatic regulation of the filters to furnish the desired quantity of water.

Several of the most recent plants have used the Venturi meter for the measuring device, controlling the rate of flow by hand. This makes a very satisfactory and compact arrangement. Fig. 133 illustrates this arrangement as used at Washington, D. C. Here the effluent pipes from four filters are led to a single chamber. In the design of Fig. 134 the Venturi meter is used in addition to the automatic regulator.

524. *Automatic Regulation.* — Automatic regulators for delivering water at a constant rate are in use in a number of places. They usually

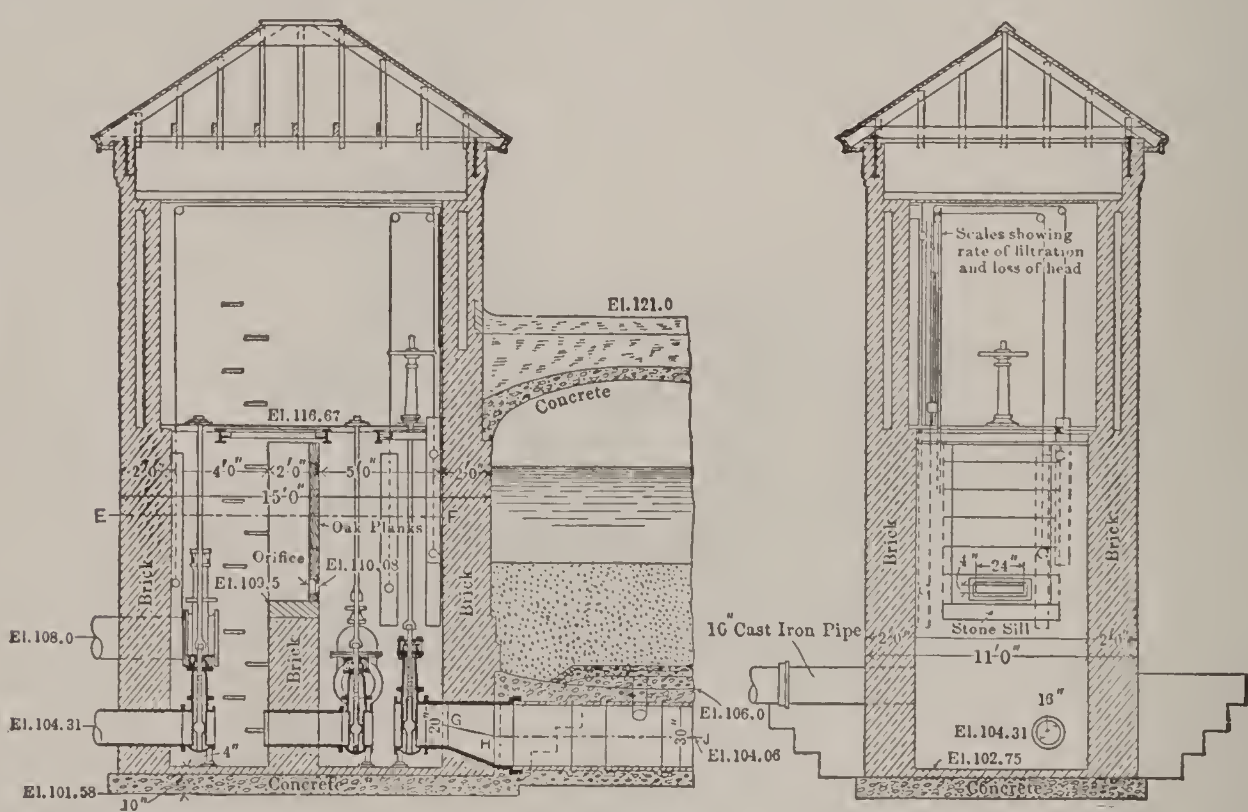


FIG. 132. — REGULATING CHAMBER, ALBANY FILTER-BEDS.
(From Trans. Am. Soc. C. E. vol. XLIII.)

consist of a weir in the form of a telescopic tube which is supported by means of a float in the chamber connecting with the under-drain. By adjusting the float, the edge of the weir can be maintained at any desired distance below the water-surface. A weir of this general type is illustrated in Fig. 134. The rate of discharge is varied by changing the relative height of float and weir. A variation of this form is used at Pittsburg. Here the movement of the telescopic tube and float is arranged to operate a balanced piston valve in the pipe leading from the under-drain. By this means a very slight movement of the float is sufficient to regulate the loss of head and the rate of filtration. A difficulty connected with the use of the open weir is caused by the drawing into the pit of a large amount of air with the water. This may be obviated by using a submerged orifice. A form of balanced pressure-valve devised by Burton * and used for automatic regulation is illustrated in Fig. 135. The quantity of water passing the valve is maintained constant by keeping the difference of pressure on the two sides of an

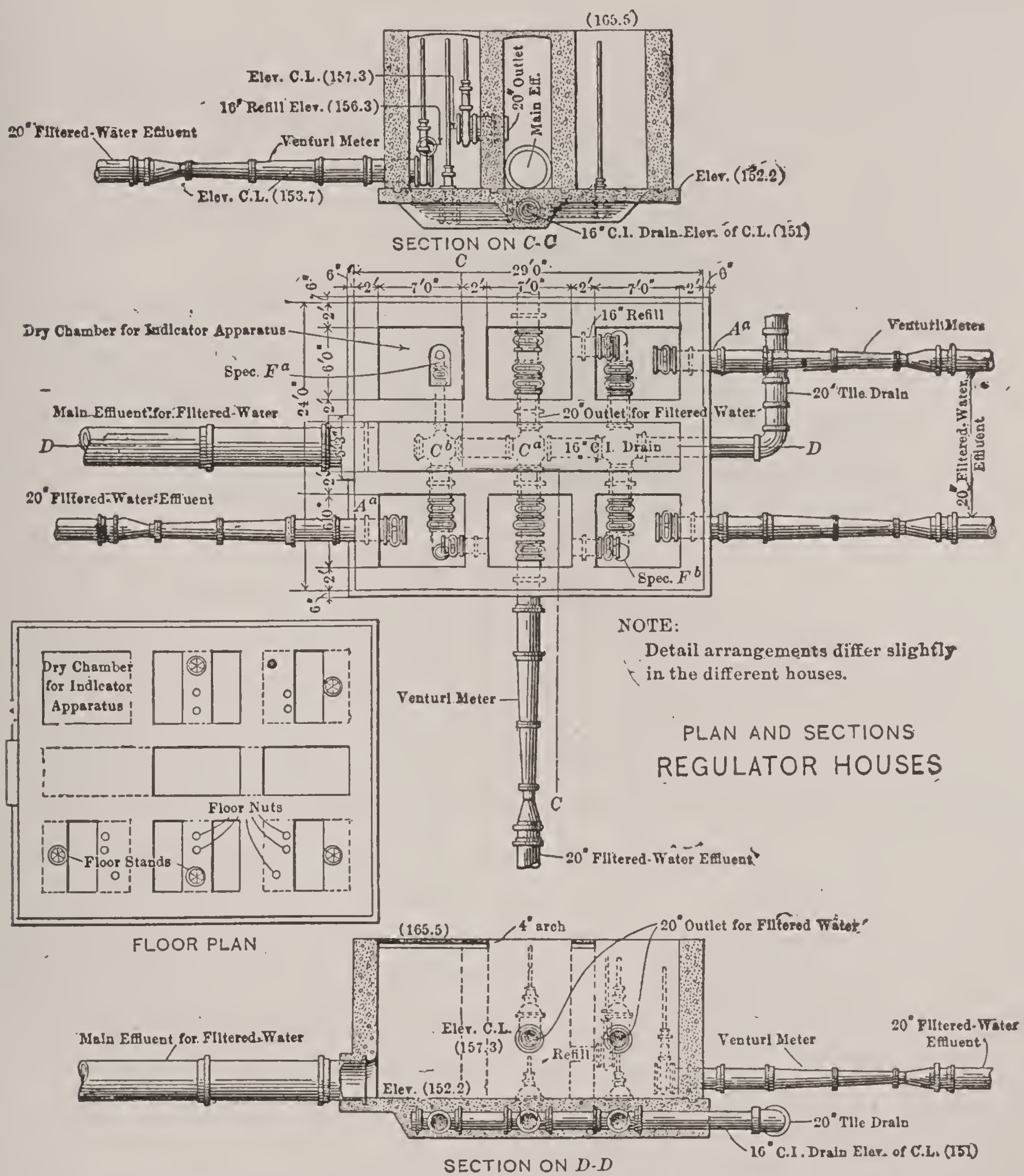


FIG. 133. — REGULATING CHAMBERS, WASHINGTON, D. C.
(From Trans. Am. Soc. C. E., vol. LVII.)

orifice in the plate *e* a constant quantity. This is done automatically by the balanced valve *c*, controlled by the piston *d*, which is open to water-pressure both from the outside well and from the valve-chamber. A somewhat similar form is shown in Fig. 138k, of the next chapter.†

* Proc. Inst. C. E., CXII. p. 321.

† See paper by Anthony on Automatic Modules, Trans. Am. Soc. C. E., 1903, LI. p. 136.

525. Other Pipes and Valves.— Besides the inlet- and outlet-pipes, a drain-pipe must be provided through which the water may be drawn off. This is usually connected with the chamber into which the main drain opens, as shown in Figs. 133 and 134. An overflow-pipe is also

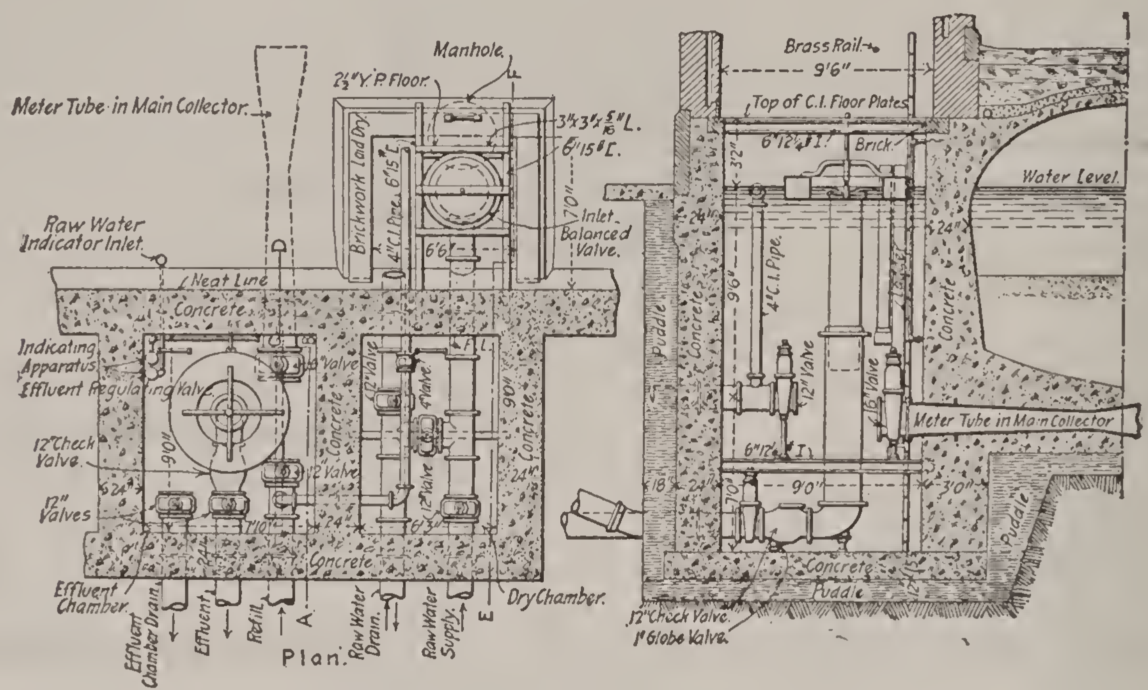


FIG. 134. AUTOMATIC REGULATOR. PHILADELPHIA.
(*From Engineering Record*, vol. XLII.)

necessary to provide against any failure on the part of the inlet-regulator. This connects with the drain-pipe. (See also Fig. 128.)

After a filter has been drained and cleaned it is desirable to fill with filtered water from below to a short distance above the sand. If the water in the pure-water basin is at a level higher than the surface of the sand, water can be admitted from it to the under-drains by means of a by-pass around the regulating-apparatus or through the partition-wall. If the pure-water basin is too low for this, water may be piped from an adjoining filter which is in operation.

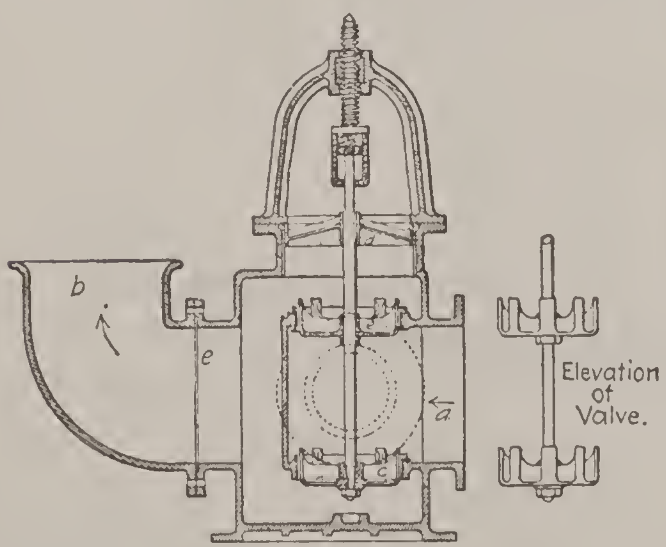


FIG. 135.— AUTOMATIC REGULATING-VALVE. (BURTON).

Arrangements should be made for wasting the filtered water in case it should be necessary, also for drawing off the water from above a filter down close to the sand layer in order to save time in emptying; and facilities should be provided for sampling water from various points in the system. By-passes should be provided to enable either settling-basin or filters to be cut out if necessity arises. For furnishing water for sand-washing and various purposes, connection must be made with high-pressure mains.

526. General Arrangement of Piping. — The location of main supply-pipe, effluent-pipe, and drain varies according to local conditions. At Albany (Fig. 120) the supply-pipes and inlet-chambers are placed along one end of the beds, while effluent- and drain-pipes, with regulating-chambers, are placed along the other end. At Hamburg a similar arrangement is adopted. At other places, as at Berlin, all pipes and chambers are placed along one side of the group of filters. This is a more compact arrangement, and is the more common one in modern plants. For convenience of operation, several filters, two to six, should be operated from a single regulating house. This concentrates the operating mechanism and aids in supervision. Covered valve-chambers or gate-houses should be provided with open filters as well as with closed.

527. Pure-water Reservoir. — Where practicable a pure-water reservoir should be provided of sufficient capacity to prevent the necessity of frequent variations in the rate of filtration. If this is not done, it is at least necessary to furnish a capacious pump-well to prevent the fluctuations of the pumps from being directly felt by the filters. To enable the filters to be independently regulated the highest level in the pure-water reservoir should always be lower than the level of the water in the regulating-chambers of the filter. This may not always be practicable, as at Albany. From the data of Chapter II, Art. 31, it will be found that to equalize the supply for an ordinary day requires usually from two to three hours' average consumption, and if the pure-water reservoir is given this capacity, plus a moderate fire reserve, it will be necessary to vary the rate of the filters but slightly from day to day.

528. Cleaning Filters. — When a filter has become clogged and has reached its highest allowable loss of head, it is drained and then cleaned by removing the layer of clogged sand which is usually from $\frac{1}{2}$ to $1\frac{1}{2}$ inches thick. The scraping is ordinarily done by using broad, thin shovels, but at Pittsburg a sand scraping machine has been adopted which is expected to be more economical. A distributing machine is also to be used there.* Sand is removed by wheelbarrows, or, as now more generally done, by portable ejectors (see Art. 532) to sand washers where it is cleaned and stored or returned to another bed. After scraping, the filter is filled, preferably from below, with filtered water until covered 2 or 3 inches deep; then raw water is run on to the

* *Eng. Record*, 1906, LIV, p. 664.

usual depth, and the filter again started into action. At intervals of a year or so, and before the layer of sand has been reduced below a desirable minimum thickness, the bed is restored to its original depth by the addition of clean sand. The minimum thickness allowable by the German rule is 12 inches, but a considerably greater thickness is to be preferred for the reasons already given in Art. 514. At the last cleaning before refilling, a thicker layer than usual should be removed, and the remaining sand to a depth of several inches dug over and loosened. This procedure is to avoid the effect of stratification and to aerate the filter to some extent. At some works it is the practice to occasionally remove all the remaining sand and even the gravel. After cleaning and filling, the filter should be started slowly and gradually. At some works it has been found beneficial to allow the raw water to stand upon the filter for several hours before operation begins, in order that some sediment may collect on the surface and so hasten the establishment of the surface sediment layer.

529. *Cleaning Open Filters in Winter.*—To accomplish this properly is the chief difficulty in operating open filters. If the ice is removed and the filter drained and cleaned in the usual way, there is much danger that the sand will be frozen and the operation of the filter greatly interfered with. It may also be very inconvenient to wait for a warm spell in which to do the work. To avoid removing all the ice, filters have been cleaned one-half at a time. The ice is removed from one-half of the bed, the bed drained, and that half cleaned. Water is then admitted, the remaining ice floated to the clean half of the bed, the bed drained, and the other half cleaned. At Hamburg, filters have been cleaned without draining by means of a special form of dredge suspended from a scow and pulled back and forth across the filter. More recently a special device has been in use consisting of a scraper and a large pouch to hold the sand. The whole is attached to a large float and is pulled back and forth under the ice by means of cables, it being thus necessary to cut away only a strip of ice along each side.

530. **Period of Service.** — The period of service is the time that elapses between two scrapings of the filter. It may be measured in days or in an equivalent manner in terms of amount of water filtered. The period of service depends upon the character of the water, upon the fineness of the sand, and upon the maximum allowable loss of head. It is directly affected by the rate, a rapidly working filter becoming clogged proportionally sooner. In practice it varies from a few days, if the conditions are especially bad, to five or six weeks or more where the conditions are good. The amount of water filtered between

cleanings ordinarily ranges from 40 to 80 million gallons per acre. For many waters the worst period is in the algal season.

531. **Effect of Scraping on Efficiency of Filtration.** — In many cases there is a considerable decrease in the efficiency of a filter for some time after scraping, and in some works it is the practice to waste the effluent for one or more days at this time. At other places it has been found sufficient to begin the operation very slowly after scraping. This method is followed in a number of the larger German filter-plants. In the Massachusetts experiments there was, in many cases, no deterioration of the effluent after scraping; in others, such was not the case. As has already been noted (Art. 504) the effect of irregularities in operation, including that of scraping, was, in these experiments, greatest with thin filters and with coarse sand. The effect depended also upon the depth of sand removed and on the subsequent treatment. Filling a filter slowly from below was found to give much better results than filling from above. A good effect was also observed if the water was permitted to stand a few hours on the filter before starting the operation. If these precautions are followed, there is likely to be little need of wasting the effluent, but the necessity for this in any particular plant can be readily determined by experience. When the sand is renewed the necessity of wasting the effluent is much greater. In the operation of the Albany plant the effect of scraping is very small in the warmer months. During the winter months, however, the effect is marked. The effect of the occasional refilling is also very marked. Detailed data are given in Table No. 71A.*

TABLE NO. 71A.

BACTERIAL RESULTS FROM ALBANY FILTERS DURING PERIODS OF SCRAPING AND REFILLING, 1899–1903.

Time.	256 Scrapings during the eight warmer months (April to November).	115 Scrapings during the four colder months (December to March).	Fourteen Refillings.
Third day before	44	194	68
Second day before	48	213	52
Last day before	62	272	68
First day after	91	386	498
Second day after	74	741	570
Third day after	82	1100	444
Fourth day after	91	1468	461
Fifth day after	82	1312	586
Raw water	14500	74800	25200
Mixed effluent	60	596	131
Average efficiency of whole plant for the same periods	99.58%	99.20%	99.48%

* Trans. Am. Soc. C. E., 1904, LIII. p. 247.

532. **Sand-washing.** — Various methods have been employed for washing dirty sand, two of which deserve notice. The revolving drum washer, used largely in Germany, at Berlin and other places, consists of a large iron drum, slightly conical in form and open at both ends. The axis is horizontal. Sand is run in at the large end, and as the drum revolves it is gradually moved towards the smaller and higher end by means of interior screw-blades. Water enters at the other end and in flowing over the sand thoroughly cleans it. The amount of water required at Bremen is stated to be 7 to 8 times the amount of sand washed,* or about 1500 gallons per cubic yard of sand.

The other form, known as the ejector sand-washer, has been in use

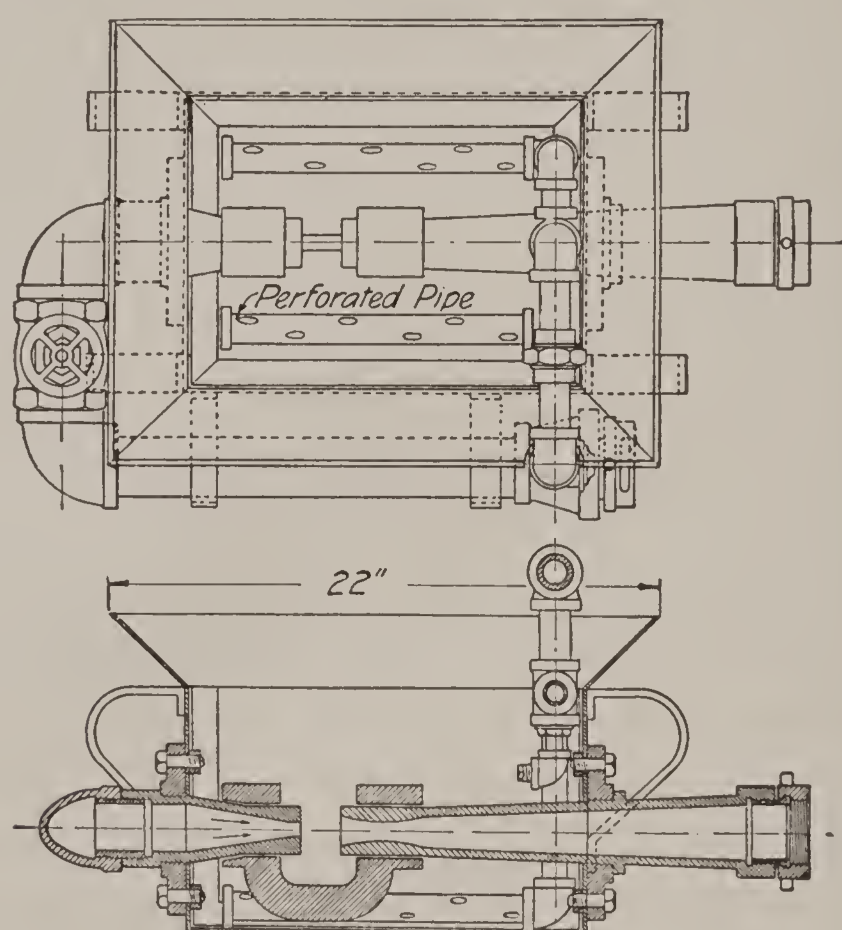


Fig. 136. — Portable Ejector, Washington, D. C.

in England for many years. It has also displaced the drum washer at Hamburg and is now generally employed in this country, both for elevating and washing the sand. The filter plant is fitted up with high-pressure water mains, a 3- or 4-inch branch running to each filter. In removing the sand from a bed a portable ejector is connected up with the high-pressure pipe line by means of a short line of hose. The sand is then shoveled into the ejector which forces it through another line of pipes to the washer. There it is washed by other sets of ejectors and forced again through pipes to storage or back at once to one of the

* For details see Trans. Am. Soc. C. E., 1904, LIII. p. 227.

filters. Very little manual labor is required and the sand is handled very economically.

The design of ejectors and pipe system was very carefully worked out at the Washington plant. Fig. 136 shows the portable ejector there used. The sand is shoveled into the steel box, is there lifted and made liquid by water forced through the perforated pipes near the bottom of the box, and is then carried away by the action of the ejector jet. The mixed sand and water is carried through a 4-inch pipe to the washer. Fig. 136a shows the ejector used for washing purposes. The sand, containing a large proportion of water, is discharged into the hopper from above, the water overflowing the edge and carrying away with it the dirt. The clean sand settles and is forced out through the ejector at the bottom. As the ejector tends to carry out more water than it supplies, some of the dirty water from above would be carried along with the sand if no additional water were supplied. To avoid this an auxiliary supply is introduced near the bottom sufficient in amount to prevent a downward current. By this means a single hopper will effect good results although two hoppers are provided which may be operated in series. In earlier plants the auxiliary jet was not used, with the result that four or five hoppers were required. A photograph of the complete washing apparatus is shown in Fig. 136b.*

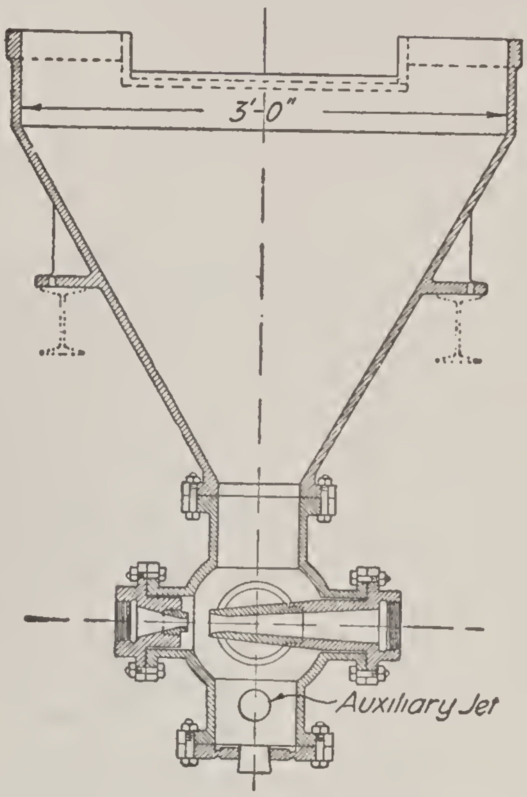


FIG. 136a. — EJECTOR WASHER, WASHINGTON, D. C.

A valuable investigation was also made at the Washington plant on the flow of mixtures of sand and water through pipes. It was found that velocities of from 3 to 4 feet per second were needed to prevent stoppage.†

The cost of cleaning and replacing sand will usually range from $1.00 to $1.50 per cubic yard. At Washington it is estimated to cost but 40 cents per cubic yard for labor. From 1500 to 2500 gallons of water are used per cubic yard of sand.

533. Bacterial Control of Filter Operations. — The most accurate way in which to control the operation of filter-plants is to subject the water to a bacterial examination. This should be made at frequent

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* See reference No. 43, p. 501, relative to new method of sand washing.

† Trans. Am. Soc. C. E., 1906, LVII. p. 586.

intervals so as to note any possible changes in quality. The experience with European filter systems has shown that an impairment in quality has not infrequently been detected in time to prevent outbreaks of disease. In the larger filter-plants, a bacteriological laboratory should be installed, and daily tests of the effluent made. The filter-beds should be arranged so that the effluent from each can be tested separately, and provision made so that the filtered water can be rejected from any one filter if not up to standard.* In this way a scientifically controlled study can be made of all the filter operations and optimum con-

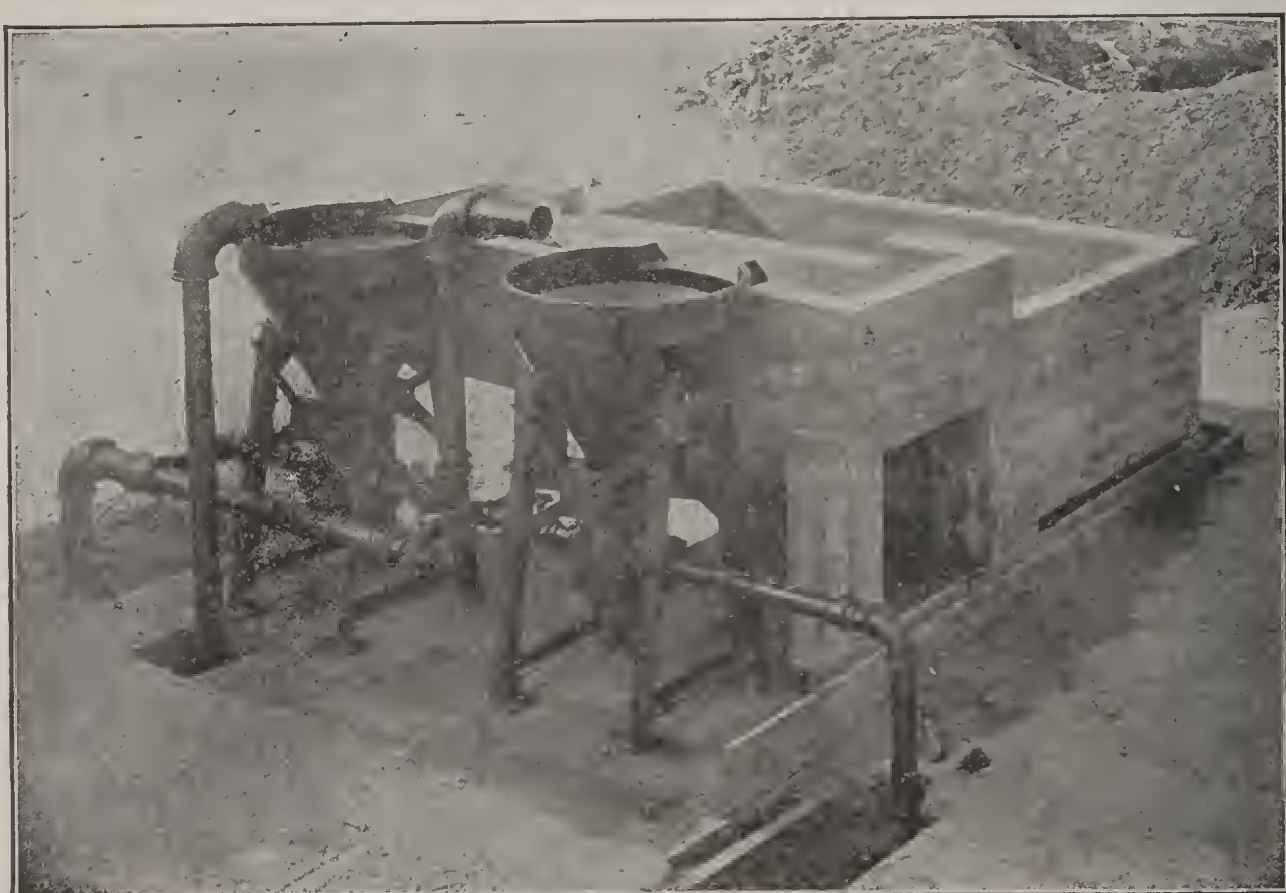

Fig. 136b. — Sand Washer, Washington, D. C.
(From Trans. Am. Soc. C. E., vol. lvii.)

ditions as to rate of filtration, cleaning, filling, etc., determined. In Germany, compulsory examination of all sand filters is now in force, reports of the working of the same being sent to the Imperial Board of Health at stated intervals.

The control of such operations is a matter of some importance. If the examinations are conducted under the direct supervision of the superintendent of works, it is possible to more satisfactorily study the problems that arise in connection with the operation of the filters; but at the same time, tests made by disinterested parties, such as Boards of Health, are received with more confidence by the public. The work,

* Koch traced the cholera outbreak in Altona in the winter of 1893 to the imperfect operation of one filter

while requiring familiarity with bacteriological technique, is of such a character that it can be carried out under proper supervision by persons having but limited experience in bacteriological work.

Rules for Bacterial Control. — The rules formulated in 1898 by the German Imperial Board of Health are still representative of good practice. They are in brief:

1. Each filter shall be tested daily. This necessitates an arrangement to secure samples from drains at any time, a feature that is now regarded as essential for bacteriological work, but one which has frequently been neglected in past construction.

2. Rate of filtration must not exceed 100 mm. per hour (2.57 million gallons per acre per day).

3. No filtered water should be admitted to the mains that contains more than 100 bacteria per c.c.

This quantitative limit is purely arbitrary, but good filter practice indicates that this is not beyond reach. Generally speaking, the average of properly constructed filters will fall below this, although, as has been noted previously, even in the best-manipulated filters there is considerable variation in germ content from day to day. An additional valuable control, which is coming to be frequently employed, is the test for the presence of *B. coli*. In testing filters as to their efficiency, samples should be collected at periods when the effluent is likely to be the least favorable, as during frost periods, heavy rains, and periods of greatest consumption.

534. **Preliminary Treatment of Water for Slow Sand Filtration.** — Nearly all waters contain at times suspended matter to such an amount, or of such a character as to render desirable a preliminary treatment for the removal of a portion of this sediment. This may be simply a question of the most economical method of treatment, or the water may be of such character as to render such preliminary treatment necessary for satisfactory results. Large quantities of clay or silt clog up a filter quickly, and if the sediment is very fine it penetrates deeply into the filter and may make the effluent turbid.

Preliminary treatments may consist of simple sedimentation, sedimentation with coagulation, or preliminary rapid filtration with or without coagulation and sedimentation.

535. *Sedimentation.* — In the filtration of river-waters it will nearly always be economical to provide at least a few hours' preliminary sedimentation. This subject has already been discussed in Art. 466.

While river-waters are most subject to great turbidity, supplies derived from lakes may also give trouble from this cause. A noteworthy example is at Ashland, Wis., where, during the "break-up" of the ice in the spring with strong wind action, the bay from which the water-supply is derived is rendered so turbid as to greatly impair the efficiency of the filtration process. Examinations made by Russell * during such a period showed only 20 to 30 per cent bacterial efficiency. In some cases the period of service of the filters was reduced to four days; and under such conditions the effluent was quite cloudy. The various details connected with the construction and operation of settling-basins are discussed in the preceding chapter.

536. *Sedimentation with coagulation.* — The elaborate experiments at Cincinnati, Louisville, and New Orleans, and the accumulated experience in treating the water of the streams in the Mississippi Valley, have shown that filtration preceded only by plain sedimentation is inadequate to give satisfactory results. Ordinarily from 30 to 50 parts of suspended matter per million can economically be taken care of by the filters, although it is not the amount but rather the nature that determines whether a good effluent can be secured. Where this is not possible the use of a coagulant is necessary. This should be employed in connection with settling-basins as described in Chapter XX. The construction and operation of the filters is the same.

Experience in the operation of the slow sand-filter plant at Washington, D. C., has also shown that perfectly clear water cannot always be secured even with the long period of sedimentation there obtained. The amount of the turbidity is not, however, sufficiently great to render the use of a coagulant necessary.

537. *Preliminary Filtration.* — In purifying badly polluted waters, and especially those of high turbidity, some form of rapid filter may often be adopted to advantage for preliminary treatment, slow sand filters being employed for the final process. In other cases a preliminary filter may be used for reasons of economy, the increased rate thus permitted in the main filters effecting a greater saving than the cost of the preliminary treatment.

At Albany, rapid sand-filters have been adopted for preliminary treatment after a careful study of the most economical method of enlarging the capacity of the present slow sand-filter plant. A rate of about 100,000,000 gallons per acre per day will be used in the rapid filters with twelve hours plain sedimentation. This will allow the slow sand-filter plant to be operated at a rate of about 6,000,000 gallons per acre

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* Purification of Ashland Water-supply by Sand Filtration, 16th Rept. Wis. Bd. Health, 1895, p. 78.

per day, thus doubling its capacity. Considerable saving in cost will be effected and a more reliable effluent secured than if the present plant were duplicated.

At Philadelphia, preliminary filters are used in some of the plants. At the Belmont plant these consist of rapid sand-filters operated at a rate of about 80,000,000 gallons per acre per day. At the lower Roxborough plant they consist of so-called "scrubbers" designed by Mr. P. J. A. Maignen and described in Art. 556 (Chap. XXIII). These remove about 60 per cent of the turbidity and 75 to 80 per cent of the bacteria, and enable the sand filters to operate at a rate of about 6,000,000 gallons per acre per day, with a saving in cost. Preliminary filters of the Maignen type are also used at South Bethlehem, Pa., slow sand filters being operated at a rate of 7,000,000 gallons per acre per day.*

538. Double Filtration. — Double, slow sand filtration is in use in a number of European works, notably at Bremen, Germany, Shiedam, Holland, and Zurich, Switzerland. Two sets of sand filters are used, operated in about the same manner, although the rates of filtration are usually different in the two sets. At Bremen, this method was adopted especially to secure adequate results at times of floods when the bacterial content in the raw water is very high. A somewhat higher rate than the normal is used in the final filter.† Some typical bacterial results secured during a period of high water are here given:

BACTERIA PER CUBIC CENTIMETER.

Raw Water.	Preliminary Filter.	Final Filter.
4500	94	10
9600	96	9
29000	105	2
39200	130	10
38300	525	15
35600	385	32
17200	165	35
7600	100	30
6400	75	13

This method of double filtration should be distinguished from the use of rapid preliminary filters as described in the preceding article. It is quite probable, however, that some method of rapid filtration such as used in the United States would prove more advantageous than the method of double sand filtration here described.

* *Eng. Record*, 1905, LII. p. 61.

† Trans. Am. Soc. C. E., 1904, LIII. p. 210.

539. Intermittent Filtration. — By intermittent filtration is meant filtration of water through sand in an intermittent instead of a continuous manner, thus permitting the filter-bed to be exposed to the influence of air during the periods of rest. This method is the one used in sewage filtration and is necessary in that case because of the large amount of organic matter present. In water purification, however, the amount of unstable organic matter present is never very large, and it is found by experience that continuous operation gives practically as good results as intermittent operation. This is due to the fact that the amount of oxygen dissolved in the water is sufficient for the nitrification process without aeration of the filter-bed.

A few plants have been constructed to operate on the intermittent plan, the most noteworthy and one of the first plants built in the United States being that at Lawrence, Mass. The intermittent plan was there adopted because of the excessively polluted water to be treated, it being thought at that time that continuous operation would not give satisfactory results. Besides the method of operation, certain other special features were introduced on account of the necessity for great economy. The filter was composed of a single bed of $2\frac{1}{2}$ acres and constructed without a water-tight bottom. The drainage system was made very much less extensive than in ordinary filters, considerable lateral movement through the sand thus being necessary. The cost of the filter (open) was only $26,000 per acre.

On account of the demand for water the Lawrence filter has been operated more generally as a continuous filter and it has been found that the results, both chemically and bacteriologically, are practically the same by both methods. (See data in Art. 489.)* While the first cost was very low the cost of operation has been very high, owing to the lack of division walls and the trouble from ice in the winter. Additional filters built in 1906 are of the usual covered type.

Following the practice at Lawrence, an intermittent filter was constructed at Mt. Vernon, N. Y. This plant, like the Lawrence plant, is now, however, operated on the continuous plan. The experience of these cities and the results of operation of many continuous filters handling badly polluted water indicate that intermittent operation is more expensive and troublesome than continuous operation and that it is rarely if ever advantageous.

540. Cost of Filters. — The cost of sand filters depends greatly upon local conditions as influencing cost of excavation, cost of sand, etc.

* Trans. Am. Soc. C. E., 1901, XLVI. pp. 299, 335.

Large beds and extensive works will cost less per unit area than smaller ones, other things being equal. At Berlin, covered filters of about 0.6 acre each have cost about $70,000 per acre. At Zurich, filters of $\frac{1}{6}$ acre each cost, for the masonry and filtering materials only, about $48,000 per acre for open and $72,000 for closed beds. Engineer Lindley estimates as a reasonable cost in Europe for carefully designed filters about $68,000 per acre for covered and $45,000 for open filters.

At Ashland, Wis., three covered filters of $\frac{1}{6}$ acre each cost $40,178, but the engineer estimated that under normal conditions the cost there would be about $35,000 for beds of $\frac{1}{2}$ acre each, which is equal to $70,000 per acre. At Poughkeepsie a single open bed of 29,640 square feet cost $28,899, equal to $42,000 per acre. At Berwyn, Pa., three open beds of 7500 square feet each cost $18,536, equal to $36,000 per acre. At Albany the cost for eight covered filters of an area of 0.7 acre each was $45,600 per acre, not including land and engineering; the latter item, figured pro rata from the total cost, would add about $2500 per acre. The covers were estimated to have added about $13,000 per acre to the cost. The cost of covered filters at Washington was $75,000 per acre, the high cost compared to that at Albany being due to higher unit prices. To the cost of filters will have to be added the cost of clear-water reservoir, and usually sedimentation-basins, amounting to from $3000 to $10,000 per million gallons capacity according to the circumstances.

541. Cost of Operation. — The principal items in the cost of operation are the scraping of the filters, and the cleaning and renewal of the sand. The cost of scraping will ordinarily range from 60 cents to $1.20 per million gallons filtered, although removal of ice may greatly increase this. It is stated to have cost at London 86 cents per million gallons; at Liverpool, $1.14; at Hudson, N. Y., 88 cents; and at Poughkeepsie as high as $2.78, due to ice. At Lawrence, Mass., the average cost of removal of ice from 1895 to 1900 was about $2.27 per million gallons. The amount of sand removed may be taken at from 1 to 2 cubic yards per million gallons filtered. The cost of washing is about 30 cents per cubic yard, making the cost per million gallons from 30 to 60 cents. Then the items of replacing the sand, repairs, and superintendence will bring the total operating expenses up to from $2.00 to $3.00 per million gallons. At Poughkeepsie the average cost for twenty years has been $2.99 per million.

At Washington the cost is especially low due to very economical methods of sand handling and the comparatively long period of service possible. For the first six months of 1906 the average cost for cleaning and for office and laboratory expenses is given as about $1.25 per million gallons. At the upper Roxborough filters of the Philadelphia plant the cost for 1903 was $0.95 for scraping and washing per million gallons filtered.

The cost of operation at Albany, including superintendence, from July 26, 1899, to July 1, 1900, is given as follows: *

	Av. Cost per 1,000,000 Gals.
Scraping	$0.25
Wheeling out sand	.50
Washing sand { labor	.54
Washing sand { water	.05
Refilling	.39
Cleaning sedimentation basin	.06
Incidentals	.20
Total	$1.99
Laboratory expenses	$0.34

The total cost of filtration, including interest and depreciation, may, under ordinary circumstances, be estimated at from $7.00 to $9.00 per million gallons filtered.

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(See also Chapter XIX.)

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CHAPTER XXII.

RAPID SAND FILTRATION.

542. **General Description of the Rapid Sand Filter.** — This type of filter, also called the "mechanical filter" and the "American filter," is a form of filter designed to accomplish results in the way of purification comparable with those obtained by the slow sand filter already discussed, but with a much smaller sand area. It is similar to the slow sand filter in that the filtering material consists of a bed of three or four feet of sand or crushed quartz, but in other respects the construction and operation are widely different. The essential points of difference are: the very rapid rate of filtration (100 to 125 million gallons per acre per day), the use of a coagulant to aid in filtration and the manner of washing the sand bed. These peculiarities lead to noteworthy differences in construction. The units are relatively small in area, the coagulating basin becomes an essential part of the plant together with adequate means for mixing and regulating the coagulant, and the washing of the sand, which, in this type, must be done every few hours, requires the use of special devices of a more or less elaborate character. In the operation of a rapid filter plant, the frequent attention required of each unit renders the question of compact and convenient arrangement of piping and operating valves of much importance. At the same time the small size of the unit enables this to be readily done, and a part of all the plant to be placed under roof. The washing of the sand beds is accomplished by a reverse flow of water, assisted, usually, by agitation of the sand bed by means of mechanical rakes or compressed air. The details relating to this part of the process constitute the chief differences between the various types of rapid filters.

The development of the rapid filter arose from the effort to settle and clarify very turbid water by the use of a coagulant, followed by rapid filtration. Various devices used in construction and operation, such as sand agitators, supporting screens, coagulant regulators, as well as certain combinations of processes and parts, were patented, and for several years this type of filter was almost exclusively constructed by various filter companies, being built and sold in the form of complete units of wood or steel. When bacterial purification became of greater importance the rapid filter was looked upon with much suspicion, owing to the extremely high rate of filtration used as compared to the rate employed in the better known slow sand filter. Results of daily operation in practice, and of many special experiments have shown, however, that with proper supervision the rapid filter will give essentially the same results as the slow filter, and that in some waters the results are better than can be obtained by the slow filter without the use of a coagulant. This condition has led to the quite general use, in the United States, of the rapid filter whenever it is the better adapted to local conditions. The extent of the present use of this type of filter, as given in Art. 460, is sufficient evidence of its importance as an efficient means of purification. While many of the patented devices are excellent, their use is not essential and several very large plants have been constructed since 1900 by well known engineers in which no such device has been employed.

The name "mechanical filter" has, perhaps, been used to designate this type of filter more commonly than any other, it having been applied at first largely because of the mechanical means used in cleaning the sand and the manner in which complete units were made up and sold. Inasmuch, however, as the fundamental distinction between this type of filter and the slow sand filter relates to the rate of filtration, with the accompanying use of a coagulant and special means of washing, and as the modern plants are now usually being constructed of concrete without mechanical agitators, it would seem that the term "rapid filter" or "rapid sand filter" is more suitable. It will hereafter be the one employed in this work. The term "American filter" has also been used to some extent, the slow sand filter being called the "English filter" in consideration of the places where the respective types originated.

543. **Types of Construction.** — The usual form of rapid filter, as constructed and sold by the proprietary companies, consists of units made up of circular wooden or steel tanks. These contain the sand, supported on suitable strainers, and each is equipped with piping arrangements for washing and means for agitating the sand. In one of the most common designs formerly employed each tank was divided by a horizontal partition, the lower portion acting as a coagulating chamber. The coagulating basins are now usually built separate from the filters so as to provide larger settling capacity. The form of construction here described is illustrated in Fig. 137, which shows one of the filters installed at Chester, Pa., in 1903. The filter unit consists of a cypress tank 15 feet in diameter containing a sand bed $2\frac{1}{2}$ feet thick. This is

supported on a layer of gravel, near the bottom of which are numerous brass "strainer-heads" through which the filtered water passes into a system of wrought-iron collecting pipes. These pipes are connected to a large, central, cast-iron collector which passes through the tank and joins the effluent pipe outside. When the sand is to be washed, water

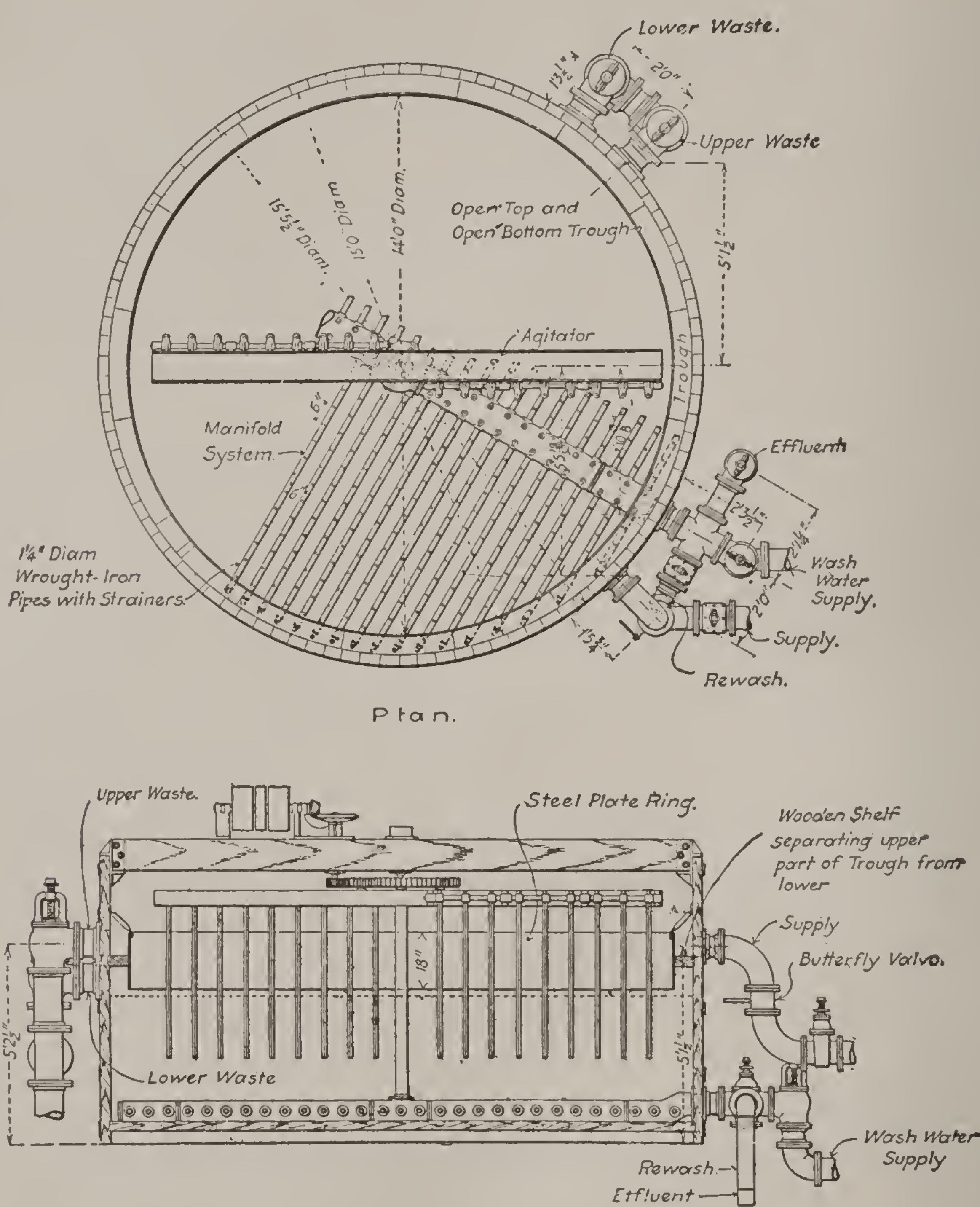

FIG. 137. WARREN FILTER AT CHESTER, PA.
(From *Engineering Record*, vol. XLIX.)

is forced backwards through the strainers, and at the same time the sand is stirred up to its full depth by means of long iron fingers reaching into the sand and which are attached to a transverse arm mounted on a vertical shaft, the whole being rotated by means of suitable gearing. The agitation and upward flow of water thoroughly cleans the sand in a few minutes The waste water escapes into a circular trough supported around the inner edge of the tank, and thence passes to a waste pipe. In this particular form, a lower waste is also provided to assist in washing the surface of the filter by surface agitation and drainage from the top, but without reverse flow. Suitable regulating valves are provided to maintain a constant level of water on the filter and a uniform rate of filtration. Each strainer consists of a perforated bronze plate attached to a cylindrical-shaped strainer-head. These strainer-heads are screwed into the branch pipes which form the manifold system. Further details of strainers are illustrated in Art. 550 *b*.

Instead of mechanical agitators, compressed air may be used for

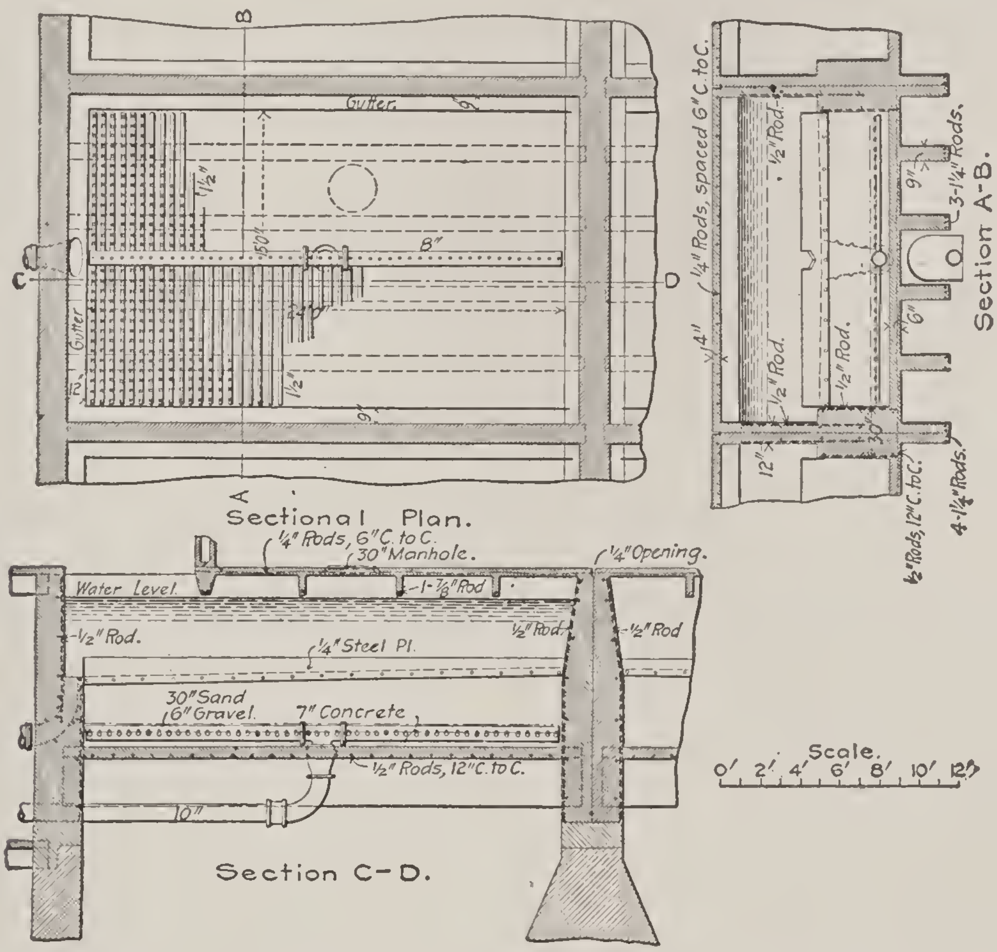

FIG. 138. FILTER UNIT, LITTLE FALLS, N. J.
(From *Engineering Record*, vol. XLIII.)

agitating the sand, the air being forced through the strainers alternately with the wash water. Mechanical agitators of the type illustrated require the use of circular tanks, while compressed air is readily adapted to any form. In another form of commercial filter the entire bed is enclosed in a cylindrical steel tank and is operated under pressure. It is called the "pressure" filter. Compressed air is used to agitate the sand. This type is now seldom used as it is not as satisfactory as the open gravity type.

In many of the modern plants, especially those of large size, the tanks are made of concrete, usually rectangular in form, mechanical agitation not generally being employed. In this case the special devices usually include only the strainer system and the controllers; and several plants have been built where these parts have been furnished by special manufacturing companies, other portions of the plant being designed and constructed independently. Fig. 138 illustrates this form of construction. It represents one of the units of the plant at Little Falls, N. J., the complete plant consisting of thirty-two of these units. As shown, the tank is made of reinforced concrete and partially covered with the same material. The strainer system is, in general, quite similar to that shown in Fig. 137 excepting as to the rectangular arrangement. Compressed air is used for agitating the sand. During the washing process the dirty water is carried off by means of steel troughs leading to a gutter and thence to a waste pipe. The convenient arrangement of piping is an important part of the design of such plants; this feature is discussed in Art. 549 *g*.

544. **Principles of Operation.** — The action of rapid sand filters is somewhat unlike that of slow sand filters, although the results are not greatly different. The effect of a coagulant in gathering the sediment into relatively large masses has been explained in Chapter XX. It aids filtration in this way, and also forms a substitute for the organic coating on the sand grains and on the surface of the ordinary sand filter. It is the use of a coagulant which enables such high velocities to be employed. To avoid too frequent washing, it is common to employ heads as high as 10 or 12 feet, but with such high heads and velocities the sand becomes clogged to a considerable depth. The methods of washing, however, enable this sediment to be readily removed. The interval between washings, i.e., the "run," is 24 hours or less, and the operation of washing requires from 10 to 15 minutes.

In the design of a rapid filter plant the preliminary treatment of the water is often a question of much importance if the most efficient methods are to be used. Very turbid waters can often best be handled by permitting a considerable period of subsidence in large basins, followed by coagulation with a further short period of settling; others may require the use of a coagulant at both periods. Many waters not too turbid can be handled by a single brief period of sedimentation accompanied by coagulation. For effective filtration complete clarification is not desirable as the flocculent precipitate is necessary to secure good results in the filter. In many of the early plants the regulation of the rate of filtration and the quantity of chemical applied was very poorly done, but in the later designs very efficient devices have been introduced to accomplish these objects. In the removal of color the rapid filter is advantageous because of the accompanying use of a coagulant. Brown and peaty waters are quite markedly improved in the process.

545. **Experiments on Rapid Filters and Results of Operation.** — Important experiments on rapid filters have been carried out at Providence in 1893–4 by Mr. E. B. Weston; at Louisville and Cincinnati by Mr. George W. Fuller in 1895–6 and in 1898 respectively; at Pittsburg in 1897 by Mr. Allen Hazen; at Washington in 1899 by Col. A. M. Miller; and at New Orleans in 1901 by Mr. R. S. Weston. Each of these series extended over a considerable length of time and was very carefully conducted. Several shorter series of analyses from plants in regular operation are available and give valuable information.

The Providence experiments were conducted on a small experimental Morison filter. The results were of a rather varying character, but when the filter was considered to be under normal conditions the removal of bacteria was from 95 to 99.9 per cent, averaging about 98½ per cent. The number in the original water was usually from 3000 to 10,000 per cubic centimeter. The rate of filtration was 128 million gallons per acre per day.*

(*a*) *The Louisville Experiments.*† — These experiments were undertaken to determine the efficiency of rapid filters in the purification of the Ohio River water. Besides important results as to bacterial efficiency, much of value was derived in regard to features of construction and operation. Unusual difficulties attended the operation at this place on account of the great amount of sediment carried at times, its greatly varying character, and the fact that the water did not undergo a preliminary subsidence.

With regard to the qualitative results, the turbidity was practically all removed and also a part of the dissolved organic matter. The bacterial efficiency was irregular, but when the filters were operated normally the efficiency averaged from 97½ to 98½ per cent in the various systems experimented with.

The greatest fault of all filters was the lack of adequate settling-tanks, the capacity of those used permitting only from 20 to 60 minutes of subsidence as a maximum. In no case did the filters give

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* Report of the Rhode Island State Board of Health, 1894.
† Fuller. Water Purification at Louisville. New York, 1898.

results satisfactory from the standpoint of economy and efficiency, chiefly because of the lack of a previous settling of the water; and such subsidence for a day or more is regarded as imperative in the treatment of this water. With such subsidence, however, it was estimated that the average amount of coagulant (sulfate of alumina) would be about 1.75 grains per gallon, and with proper attention to the operation it was considered that the result would be thoroughly satisfactory under all ordinary conditions.

Mr. Fuller considered it desirable to employ a layer of sand at least 30 inches in thickness and of an effective size of 0.35 mm., in order to increase somewhat the frictional resistance over that offered by the sand used, which was from 0.43 to 0.51 mm. in diameter. The permissible rate was considered to be 100 million gallons per acre, or over, and the maximum loss of head about 10 feet. In washing it was observed that agitators were of much help, and if the washing was thoroughly done no deterioration of the effluent was noticeable on the renewal of operations.

(*b*) *The Cincinnati Experiments.** — These experiments were undertaken primarily to determine the applicability of slow sand filters, and this phase of the subject has already been discussed in Art. 536. At the same time a rapid filter of the Jewell type was experimented with, using water which had undergone plain subsidence. The character of the water is similar to that at Louisville. The general result as to efficiency was the removal of an average of 98½ per cent of the bacteria, the original number averaging about 27,000. Excluding results obtained at the time when certain changes were being made in the sand, the efficiency was 99.4 per cent, with an average application of 1.25 grains of sulfate of alumina per gallon. Mr. Fuller considered that with about one-third of a grain in addition, "the bacterial results would be as good as is practicable to obtain by any method now known; that is to say, the bacteria in the effluent would average less than 100 per cubic centimeter, and the average annual removal of the river-water bacteria from present evidence would amount to fully 99½ per cent." This is a considerably higher efficiency than obtained at Louisville, but in the latter case no preliminary subsidence was allowed.

Regarding the relative advantages of rapid filters, and slow sand filters with the use of a coagulant, Mr. Fuller considers that the former "would be the less difficult to operate, would be somewhat cheaper,

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* Fuller. Report on Water Purification. Cincinnati, 1899.

and would give substantially the same satisfactory quality of filtered water, and could be much more readily and cheaply enlarged for future requirement."

Other points of value deduced from these experiments were, that the maximum loss of head should be 10 or 12 feet, that the rate could be 125 million gallons or perhaps more, and that provision for at least 6 hours' flow should be made for coagulation and subsidence immediately previous to filtration.

(*c*) *The Pittsburg Experiments.** — These experiments were made on two experimental slow sand filters, one Jewell and one Warren rapid filter, and a set of artificial-stone tiles of the Fischer system (see Art. 554). The average bacterial results for seven months were as follows:

	Bacteria per c.c.	Efficiency per cent.
River water	11,337	
Effluent from Warren filter	201†	98.2
Effluent from Jewell filter	293†	97.2
Settling-basin for slow sand filters	9,224	
Effluent from slow sand filter No. 1	106	99.1
" " " " " " 2	148	98.7

Sand filter No. 2 was operated for four months with unsettled water, otherwise the water for the slow sand filters was given from 12 to 24 hours' subsidence. The rapid filters were operated without preliminary subsidence. The efficiencies obtained throughout the tests were in general very uniform, but a lessened efficiency frequently followed, for a short time, the washing of the rapid filters. The amount of coagulant was found to be of vital importance, and up to 1 grain per gallon the efficiency increased rapidly with the coagulant. With no coagulant it was only 50 or 60 per cent.

(*d*) *The Washington Experiments.* —The city of Washington is supplied with a water which is turbid for a large portion of the year, in spite of the fact that it passes through two large reservoirs. The effect of storage is, in fact, greater upon the bacteria than upon the clay. Experiments were made upon a slow filter, and upon a rapid filter using a coagulant. The results were, in general, better from the rapid filter than from the slow sand filter, although neither gave at all times a clear effluent. The rapid type of filter was recommended by Col. A. M. Miller, but it was thought by Mr. R. S. Weston, who reported on the

* Report of the Filtration Commission. Pittsburg, 1899.

† Excluding inferior results obtained during special experiments.

chemical and biological work, that a slow sand filter with the aid of a coagulant would also give satisfactory results. This system, however, was not tested.*

Slow sand filters were later installed and it is noteworthy that the results of their operation indicate that an entirely clear effluent cannot be had at all times without the use of a coagulant, although the bacterial results are satisfactory. (Art. 536.)

(*e*) *The New Orleans Experiments.* — Extensive experiments were made in 1901 by Mr. R. S. Weston on the purification of the Mississippi River water at New Orleans by both slow and rapid filters. This water has an excessive amount of very fine sediment (Art. 462) and the chief problem was one of clarification rather than bacterial purification. It was found that either system would give satisfactory results when the water received proper preliminary treatment, but that plain sedimentation was an inadequate preparation for even the slow filters. In this case it was found that the most economical method of treatment for rapid filtration consisted in a preliminary period of plain subsidence of about 12 hours followed by an equal period of subsidence with coagulation. With slow filters the second period of subsidence could economically be carried on for about 24 hours. It was estimated that on the average a period of 12 hours plain subsidence would reduce the turbidity to about 485 parts per million, silica standard (435 parts suspended matter), and that the 12 hour period of coagulation would reduce the turbidity to below 75 parts, silica standard. It was not thought economical to attempt to reduce the turbidity, previous to filtration, below 50 parts, silica standard. The average amount of coagulant required for a turbidity of 485 parts in the settled water would be 4.5 grains sulfate of alumina per gallon. The amount of wash water was estimated at 4 per cent. The most economical size of sand to use was found to be from .30 to .40 mm. effective size with a uniformity coefficient of not more than 1.5. A finer sand would lead to too rapid clogging of the bed, while a coarser sand would permit the passage of coagulated material before the maximum desirable head of about 10 to 12 feet was utilized. A depth of sand of 2.5 feet and a rate of filtration of 125,000,000 gallons per acre per day were considered satisfactory. The cost of filtration, including capital charges, was estimated at $15.00 per million gallons.†

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* The results of the experiments together with much other information on the subject of filtration is contained in Senate Report No. 2380, 56th Cong.; second Session, on "Purification of the Washington Water Supply."

† Report on Water Purification Investigation, 1903.

546. *Tests of Plants in Operation.*—*At Little Falls, N. J.*, the average results of the first five months of operation of a rapid filter plant were as follows: *

RESULTS OF FILTRATION AT LITTLE FALLS, N. J.

Month.	Sulfate of Alumina, Grains per Gallon.	Turbidity.			Color.			Bacteria.			
								Per Cubic Centimeter.			Percentage Removal.
		River Water.	Coag. Basin Effluent.	Filtered Water.	River Water.	Coag. Basin Effluent.	Filtered Water.	River Water.	Coag. Basin Effluent.	Filtered Water.	
1902.											
Sept. . .	0.74	10	6	3	31	20	11	5400	3900	190	96.5
Oct. . .	1.59	6	5	1	52	31	7	3800	650	90	97.6
Nov. . .	1.63	5	4	2	45	28	7	3500	1100	60	98.3
Dec. . .	1.70	7	4	1	44	24	5	5800	1800	50	99.1
1903.											
Jan. . .	0.84	6	5	0	31	21	5	4000	1700	110	97.2

The river water is badly polluted but is usually low in turbidity. The average period of coagulation was about three hours.

At Moline, Illinois, the Mississippi river water is purified by means of rapid filtration, with coagulation for about 2 hours using iron and lime as a coagulant. Average results obtained in a test run of 19 days were as follows: †

	Bacteria per c.c.
River water	10140.
Settled water	610.
Filtered water	31.
Percentage reduction	99.7

	Color, parts per million.
River water	77.
Filtered water	17.

Iron sulfate used, gr. per gal... 1.38
Lime used, gr. per gal........... .95

The maximum number of bacteria on any day was 70. The turbidity of the raw water averaged 163 parts per million.

At the Baisley's and Springfield Filter Plants, two small plants of the Brooklyn Water-works, the following results represent daily

* Trans. Am. Soc C. E., 1903, L. p. 438.
† *Eng. Record*, 1907, LV. p. 708.

analyses for six weeks, and are typical of the work done at these plants:

	Baisley's.	Springfield.
Turbidity of raw water	14.9	6.3
" " filtered water	.2	.41
Color of raw water	31.0	17.0
" " filtered water	3.0	1.0
Iron in raw water	1.14	.39
" " filtered water	.03	.09
Bacteria, when No. in raw water exceeded 2500.		
Raw water	4044.	11,251.
Filtered water	62.	114.
Percentage reduction	98.5	98.6
Bacteria, when No. in raw water was less than 2500.		
Raw water	1089.	1133.
Filtered water	23.	22.
Percentage reduction	97.9	98.1

547. *Summary.* — The experiments here described, and the results of operation, indicate that when rapid filters are properly operated turbidity can be practically all removed, a large percentage of color, and a considerable portion of dissolved organic matter. Bacterial results are also in general as satisfactory as those obtained by slow sand filters.

To obtain uniformly good results with economy requires very careful operation. The coagulant must be closely regulated to correspond with the quality of the water, — in the case of waters low in alkalinity this is particularly necessary. The efficiency depends so entirely upon the control of these matters that the operation of a rapid filter involves greater care on the part of the attendants than that of a slow filter. It is fully as important in this case also that the whole plant should be under control of bacteriological tests, regularly and frequently made. Many points of operation, such as period between washings, wasting of water after washing, exact amount of coagulant required, can be learned only after experience in the light of such analysis.

Considering the economic advantages of rapid filters, it may be said that they are especially adapted to those cases where the cost of land is high, where the water is so turbid as to require large settling reservoirs or the use of a coagulant, and in small plants where the unit for slow filters would be very small. They are also well adapted for the rapid removal of iron from ground-waters, or of the precipitate in softening plants. (See Chapter XXIII.)

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* *Eng. Record*, 1905, LII. p. 238.

548. **General Arrangement of a Rapid Filter Plant.** — In a complete rapid filter plant the essential elements are: (1) the coagulating and settling basin and appliances, (2) the filters, and (3) the clear-water reservoir. In addition to these there may be preliminary settling reservoirs in the case of a water carrying large quantities of sediment. Such reservoirs would usually be constructed quite separate from the filter plant, and, as regards details, need not be considered here. The coagulating basin and the clear water reservoir may likewise be arranged independently of the filters, but usually one or both are built, with the filters, into a single structure forming the purification plant. Inasmuch as the coagulating basin constitutes a necessary and vital part of a rapid filter plant, and requires as close attention as the filters themselves, it is especially important that the appliances for operating the basins and the filters be under the same roof and conveniently arranged for operating purposes. The clear-water reservoir, requiring little attention, may be located at any convenient point near at hand.

The best arrangement of parts will depend much upon local conditions. A convenient arrangement of filter units, especially if rectangular in form, is similar to that for large, slow filters; that is, to place them in two or more rows side by side, with the necessary piping, valves, etc., in a gallery between the rows. The coagulating basin may be conveniently located adjoining the filters, and the clear-water reservoir separately, or, as is quite common, immediately underneath the filters. The introduction of reinforced concrete makes this arrangement economical and satisfactory.

Fig. 138a illustrates the arrangement of filters and coagulating basin at Youngstown, Ohio. The whole is under roof and represents a convenient and compact plan. The clear-water reservoir is located at some distance from the filter plant. The filter units are 14′ 6″ × 21′ in size at the sand level, and between the rows are the pipe gallery and operating platforms. Details of the filter unit and pipe system are shown in Fig. 138b.

Fig. 138c illustrates the general plan of the purification plant at Watertown, N.Y., and Figs. 138d and 138e the details of the filter plant. The coagulating basin and the pure-water reservoir are concrete, vaulted reservoirs located near at hand, but the solution tanks and appliances are located in the filter-house. The filter-house roof is extended to cover the operating platform between the filters and a portion of the filters themselves, the remaining portion being covered with a reinforced concrete cover and earth filling.

Fig. 138f shows the filter plant at Columbus, O. The filters are

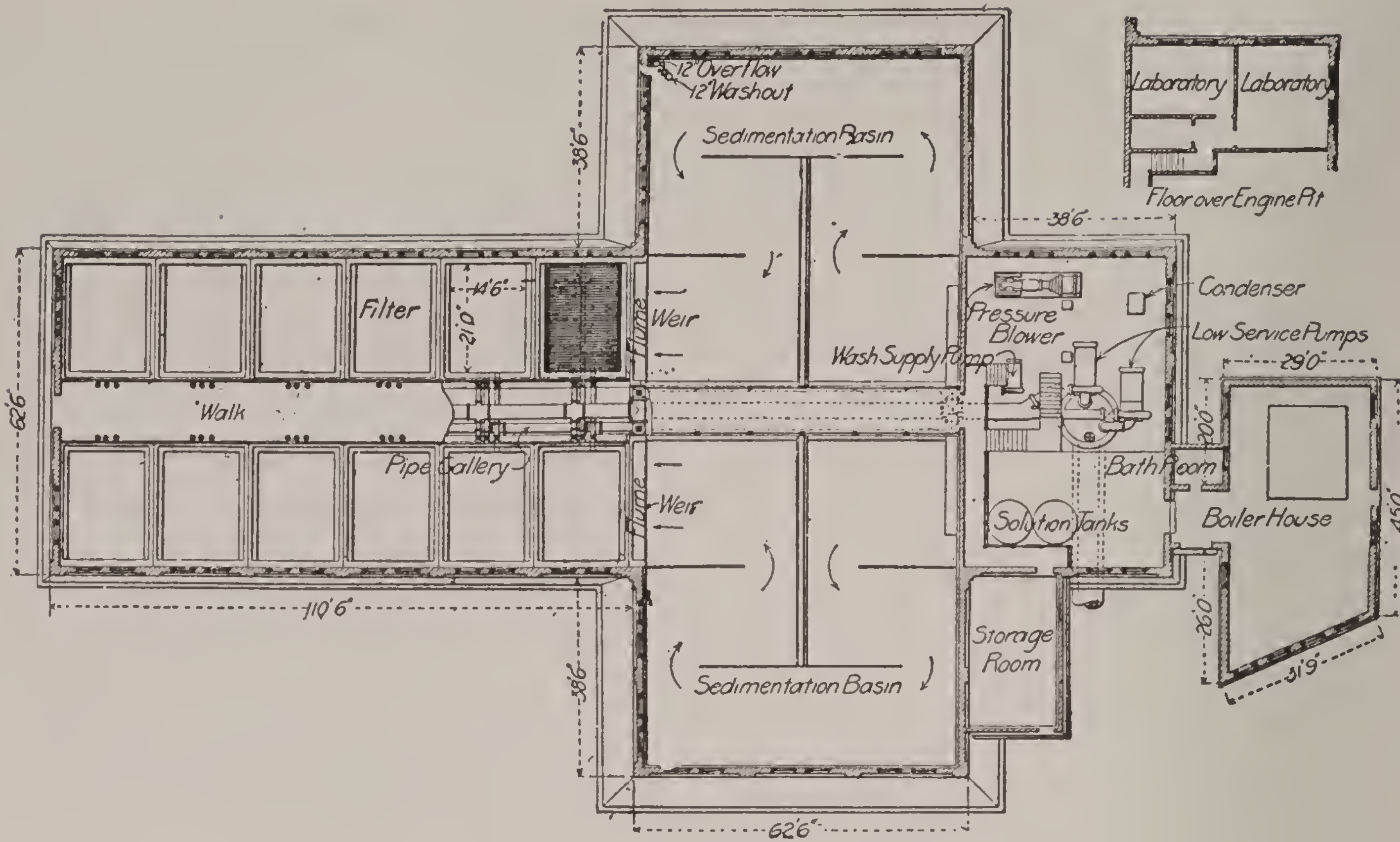


FIG. 138a. FILTER PLANT, YOUNGSTON, O.
(From *Engineering Record*, vol. LII.)

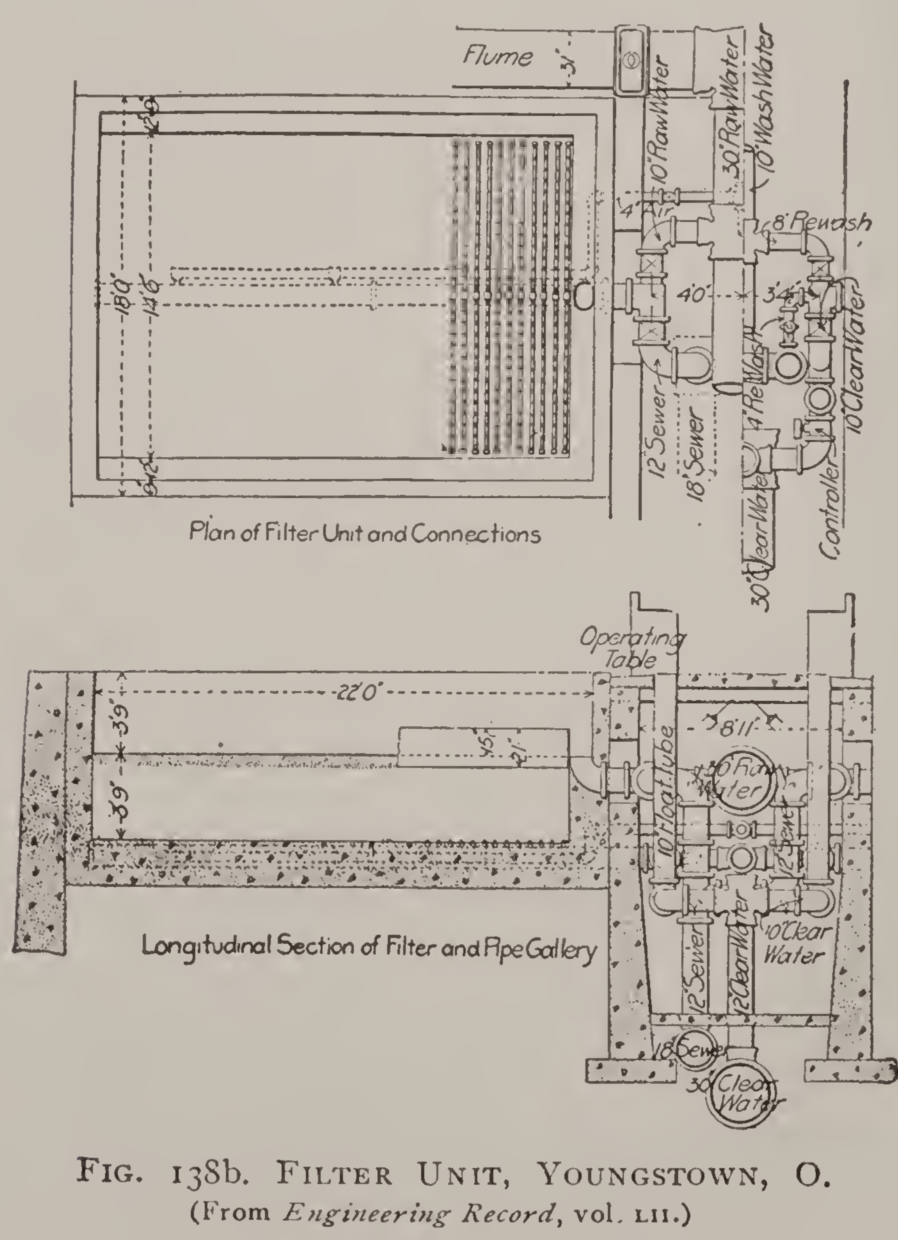


FIG. 138b. FILTER UNIT, YOUNGSTOWN, O.
(From *Engineering Record*, vol. LII.)

relatively large here, each tank unit being 11′ 8″ × 50′ 8″ in dimension. The general arrangement of filters and piping is the same as in the other plants illustrated, the filters being mostly covered with reinforced concrete. The filters are used primarily as a part of a softening plant.

In the large plant at Cincinnati (Fig. 138g) the filters are arranged in a manner similar to that at Columbus, but the filter-house extends entirely over the filters, an arrangement possessing some advantages in

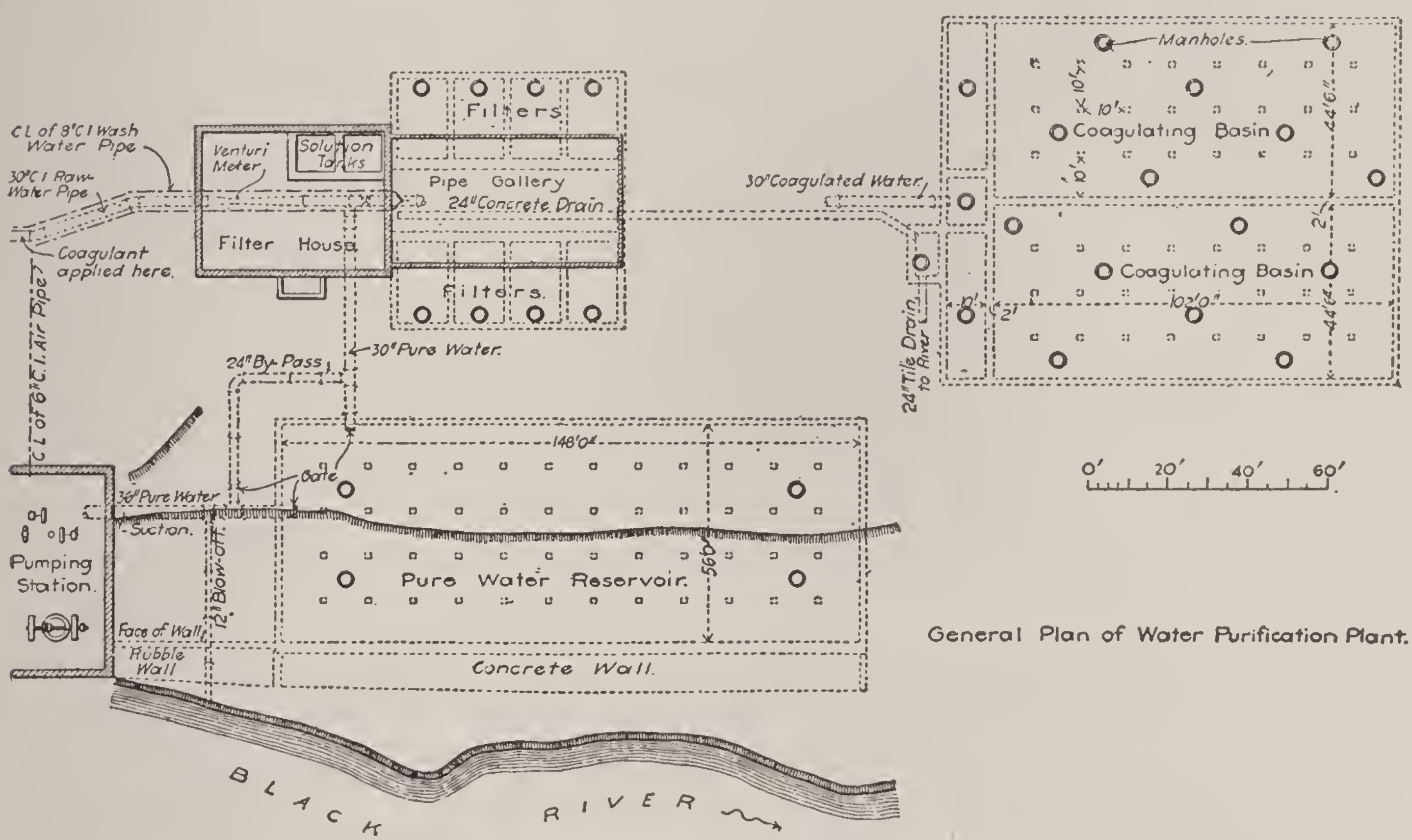


FIG. 138c. FILTER PLANT, WATERTOWN, N. Y.
(From *Engineering Record*, vol. XLIX.)

regard to inspection and control. The clear-water reservoir is underneath the filters in both these plants.

549. **Details of Construction and Operation.** — The most important details of construction include: (*a*) the sand bed; (*b*) the strainer system and collecting pipes; (*c*) the agitating system; (*d*) the wash-water appliances; (*e*) means for controlling the rate of filtration; (*f*) the coagulating system, including means for preparing solutions and regulating their application, and the arrangement of coagulating basins; (*g*) arrangement of piping and valves; and (*h*) various other devices for operating the plant.

(*a*) *The Sand Bed.* — Experience has shown that the most satisfactory sand is one of quite uniform grain and of an effective size of from .3 to .4 mm., the best size depending somewhat upon the character of the water to be treated. A uniformity coefficient of not more than

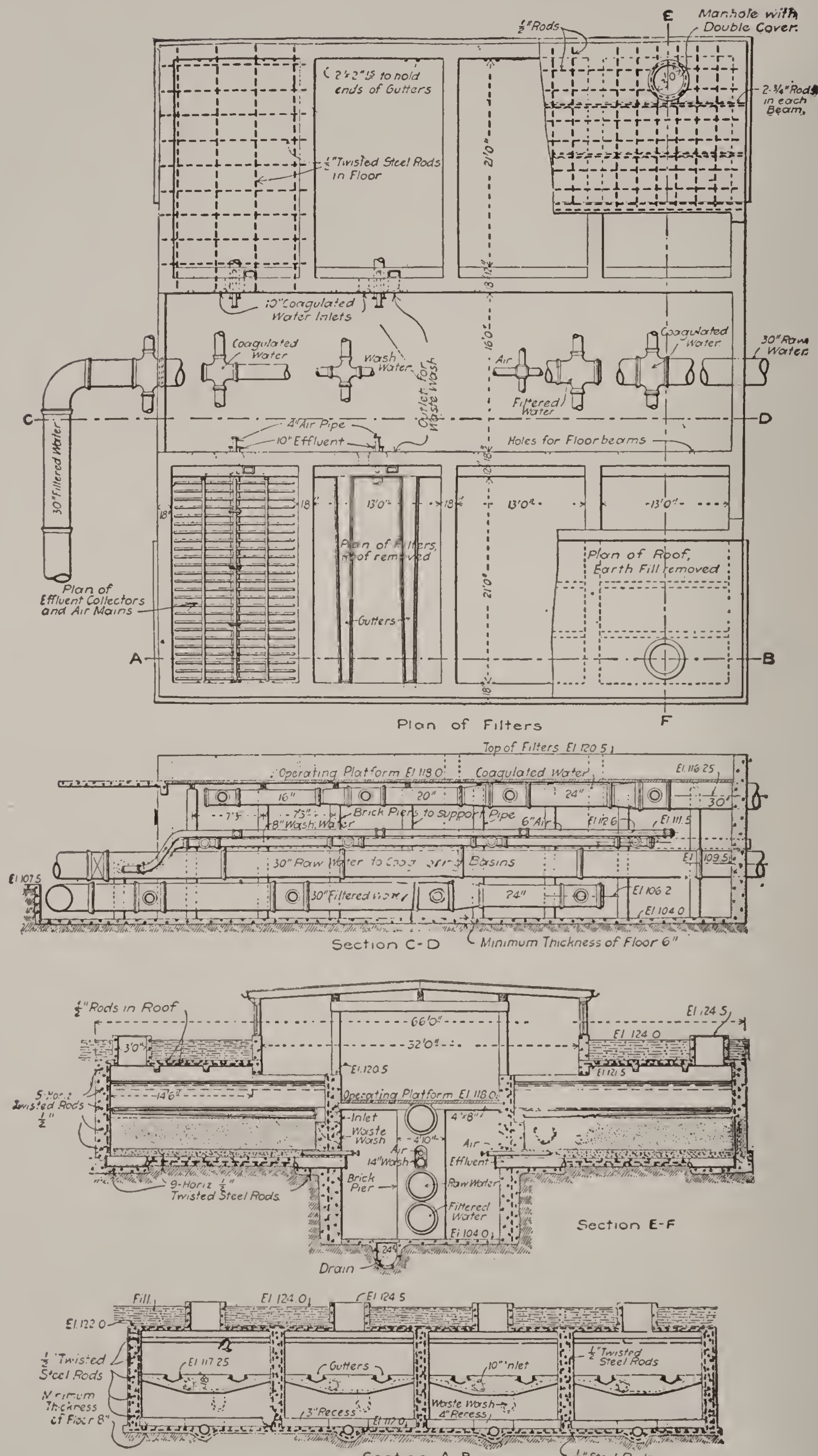


FIG. 138d. FILTER PLANT, WATERTOWN, N. Y.
(From *Engineering Record*, vol. XLIX.)

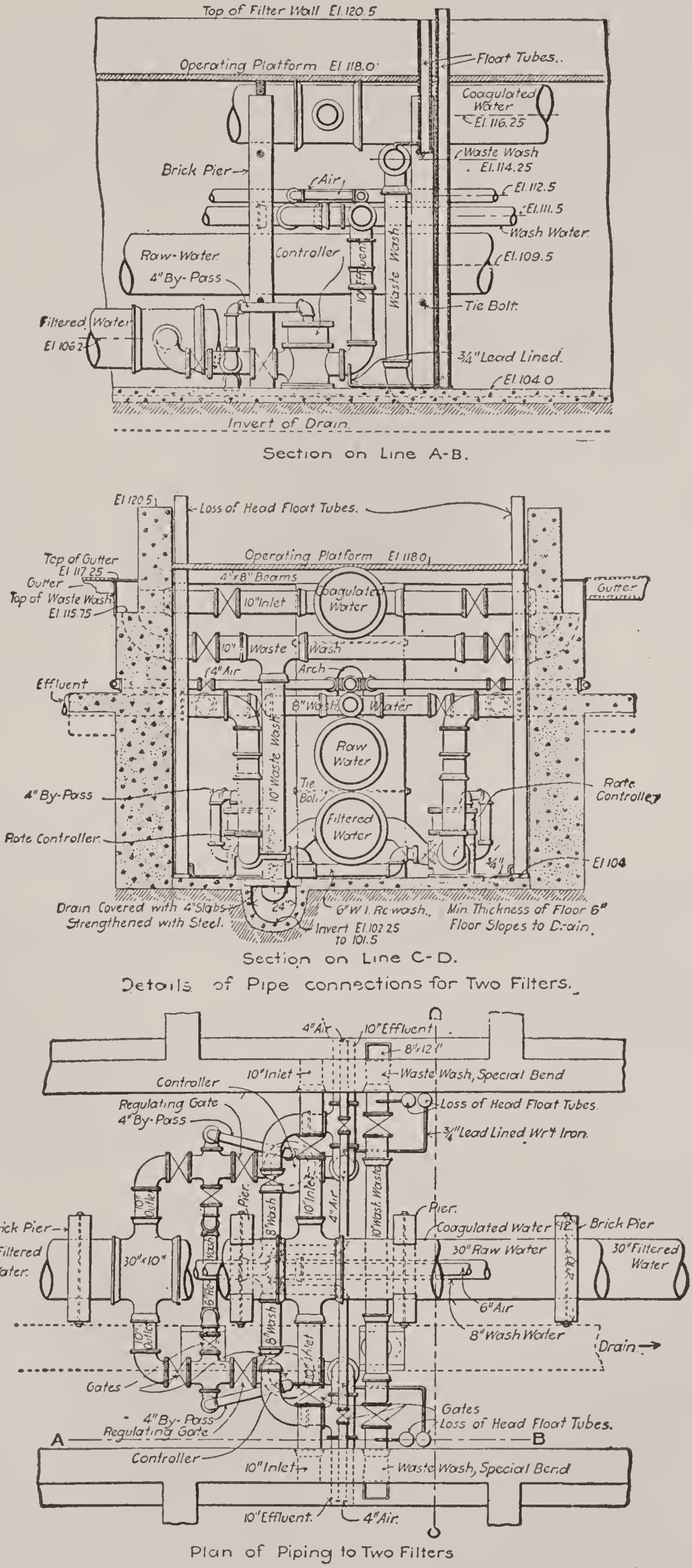


FIG. 138e. DETAILS OF FILTER, WATERTOWN, N. Y.
(From *Engineering Record*, vol. XLIX.)

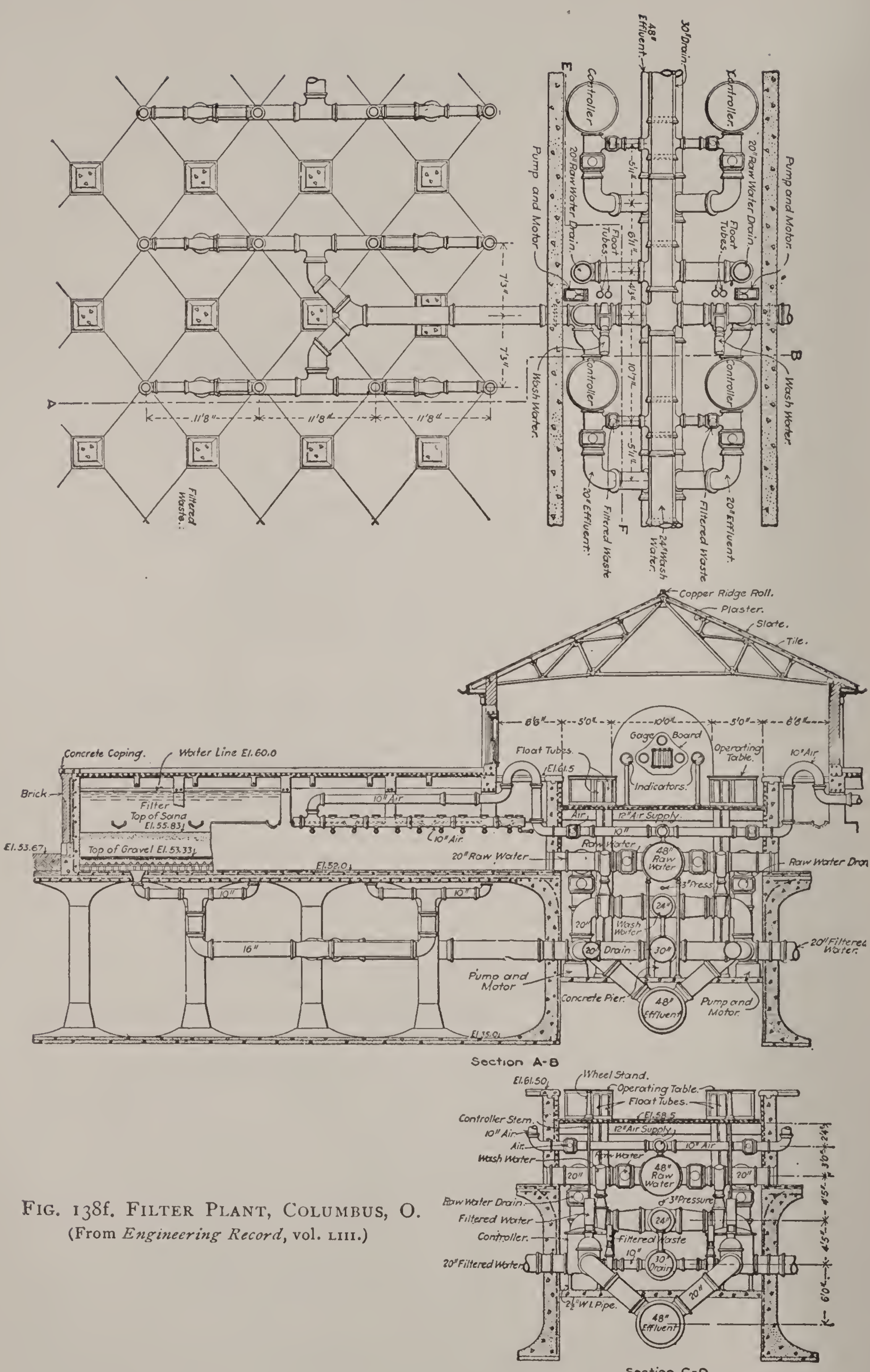


FIG. 138f. FILTER PLANT, COLUMBUS, O.
(From *Engineering Record*, vol. LIII.)

1.5 is desirable. A sand of low uniformity is undesirable as it tends to stratification in washing, and the finer particles are likely to be carried away in the process. Such a sand also offers greater resistance to the passage of the water. A fine sand also clogs more quickly than a coarse sand, thus increasing the cost of operation. On the other hand, a coarse

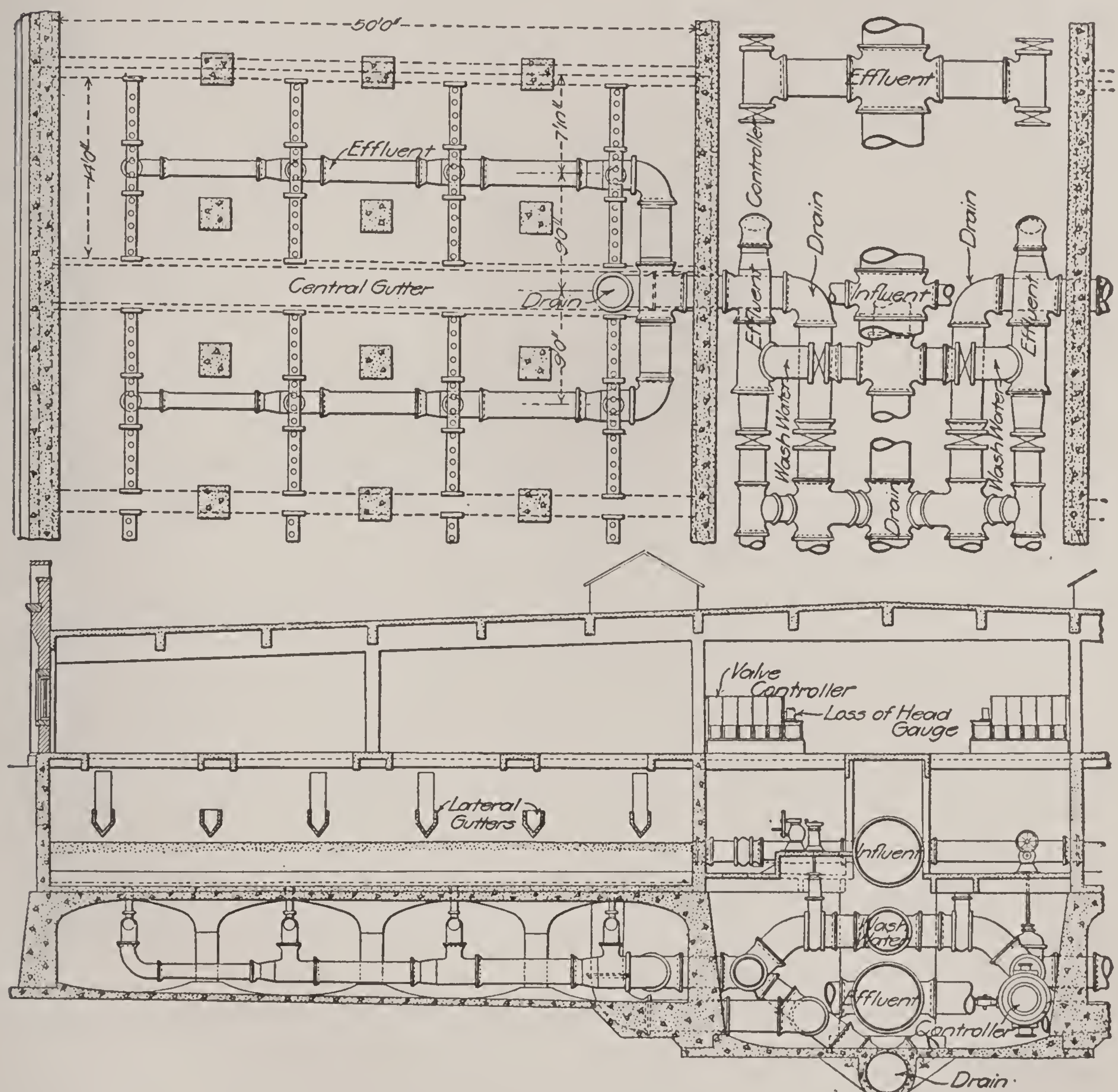


FIG. 138g. FILTER PLANT, CINCINNATI, O.
(From *Engineering Record*, vol. LV.)

sand allows the sediment to penetrate to a greater depth, and if too coarse it may need to be cleaned before the maximum available head has been utilized. A depth of 30 to 36 inches is usually employed in the more recent plants. The sand bed is supported on a layer of fine gravel, 6 to 8 inches or more in thickness, which permits the perforations in the strainers to be of fairly large size. This gravel should be carefully screened and of as large a size as practicable without allowing the superimposed sand to penetrate into the pore spaces. Usually two or three grades are employed, the upper one being about .05 to .1 inch in size, and the lower one about $\frac{1}{4}$ to $\frac{1}{2}$ inch. The gravel should be free from fine material, and as uniform as possible in order to avoid being disturbed in the washing process. Crushed and screened quartz is often used for the sand and gravel, but natural materials well screened are equally satisfactory.

(*b*). *The Strainer System and the Collecting Pipes.*—The design of the strainer and collecting system is a matter of greater difficulty than in the case of the slow sand filter. As in that type, the collecting system must, first of all, be sufficiently extensive to cause the total loss of head to be nearly uniform over the entire area. If this were the only requirement it could readily be met by the use of coarse gravel and drain pipes with large openings. The strainer system, however, must serve also to distribute the wash water uniformly into the sand bed. To accomplish this there must be a considerable resistance to flow through the strainer, as compared to that through the pipe system, so that, as the water forces its way through the sand, a considerable reduction of resistance in the sand at one point will not materially change the pressure at other points. This requires the strainer openings to be small, numerous, and well distributed, and the pipe system to be relatively large and arranged so as to give practically equal pressures at all points. The collecting pipes must be designed with reference especially to the amount of wash water required and must be arranged in units of not too large size. The unit of area served by one collecting main is commonly made from 10 to 15 feet wide by 15 to 20 feet long. In large plants it is convenient to group together from two to four such units to serve a single tank, the size of tank depending much upon the total size of plant.

Figs. 137, 138, and 138b illustrate common arrangements of collecting pipes and strainers. In each case the effluent pipe connects with a large central cast-iron collector, or "mainfold," into which are screwed lateral collecting pipes placed about 6 inches apart. Into these are screwed brass strainers which are also spaced about 6 inches apart. These strainers are perforated with numerous small holes. In the earlier practice the holes were very small, but in later plants they are made larger, $\frac{1}{16}$ inch to $\frac{3}{32}$ inch being a common size. Large holes are less apt to clog up, and, with the use of gravel beneath the sand, are much more satisfactory.

The general arrangement of collecting pipes in the Watertown plant

(Fig. 138d) is the same, but the strainers are here made of small brass tubes, attached by means of T's to the laterals of 2-in. wrought-iron

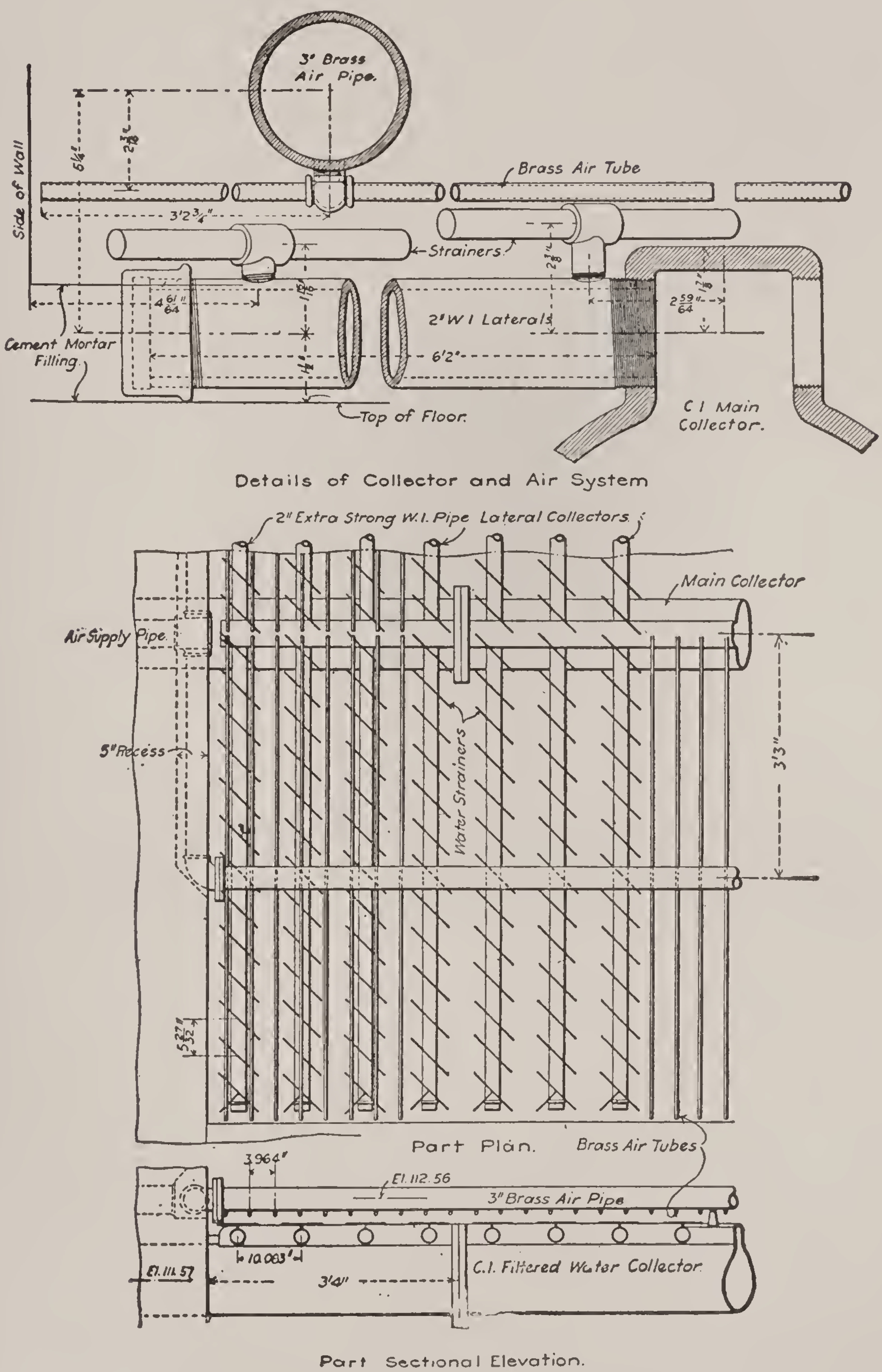


FIG. 138h. STRAINER SYSTEM, WATERTOWN FILTERS.
(From *Engineering Record*, vol. XLIX.)

pipe. Details of this strainer system are shown in Fig. 138h. The laterals are about 10 inches apart and the strainers 6 inches.

In some of the latest plants the laterals consist mainly or wholly of channels in the concrete floor, and the strainers are made of brass plates set directly into this floor. Fig. 138i illustrates such a system as employed in the Columbus plant. The strainer itself is a brass plate perforated with $\frac{1}{16}$-in. holes. These plates are set into slabs of reinforced concrete on $8\frac{3}{4}$-in. centers, which in turn are set into the concrete floor $8\frac{3}{4}$ inches apart. Below these slabs are lateral channels, as shown in the section. Between the rows of strainers triangular concrete ridges are constructed in the floor to a height of 3 inches above the strainer level.

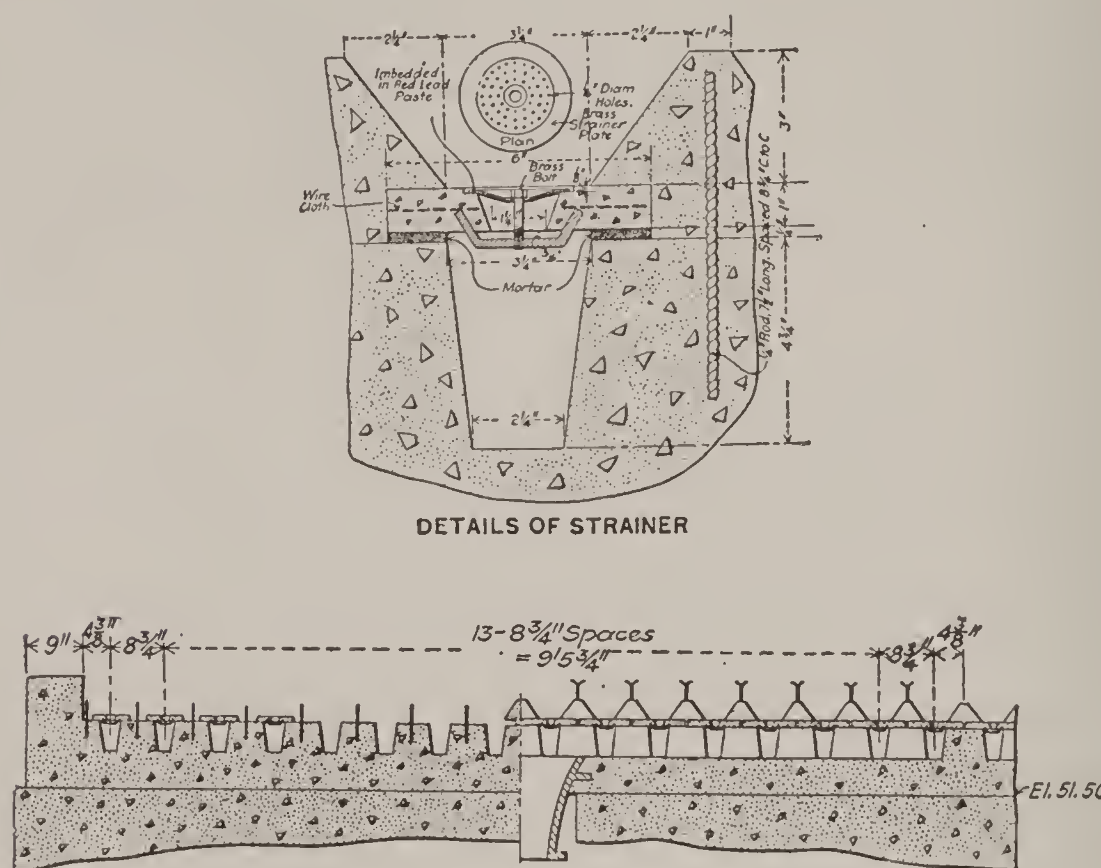


FIG. 138i. STRAINER SYSTEM, COLUMBUS FILTERS.
(From *Engineering Record*, vol. LIII.)

The furrows between are filled with gravel, which also extends a few inches above the ridges. This arrangement aids in securing a good distribution of the wash water. The collecting unit is 11′ 8″ square, at the center of which is a cast-iron connection with the main effluent pipe. Four such units make up one tank unit, and two such tank units are operated together in pairs as a single filter unit. Fig. 138f shows the arrangement of collecting pipes leading from the collecting unit, and it will be noted that the length of the pipe, up to its connection to the filter, is the same for all units, thus causing a uniform resistance to flow in the pipe system.

Fig. 138j shows a somewhat similar detail adopted for the Cincinnati plant. The laterals consist of concrete channels 12 inches apart, with strainers of long, brass plates, perforated with sixty-four $\frac{3}{32}$-in.

holes per lineal foot. From each of these lateral channels, connections are made by means of $3\frac{1}{2}$-in. cast-iron risers to the main collector located beneath the floor. The furrows in the concrete floor are 8 inches deep and contain all the gravel. Above this gravel is placed a wire screen of No. 20 brass wire, having 10 meshes per inch. The purpose of this screen is to hold the gravel in place during washing, the water-pressure employed here being especially high, as no other method of agitation is used. The units of the collecting system are $12\frac{1}{2}' \times 14'$ and four such units make up one tank unit. The filter unit comprises

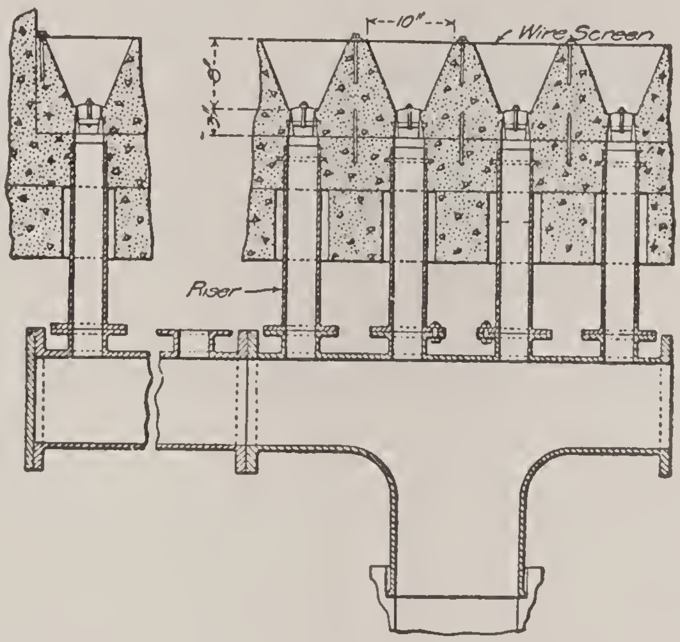


FIG. 138j. STRAINER DETAIL, CINCINNATI FILTERS.
(From *Engineering Record*, vol. LV.)

two tanks as in the Columbus plant, thus giving a filter unit of $28' \times 50'$ net area. In the New Orleans plant the same general system has been adopted, no air being used in cleaning.

(*c*) *The Agitating System.* — Two general methods of agitating the sand are in use, the mechanical agitator and agitation by compressed air. The usual form of the mechanical agitator is illustrated in Fig. 137. It consists of deep rakes which are moved through the sand during the washing process. After the operation is complete the rakes are lifted out of the sand. While the circular form of tank is better adapted to the use of this form of agitator it has been used to some extent with rectangular tanks, but compressed air is much more convenient in that case.

In the use of compressed air, the air, under a pressure of 3 to 5 lbs., is forced through the sand bed, either through the strainer system itself or through a separate pipe system located in the gravel just above the strainers. In the air distributing system, as in the water system, it is necessary to use such an area of openings as to admit the desired amount of air and at the same time to offer a considerable resistance to exit as compared to the resistance in the pipe system. These requirements make it necessary to use a much smaller total area of openings than in the water system, an area of .02 to .03 sq. in. per sq. ft. of filter being common. Where the air is forced through the strainer system an ingenious arrangement of the strainer traps off the main opening so that the opening for air is reduced to the desired amount. In this case the air pipes are connected at intervals with the collecting main, through which the air passes to the trapped strainers. In the operation of washing, air and water are used alternately.

Fig. 138g shows the details of the air system in the Watertown plant. The pipes consist of small slotted brass tubes spaced about 4 inches apart. At Columbus the air is conveyed through separate brass air tubes placed 2 feet 11 inches apart and supported about 6 inches above the strainers.

In some plants water alone is successfully used without other means of agitation, notable examples being the large plants at Cincinnati and New Orleans. In these plants no provision is made for special agitation, but the wash water is used under relatively high pressure. Experience in some plants indicates that the results gradually deteriorate if water alone is used, although at first there appears to be no difference.

(*d*) *Wash-water Appliances.* — The wash-water appliances consist of the pipe connections, arranged so that the flow may be reversed in direction, the agitating system, and the means for taking off the dirty water from above the filter. The wash water is supplied under a pressure of about 10 to 15 lbs. either by means of suitable pumps, or from the high pressure system through reducing valves, the former being generally the more economical method. It is admitted to the effluent pipe of the filter through suitable pipe connections and controlling valves. The dirty water from the filter passes through troughs or gutters built across the bed or at the margin, and thence is conducted through pipes to the drain. In order readily to carry off the wash water the gutters need to be spaced so that the lateral movement of the water is relatively small, not more than 3 to 4 feet. These gutters are conveniently built of reinforced concrete. Various arrangements are shown in the illustrations. In Fig. 137 a wooden gutter extends around the tank along the inside surface; in Fig. 138 two gutters are constructed along the outside walls and a central trough of steel is also provided. In Fig. 138b a somewhat similar arrangement is used. In Fig. 138d two gutters of steel are employed, while in Fig. 138f reinforced concrete gutters are used, spaced about $7\frac{1}{2}$ feet apart and leading to a central gutter between adjoining beds. The gutters are placed about 1 foot above the surface of the sand bed. Generally the unfiltered water is admitted through the same pipe connection that serves as the outlet for the wash water, the gutters thus serving as weirs to distribute the raw water until the filter is partially filled. After washing a filter it is sometimes desirable to waste the effluent for a time. Provision should always be made for this by constructing suitable connections from the effluent pipe to the drain. The time required for washing is only 10 to 12 minutes, and the amount of wash water required is usually from 4 to 5 per cent of the total amount filtered.

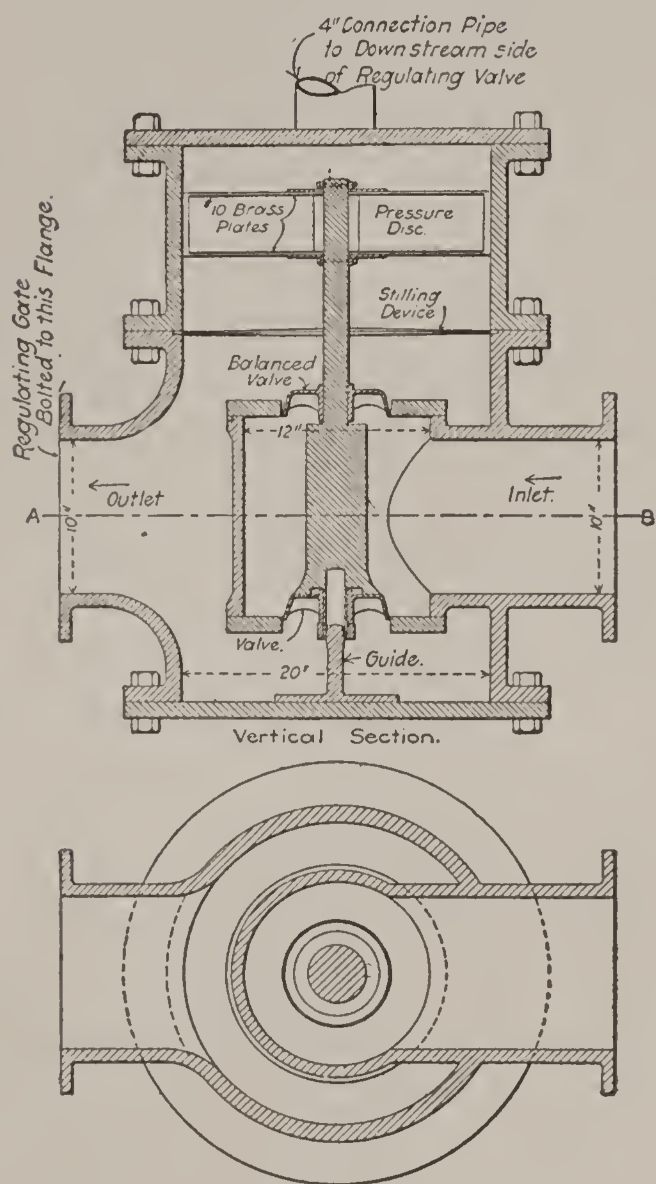
(*e*) *Head Employed and Manner of Controlling Rate of Filtration.* — At the ordinary rate of 100 to 125 million gallons per acre per day, the minimum frictional resistance is usually from 2 to 3 feet. As in the slow sand filter, arrangements must be made whereby, as the filters become clogged, this loss of head may be increased up to the maximum desirable amount, and so varied as to maintain a uniform rate of filtration. The maximum loss of head is generally made about 10 or 12 feet. A greater maximum tends to increase the cost of plant and the penetration of suspended matter into the filter, while a lesser maximum tends to increase the frequency and cost of cleaning and the proportion of area out of service. With this maximum available head the period of service will usually range from 6 to 12 hours.

The head is controlled in a manner similar to that used with slow filters, that is, by maintaining a constant level of water on the filters and varying the pressure head in the effluent pipe by some means more or less automatic. Generally automatic controllers are used, a controller being inserted in the effluent pipe of each filter unit. A form used by the Jewell Filter Company is one designed by Mr. E. B. Weston, and known by his name. The discharge is controlled by two butterfly valves which are regulated by a float so as to give a constant head over an annular opening through which the water is discharged. The annular opening may be varied in size by the use of various sized central disks. The controller is enclosed, but the discharge side is not under pressure.* A design used at the Watertown plant is illustrated in Fig. 138k. Here a regulating gate or valve acts as an orifice plate, thus enabling the rate to be more readily varied. The piston, or pressure disk, is under pressure on its upper side from the down-stream side of the regulating valve and on its lower side from the up-stream side

* See *Eng. Record*, Nov. 25, 1899.

of the valve, vibration being checked by transmitting the pressure through small openings in a stilling disk. A type of controller similar to the automatic weir and float used at Pittsburg is employed at Hackensack, N.Y.* Various other devices are used for this purpose, but those described represent the most common types.

The means employed to maintain a constant level of water on the filter is the usual balanced valve placed on the entrance pipe and regulated by a float. A butterfly valve is also often employed for this purpose. Generally the level of water in the entire system of filters is maintained at a uniform elevation, and equal to that in the coagulating basin, so that the regulating valves are placed in the basin only. The most suitable arrangement depends upon local conditions.



Section A-B.
Sections of Part of Controller.
FIG. 138k. CONTROLLER, WATERTOWN FILTERS.
(From *Engineering Record*, vol. XLIX.)

(*f*) *The Coagulating System.* — The various parts of the coagulating system are the same as described in Chapter XX. The period of coagulation is important as the success of the plant depends much upon this feature. In the early plants this was very inadequate, being but a few minutes. Generally a period of 3 to 6 hours is now provided, the most advantageous period depending upon the character of the water. Too perfect sedimentation is undesirable as it removes too fully the coagulum upon which the efficiency of the filter depends. The coagulating basin of the Youngstown plant is well shown in Fig. 138a. Here the effluent passes over a broad weir into the pipes leading to the filters.

(*g*) *Arrangement of Piping System.* — The pipe system includes the following: Inlet pipes for the raw water, outlet pipes for effluent, pipes for wash water, pipes for wasting dirty water to the drain, pipes for wasting effluent, and, generally, air pipes. The large mains of these various systems are generally placed in a pipe gallery between rows of

* *Eng. Record*, 1904, L. p. 591.

filters and branches taken off at each filter as shown in several of the illustrations. In the large units at Columbus and Cincinnati the filter unit is divided in the center by a horizontal gutter into which the raw water is discharged, and which also receives the dirty water in washing. The air pipe connections are also made by means of branches taken off in the central channel (see Fig. 138f). The branch pipes from the raw-water main and from the waste-water main usually connect to the same

FIG. 138l. OPERATING ROOM, YOUNGSTOWN FILTERS.
(From *Engineering Record*, vol. LII.)

point in the filter; likewise the branches from the effluent and from the wash-water mains. An additional cross connection is placed between the effluent pipe and the waste pipe to permit of wasting the effluent.

(*h*) *Other Devices Used in Operating the Plant.* — Besides the features already described, other details which require careful attention are the various devices used in the operation of the plant. For the operation of the valves hydraulic pressure is generally employed, the pressure pipes being all operated from tables on the operating platform. Fig. 138l shows the operating room of a modern plant. Loss-of-head gauges and water-level gauges should be provided, as in a slow filter plant, also convenient means for sampling. Proper laboratory facilities for the study and control of the operation are important.

550. **Cost.** — The cost of a rapid filter plant under ordinary conditions will range from $8,000 to $12,000 per million gallons capacity, for filters, coagulating basin, clear-water well, and auxiliary pumping apparatus. The cost of operation is largely dependent upon the amount of coagulant used. The cost of sulphate of alumina is about $20 to $25 per ton, equivalent to $1.40 to $1.75 per million gallons for each grain per gallon used. Sulphate of iron costs about $10 to $12 per ton, and lime $4 to $6. Compared to the cost of sand filters the first cost will usually be less, but if much coagulant is used the cost of operation will be more. Which system is the more economical thus depends upon the character of the water treated and other local conditions. Under ordinary conditions the cost of operation per million gallons will range from $4 to $6; and, including capital charges, interest and depreciation, the total cost will range from $10 to $12 per million gallons.

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CHAPTER XXIII.

MISCELLANEOUS PURIFICATION PROCESSES.

554. **Special Forms of Filters.** — Besides the two principal types of sand filters discussed in the preceding chapters, various special forms of sand filters, and filters composed of other materials, have been employed to a limited extent. Two types of these are of considerable importance and will be briefly described. These are the artificial, porous stone filter, represented chiefly by the Fischer system in municipal plants and the porcelain filters for household use, and the rapid, coarse filter employed for preliminary treatment such as the Maignen "scrubber." The use of asbestos for a filter membrane in domestic filters, and recently as an aid in sand filtration at South Bethlehem, should also be noted.*

555. *The Fischer Tile Filter.* — This form of filter, invented by Director Fischer of the water-works of Worms, Germany, and used at that place, consists of a series of hollow cells composed of a mixture of sand and glass fused together. The cells are about 3 feet square and 8 inches thick, with a hollow space of about 1 inch. They are set up on edge in a reservoir containing the water to be filtered, and connected together in groups so that the water filters through into the interior space and thence passes out through suitable pipes. The filters are cleaned by reversing the flow, and by washing with a hose-stream. They can also be sterilized by steam. The rate of filtration practiced at Worms is about 3 million gallons per acre per day, but the actual space occupied by the filters is only about one-fourth that of an ordinary filter. The results obtained compare very favorably with those from sand filters. The system is in use in several places in Europe. Several cells were experimented upon at Pittsburg, but they were not found very well suited for so turbid a water, the preliminary treatment there required accomplishing nearly all the purification. Other forms of stone filters have been less extensively employed.

556. *The Maignen "Scrubber."* — This is a form of preliminary filter, composed of layers of coarse gravel and slag covered with

* *Eng. Record*, 1905, LII. p. 61.

a layer of compressed sponge. The water enters at the bottom and flows upwards, the rate being ordinarily about 60,000,000 gallons per acre per day. Generally about 60 per cent of the turbidity and 75 to 80 per cent of the bacteria are removed. The action is partly sedimentation and partly filtration. It is estimated at Philadelphia that preliminary treatment with this "scrubber," will enable the sand filters

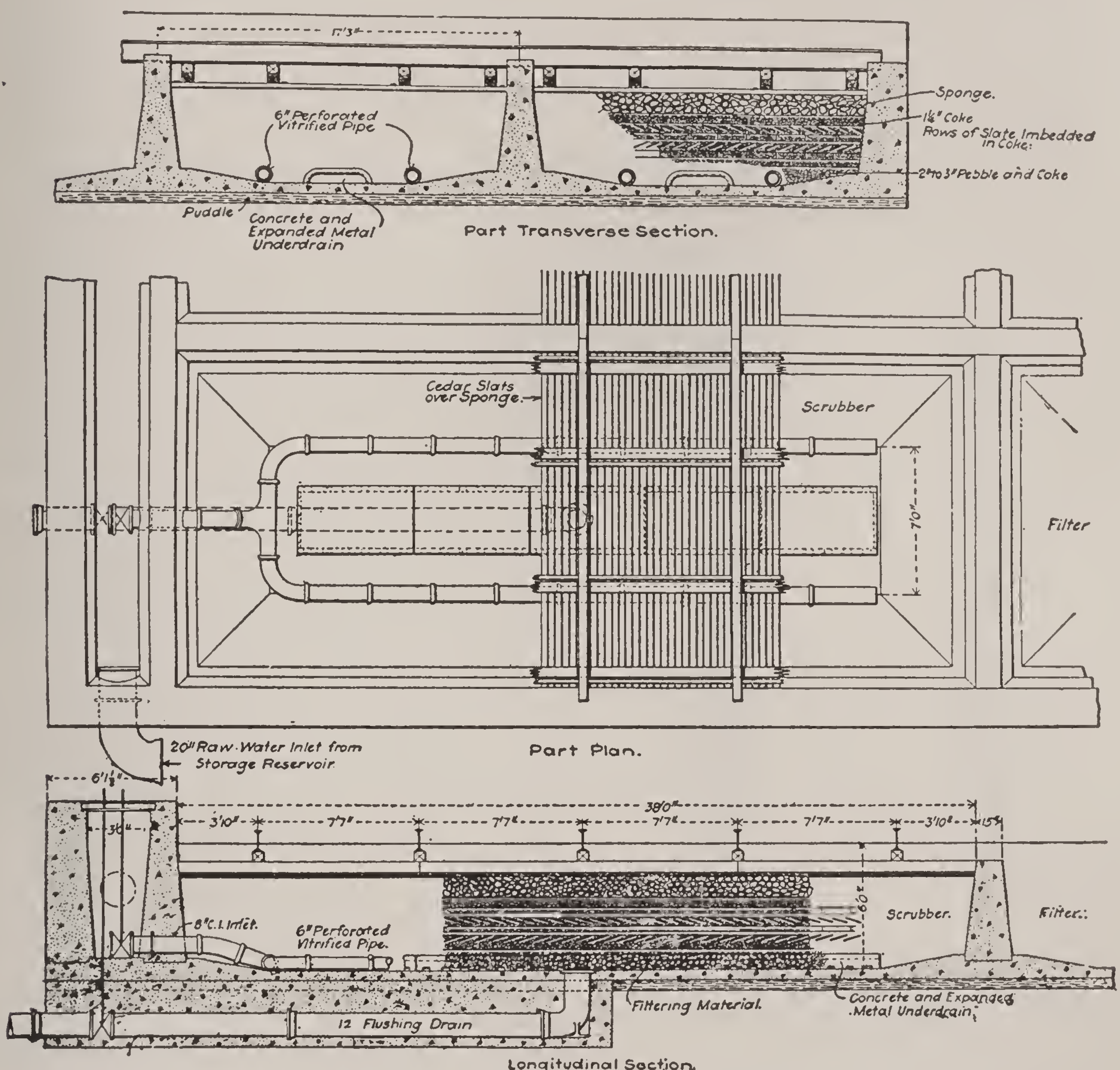
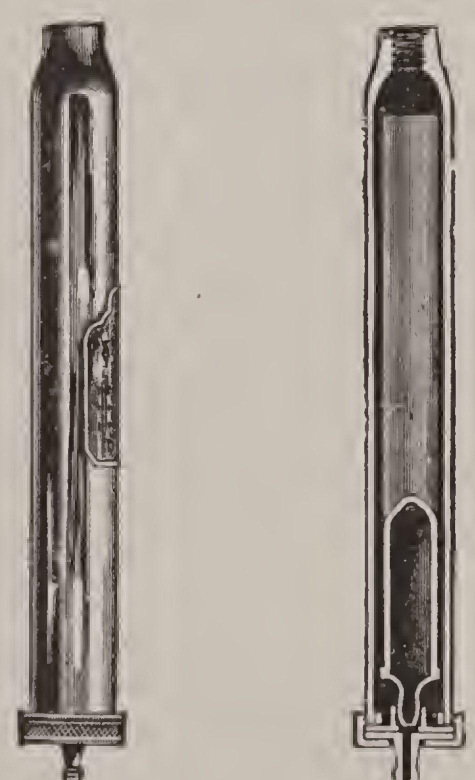


FIG. 139. MAIGNEN "SCRUBBER."
(From *Engineering Record*, vol. LII.)

to operate satisfactorily at about 6,000,000 gallons per acre per day with a considerable saving in cost over a larger plant of slow sand filters. Scrubbers of the Maignen type are also used at South Bethlehem, Pa. Here they are composed of the following layers. (Fig 139): The lower foot is made up partly of 3-in. gravel and partly of 3-in. coke. Above this are four layers of 1¼-in. coke of a total thickness of 2 feet. In each layer are placed regular rows of slates inclined about 30°, but in opposite directions in the different rows. Above these layers is another layer of $1\frac{1}{4}$-in. coke and finally a layer of about 8 inches of compressed sponge. The slate layers are intended as deflectors to aid sedimentation. The scrubber is cleaned by reversing the flow of water. The sponge is also removed and washed occasionally and, if necessary, the coke may be treated in the same way. The rate of filtration through the scrubbers is 28,000,000 gallons per acre per day and 7,000,000 through the sand filters. A further increase in the rate of the sand filters to 9,000,000 gallons has been accomplished by adding a membrane of asbestos fibre to the top of the filter.*

557. *Domestic Filters.*—Frequently it is advisable to purify water-supplies for household use. For this purpose a large number of different filters have been devised, but many of these are so inefficient as to be worse than useless; for it not infrequently happens that the possession of a filter lulls the consumer into a state of false security. The best of these filters suitable for household use are those that are made of unglazed porcelain (Pasteur filter), or fine infusorial earth (Berkefeld filter).



a *b*

FIG. 139a. PASTEUR FILTER.
a, view from outside.
b, sectional view.

Filters of this class are comparatively porous, thus permitting a fairly rapid flow. In this respect the Berkefeld is superior, as it filters considerably faster than the Pasteur. Both of these filters deliver a wholly germ-free filtrate when they are first put in service, but unless close attention is given them they sooner or later lose this property. The pore spaces in filters of this class are not smaller than the bacteria that ordinarily abound in the raw water; hence the removal of these organisms is not purely mechanical. There seems to exist a sort of attraction between the bacteria and the particles composing the filter, so that the former are prevented from being forced through the pores. This attractive property varies with different materials. Guinochet states that the pores in the micro-membranes of

* *Eng. Record*, 1905, LII. p. 61.

the asbestos filter made by Breyer are smaller than in the Pasteur filter and yet bacteria pass these quite readily. As additional water is passed through these filters, the pore-spaces become reduced in size, owing to the accumulation of organic or other matter, until finally a living pellicle or membrane is formed on the outer filtering surface. This increases the resistance offered to the passage of the water and consequently diminishes the flow of the effluent. The bacteria that abound in this slimy pellical are not destroyed, and if the temperature is favorable they begin to grow. Under such conditions, the bacteria capable of multiplication force their way through the pores of the filter and so appear in the filtrate. Filters of this class therefore retain their germ-proof qualities for periods that are in a way inversely proportional to the temperature of the water. The lower the temperature of the water, and therefore the slower the development of the contained bacteria, the longer the filtrate will retain its sterile condition. Generally speaking, these filters should be cleaned and sterilized in boiling water or in steam under pressure once a week in order to kill out the germ-life that has found lodgment in the pores. In this way not only is the sterility of the filtrate maintained, but the yield of filtered water is increased. The more rapid rate of filtration in the Berkefeld as compared with the Pasteur filter makes this filter lose its efficiency more rapidly.

It is necessary to test the soundness of these filters before they are installed. This can be done by compressing the air in one just after it has been boiled and then immersing the same in water. In a perfectly sound filter (Pasteur) no bubbles of air should be observed.

Although it is a demonstrated fact that the normal water bacteria will work their way through these filters in the course of a few days to a few weeks, still it is by no means so certain that disease organisms like typhoid and cholera would do so. Experiments which have been made by adding cultures of these organisms to water and then filtering the same have shown that these filters kept back the disease bacteria for several weeks, but that finally they could be detected in the filtrate. When, however, the amount of organic matter added was less, and the conditions therefore simulated more nearly those that would obtain in a polluted water, the typhoid germ failed to appear in the filtrate.*

In times of epidemic disease entire reliance should not be placed in the operation of these filters, as it frequently happens that some of

* Schöfer, *Cent. f. Bakt.*, 1893, XIV. p. 685.

them are defective. An outbreak of 145 cases of cholera in a single regiment of 650 men occurred in 1894 in Lucknow, India, as a result of an imperfect filter in use in the barracks; but in general the use of the best filters has reduced the amount of water-borne disease. This is especially noteworthy in the garrisons of the French army, where the typhoid death-rate has been much lessened. Rideal mentions an instance in the barracks at Melun. In 1889 there were 122 deaths from this disease; after the introduction of the Pasteur filter, the average mortality for the next seven years was only seven.

Filters of this class are not often used for city supplies,* but are admirably adapted for schools and other public institutions.

Other types of household filters, such as those constructed of porous stone, charcoal, or asbestos, have been on the market for many years. Judged from the popular standpoint of purity, which is generally the production of a clear water, many of the filters would be regarded as quite satisfactory, but as a means of removing germ-life they possess for the most part but little merit.†

558. **Aeration.** — Attempts have often been made to purify water of organic matter by aeration. The presence of oxygen is certainly necessary for the action of the nitrifying organism, and experiments of the Massachusetts Board of Health show that artificial aeration greatly increases the rate of purification in the case of sewage filtration. But to add large quantities of oxygen to water that already contains oxygen appears, from experiments by Drown and from analyses of aerated water in various places, to have little or no effect on the organic matter. Experiments on aeration were made by Down ‡ in several ways: (1) by exposing water in bottles to the air of a room; (2) by drawing a current of air through the water; (3) by shaking water in a bottle; and (4) by exposing water to air under a pressure of 60 to 75 pounds per square inch. The results of some of the experiments are given in Table No. 73. The variations shown in the amount of albuminoid ammonia are too small to be significant. Other experiments on very dilute sewage gave about the same results, with the exception that a part of the free ammonia was removed in the same way that any gas can be driven out by aeration. The general results of the experiments are confirmed by analyses made on river-waters at points above and below falls or rapids.

* Several cities in India have, however, installed filter-plants on this system.

† For description of form of domestic sand filter see article by Fletcher; *Eng. News*, 1906, LVI. p. 141.

‡ Mass. Board of Health, 1891, p. 385.

TABLE NO 73.

RESULTS OF EXPERIMENTS ON AERATION OF COCHITUATE WATER.

(Parts per 100,000.)

	Free Ammonia	Albuminoid Ammonia	Nitrogen as Nitrites	Nitrogen as Nitrates
First experiment:				
Original sample	.0014	.0182	.0002	.0275
After standing in open bottle for 48 hours	.0008	.0176	.0005	.0250
After aerating by current of air for 48 hours	.0014	.0170	.0003	.0250
After standing in open bottle for 216½ hours	.0036	.0158	.0002	.0250
After standing 49½ hours and then aerating by a current of air for 167 hours	.0026	.0156	.0002	.0250
Fourth experiment:				
Original sample	.0018	.0140	.0002	.0200
After standing for 72 hours	.0016	.0152	.0002	.0300
After aerating " " "	.0024	.0142	.0002	.0200
After being under pressure of 75 lbs. for 72 hrs.	.0036	.0150	.0002	.0250

Though aeration may effect little or no change in the organic matter present in a water, it does have a very important action in the case of waters from ponds and reservoirs which possess offensive odors or tastes because of certain dissolved gases present. These gases may arise either through the putrefaction of dead organic matter, such as the vegetation left in a reservoir when constructed, or the dead algae and other organisms which may periodically grow in the water, or they may be formed in the growth of certain microscopical organisms. In any case aeration is very effective as it causes the displacement of the objectionable gases by the gases of the atmosphere. Where waters are to be filtered that are deficient in oxygen some method of aeration should be employed. Another use of aeration is the prevention of the growth of algae in small reservoirs by the agitation produced by the process. In the removal of iron from ground-waters aeration also plays an important part as more fully described in Art. 565.

Aeration is accomplished in various ways. It may be done by causing the water to flow over cascades or weirs, or to fall freely from broad areas of perforated plates, or by still other means. The more extensive the aeration required the more thorough must be the exposure to the air. The largest plants designed especially for aeration are probably those of the Spring Valley Water-works of San Francisco. There are three separate plants, all of similar design. The one known as the College Hill plant has a capacity of 8 million gallons per day. The water rises about 20 feet in an upright pipe, is then conducted through two long wooden flumes and distributed from these through holes in the sides, to a series of wooden platforms. These are about 3 feet apart vertically and are made of 1-in. plank 6 inches wide laid $\frac{1}{8}$ inch apart. The result of the aeration appears in this case to be quite marked, according to the report of the Board of Health. The results are, for the three plants, as follows, in parts per 100,000: *

	1	2	3
Albuminoid ammonia:			
Before aeration	.00620	.00756	.00756
After aeration	.00416	.00252	.00492
Oxidizable organic matter:			
Before aeration	5.000	4.24	4.24
After aeration	1.665	2.94	1.80

It is to be noted that the albuminoid ammonia is very low before treatment.

At Albany aeration is accomplished by allowing the water to spray into the settling-reservoir through small holes in the vertical inlet-pipes (see Figs. 120, 121).

A more effective form of aerator is that used in the filter plant at Reading, Pa., and shown in Fig. 139b. As at Albany the aerator is

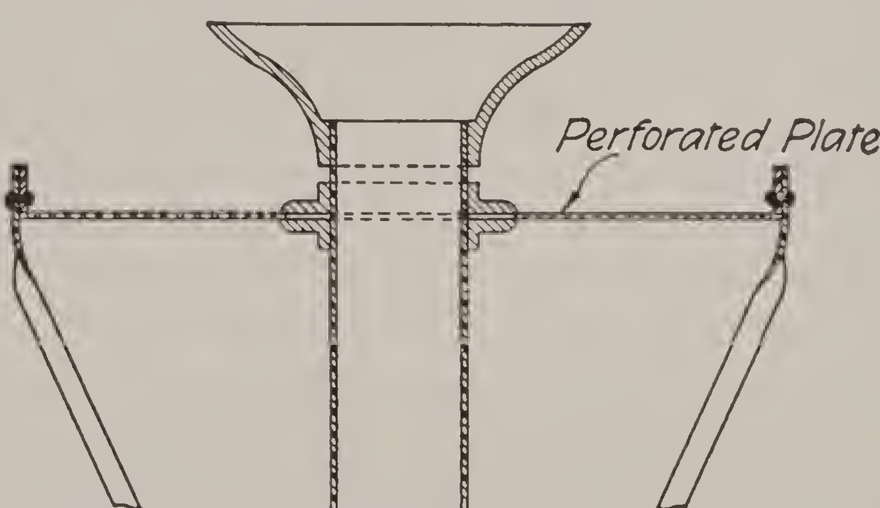


FIG. 139b. AERATOR HEAD, READING, PA.

attached to the inlet pipes of the settling reservoir. The water flows over the enlarged lip of the vertical outlet pipe and falls through a large horizontal perforated plate into the reservoir.

559. Softening of Water. — Water is rendered hard by the presence of lime and magnesia, chiefly in the form of carbonates and sulfates, but occasionally as chlorids and nitrates. The carbonates cause so-called temporary hardness (removable by boiling), while the sulfates and other compounds cause permanent hardness. The various objections to a hard water have been fully pointed out in Chapter IX

* *Eng. Record*, 1896, XXXIV. p. 201; 1899, XL. p. 155.

(Art. 159), but it may be well to repeat here the most important facts. In using a hard water for washing purposes approximately 2 ounces of soap are neutralized or wasted for each 100 gallons of water for each grain per gallon of calcium carbonate or its equivalent. In boiler use the carbonates of lime and magnesia are precipitated, forming a deposit which can usually be removed by blowing out, unless accompanied by scale-forming substances. Sulfate of lime precipitates at high temperatures and forms a very hard, objectionable scale, particularly if the water contains other suspended matter. The solubility of the sulfate is approximately given by the following table:

Temperature Fahr.	Pressure, Lbs. above Atmospheric.	Grains per Gallon, $CaSO_4$.
32	...	111
68	...	120
104	...	125
140	...	121
176	...	114
212	0	111
284	37.8	45
324.5	80.8	33
356.5	132.0	16
473	513.5	10

Of the other substances the sulfate of magnesium is the most common. It is objectionable as tending to decompose at high temperatures, forming scale.

560. *Chemistry of Water Softening.*—The softening of water is accomplished by simple processes of chemical precipitation. To remove the carbonates, lime is used as the precipitant. The carbonates are held in solution chiefly by virtue of the carbonic acid dissolved in the water, and on adding lime the acid unites with it, forming carbonate of lime. In the case of hardness due to the carbonate of lime the reaction is

$$CaCO_3 + CO_2 + Ca(OH)_2 = 2CaCO_3 + H_2O.$$

The resulting carbonate is now but slightly soluble and so precipitates out.* With the carbonate of magnesia, a similar reaction is presumed to first take place, thus:

$$MgCO_3 + CO_2 + Ca(OH)_2 = MgCO_3 + CaCO_3 + H_2O;$$

* The lime may also be considered as being present as a bicarbonate, which changes to the insoluble carbonate when $Ca(OH)_2$ is added.

but as the carbonate of magnesia is quite soluble, a further quantity of lime is required to complete the process, thus:

$$MgCO_3 + Ca(OH)_2 = Mg(OH)_2 + CaCO_3.$$

The hydrate precipitates out.

To remove the sulfates, sodium carbonate (Na_2CO_3) is used. Lime must also be added in the case of magnesium sulfate. The reactions are:

$$CaSO_4 + Na_2CO_3 = CaCO_3 + Na_2SO_4,$$

and

$$MgSO_4 + Ca(OH)_2 + Na_2CO_3 = Mg(OH)_2 + CaCO_3 + Na_2SO_4.$$

The sodium sulfate resulting from these reactions is very soluble and unobjectionable in the amount likely to be present. The chlorids and nitrates may be removed in the same way as the sulfates.

561. *General Features.*—The lime process for the removal of temporary hardness was invented in 1841 by Dr. Clark of England, and is commonly known by his name. It has been used quite extensively in that country, where many towns are supplied with water drawn from the chalk deposits. Various methods of carrying out the details of the process, relating principally to the application of the lime and the removal of the precipitate, have been devised. These are known under various names, but the general principle is the same in all. The lime is usually added in the form of lime-water, although milk of lime is also used. When both permanent and temporary hardness are to be removed it is necessary to add both lime and sodium carbonate.

The chief features of a softening plant relate to the apparatus for preparing and introducing the chemical, the sedimentation basins for the removal of the main body of the precipitate and the final filtration or clarification of the settled water. The lime water is usually prepared as a standard saturated solution. After it is introduced the mixing is accomplished and the chemical action hastened by agitation of the water either by passing it rapidly through baffled channels or by means of steam or compressed air or by mechanical devices. This agitation also assists in subsequent precipitation of the finer particles by means of the coagulating action of the larger particles. The precipitation is carried out in ordinary settling basins, after which the partially cleared water is usually filtered through some form of rapid filter. For this purpose cloth filter presses are often used, while in some of the largest modern plants the ordinary rapid sand filter is employed. Traces of free alkali which may remain in the softened water may be removed by adding CO_2, or by mixing in a small proportion of the untreated hard water.

In the original Clark process the precipitate was removed by subsidence in large tanks. In the Porter-Clark process (one of the most commonly used processes) the water, after the application of the lime, rises slowly through an iron cylinder containing broad shelves on which the precipitate settles, and from which it is scraped at intervals by means of a series of paddles. The final cleaning takes place in settling-basins.

In the Archbutt-Deeley Process, used in several modern English plants, the precipitation is aided by stirring up for several minutes some of the previously accumulated sediment. After sedimentation the small amount of $CaCO_3$ remaining in suspension is redissolved and all free alkali removed by adding CO_2 obtained from a small coke stove. This recarbonizing also renders the water more palatable.

562. *Examples of Softening Plants.* — One of the largest softening plants yet constructed is that at Southampton, England, where the entire city supply is softened to the extent of reducing the hardness from 18°, Clark scale,* to about 6°. The capacity of the plant was, in 1892, 2,400,000 gallons per day. The lime is burnt in a kiln near at hand. The slaked lime is dissolved in the softened water in two large cylinders, the amount taken in solution being about 75 grains per gallon. At this ratio it requires for this plant about one-tenth as much lime-water as the amount of water to be treated. After receiving the chemical, the water passes into a large cistern, where much of the precipitate settles; the finer particles are removed by a series of Atkin filters. These filters consist of perforated zinc disks covered with filter-cloths and arranged in pairs along a hollow shaft. They are immersed in the water to be filtered. The water passes through these disks to the space between them and thence through the hollow shaft to the outlet. The filters are cleaned every 6 or 7 hours by spraying them from fixed perforated pipes, the disks and shaft being rotated at the same time.

The cost of the plant is stated to be about $48,000, which is equivalent to $20,000 per million gallons capacity. The cost of operation is about $4 per million gallons.†

At Columbus, Ohio, a softening plant with a daily capacity of 30,000,000 gallons is being constructed (1908). Lime-water and a solution of soda ash will be used to eliminate the carbonates and sulfates. Rapid filters are used to remove the precipitate from the softening process, and during times of high turbidity of the raw water the relatively large amounts of gelatinous hydrate of magnesia will act as a coagulant. Arrangements are provided to by-pass raw water into the softened water, in order to eliminate any traces of caustic alkalinity which, if permitted to remain, would cause a hard precipitate

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* 1° = 1 grain of carbonate per Imperial gallon = 1 part in 70,000.

† Proc. Inst. C. E., 1891–92, CVIII. p. 285; *Eng. News*, April 16, 1892, p. 380.

in the pipes. When required, sulfate of iron or alumina, can be added as an additional coagulant to remove excessive turbidity. The lime-water is prepared by introducing 10 per cent milk of lime into the desired amount of raw water which is then conveyed to the bottom of the mixing tank where it is stirred mechanically. It gradually rises in this tank, clearing as it rises, and is drawn off at the top as lime water. Weirs and Venturi meters are used for measuring purposes.*

Other municipal plants worthy of note are those at Oberlin, O., and Winnipeg, Canada.†

563. *Softening of Water for Boiler Use.* — In Europe, plants for the softening of boiler feed-water have been in general use for many years and recently many such plants have been installed by railroad companies in this country. The operation of these plants has resulted in a great economy in locomotive maintenance. In some of these plants the precipitate is removed by the use of settling-tanks alone but generally some form of rapid filter is used. The chemicals used are lime and usually soda ash, or crude sodium carbonate.†

Many scale preventives have been proposed for use in boilers, but probably the best in general use is sodium carbonate. This breaks up the sulfates as previously shown, and thus prevents the formation of a hard deposit; but the precipitation of the carbonates is increased by the process. The sodium sulfate remains in solution, but should not be allowed to concentrate too greatly or it will cause foaming.

564. *Bacterial Efficiency of the Softening Process.* — Experiments by Frankland, and results of operation in practice, show a considerable degree of bacterial purification in the softening process, — in some cases quite as high as that of other processes. The lime precipitate is pulverulent instead of gelatinous in character, and experiments at Louisville showed the process to be in general quite inadequate for the removal of bacteria, unless lime be introduced in considerable excess (Art. 475). Where water contains magnesia, as at Columbus, the precipitate acts effectively as a coagulant, and by the use of rapid filters the bacterial effiiency should be very good.

565. **Removal of Iron from Waters.** — *Cause of Iron in Waters.* — Ground-waters may not infrequently contain iron in solution and so have their taste and appearance impaired. Such waters are apt to act as astringents. Where iron is present in the proportion of 0.4–0.5 part per million, little or no trouble from taste or appearance is to be noted. An excess of iron, however, is not only disagreeable to the taste, but is objectionable for domestic use, especially in the laundry.

* *Eng. Record*, 1906, LIII. p. 202.
† See references at end of chapter.

The presence of iron in ground-waters is due to a series of chemical changes that are induced by the presence of organic matter. Most soils, as sands, gravel, etc., contain more or less iron, which is generally in the form of ferric oxid (Fe_2O_3). As surface-waters percolate into soil-layers containing organic matter, they are rapidly deprived, by the oxidation of the organic matter, of the free oxygen which they contain. When this supply of oxygen is used up, the organic matter attacks the insoluble ferric oxid, reducing it to ferrous oxid (FeO), which unites with the carbonic acid naturally present in the water, thus forming ferrous carbonate ($FeCO_3$), which is soluble in waters of an acid reaction. Therefore, wherever the conditions in the soil favor these chemical changes, iron may appear in the water. In alluvial plains, river valleys, and similar locations, where organic matter is more or less abundant, waters of this class are often found. In a number of different cities along the Atlantic seaboard, as on Cape Cod, Long Island, and the Jersey shore, as well as in numerous locations in the alluvial plains of Germany, Holland, and Denmark, trouble from such a source has been experienced.

In such waters, the iron bacterium, *Crenothrix*, is very likely to grow. As this form develops in darkness, it is capable of multiplying in distributing-pipes, where it may sometimes accumulate to such an extent as to seriously interfere with the service. This organism lives on the soluble iron, utilizing it as a food, finally precipitating it as ferric oxid in its gelatinous sheath, and so causing the accumulation of flocculent masses. Upon the death and decay of these organisms, bad odors and unpleasant tastes may be produced.

Waters containing these iron salts are clear when first drawn, but soon become cloudy on standing, due to the absorption of oxygen from air and the consequent conversion of the soluble ferrous salt into ferric hydroxid. This material in time settles out as a rusty precipitate. Sometimes where there is an abundance of organic matter in solution, as in waters from peaty sources, soluble compounds are formed with the organic matter that are not readily oxidized upon exposure to air.

566. *Treatment of Iron Waters.* — It has been noted that in many cases where the iron is present as ferrous carbonate it will be removed by oxidation if exposed to the air. This reaction is utilized in the practical treatment of such waters, and in most of the plants installed for the removal of iron from ground-waters aeration is employed to facilitate this oxidation. The precipitated iron (Fe_2O_3) is generally removed from the water by filtration through sand. As a heavy

flocculent deposit is produced that does not readily penetrate the filter, even where coarse sand is employed, rapid filtration is possible.

To satisfactorily treat waters of this class, it is necessary to reduce the iron content to less than 0.5 part per million. The percentage of iron removed, therefore, is not of so much value as a determination of the content of the effluent.

The extent of aeration required varies considerably, according to the character of the water, and the conditions necessary for successful treatment cannot in all cases be determined without experiment. In some cases simple exposure in open canals gives sufficient aeration, or the mere spraying in small jets, or other simple means may be successful. At Charlottenberg, and several other places in Germany, the water is passed through aerators made of coarse pieces of coke. At Beelitzhof the aerators are made of blocks of stone, and appear to work equally well. At Provincetown, Mass., Mr. H. W. Clark found that where simple aeration would not answer, a coke-filter was successful, due, it was thought, to some chemical action of the coke.

In some cases the difficulty of aeration is probably due to excess of organic and of free carbonic acid. At Reading, Mass., lime and sulfate of alumina have been successfully used in connection with aeration and filtration. This process, however, increases the hardness very considerably. Recent experiments by Mr. R. S. Weston indicate that good results can be secured by the addition of ferric hydrate electrically produced. At Provincetown, Mr. Clark found that ferrous sulfate or ferric chlorid would precipitate the iron.

567. **Application of Electricity to Water Purification.** — Electricity is indirectly applicable to the purification of water in two ways: (1) the electrolytic production of a disinfectant and deodorizer; (2) the electrolytic production of a coagulant. In both of these cases it appears that the action of the substance produced is quite the same as when produced in other ways, and the question is primarily one of the economical manufacture of the substance in question.

The principal method of producing the first-named class of compounds is by the electrolysis of salt-water or sea-water, producing thereby principally the hypochlorite of sodium, a powerful disinfectant. Electricity has also more recently been employed in the production of ozone. The action of substances of this character is discussed in Art. 570. The process is one which has so far been chiefly limited to sewage treatment, but under certain conditions may prove of value in water purification.

The other general method has been applied to the production of hydrate of iron and hydrate of alumina with results comparable with the use of those substances as already described. This subject was thoroughly investigated by Mr. Fuller at Louisville, with the conclusion * that aluminum cannot be economically used in this way on account of its excessive cost. Aluminum in the sulfate is much cheaper per pound than the metallic aluminum.

Regarding ferric hydrate, Fuller states that "under practical conditions this process (electrolytic) can be used to produce ferric hydrate, a good coagulant, up to the point where the atmospheric oxygen dissolved in the water is not completely exhausted." Since the more recent development of the use of sulfate of iron with lime the cost of the electrolytic process will, however, rarely compare favorably.

A very considerable advantage of the electrolytic production of the coagulant is that it does not add any objectionable substance to the water, or increase its hardness.

568. **The Anderson Revolving Purifier.**—This process of purification (patented) is in reality a method of adding iron as a coagulant previous to subsidence and filtration. The water is passed through a revolving cylinder, containing a quantity of iron in the form of turnings or punchings, which, as the cylinder rotates, are lifted and scattered through the water by means of projections bolted on the inside of the cylinder. The inlet and outlet are through hollow trunnions on which the cylinder rotates. The rate of flow is such as to give the water 3 to 5 minutes contact with the iron. The result of the operation is that a small amount of the iron is dissolved by the carbonic acid of the water, forming ferrous carbonate. On exposure of the water to the air in reservoirs, or by artificial aeration, this is oxidized into hydrate which acts as a coagulant similarly to aluminum hydrate, in removing color and in aiding sedimentation and filtration (see Chapter XXI). This system is in use at a number of places in Europe, notably at Antwerp, Dordrecht, and for some of the suburban supplies of Paris. After treatment the water is filtered at a moderate rate through sand filters.

569. **Sterilization and Distillation.**—Not infrequently a public supply becomes suspicious and the prudent consumer is forced to protect his household by private means. Generally speaking, the introduction of satisfactory filters will insure safety if the same are properly managed. Another method on which even greater reliance can be placed is the use of heat. There are no pathogenic bacteria that are liable to be distributed by the way of the water-supply that are able to withstand

* Fuller. Water Purification at Louisville, 1898

the influence of boiling water for a period exceeding 10–15 minutes. Cholera and typhoid succumb in 5 minutes or less. In case of sudden outbreaks of disease or temporary disturbance of installed water-supplies, this method can always be relied on with perfect safety. There are several types of sterilizers that have been placed on the market that are adapted to individual use (see literature); also apparatus that is designed for the treatment of large quantities that would be of service in case of epidemics. Ordinarily, measures of this sort are left to the option of the water-consumer, but in times of extensive epidemics, as in the Hamburg cholera outbreak of 1893, public stations supplying sterilized water may be established. Boiling does not, however, render potable a water containing large amounts of organic matter, although it may destroy the disease-germs that may be therein. By distillation a water can be obtained free from dissolved matter as well as bacteria. This process is extensively used on shipboard to obtain potable water from sea-water, and in a few places on the sea-coast for similar purposes. In a recent test of a "triple-effect" evaporator at the Dry Tortugas, Fla., a net distillation of 13.98 pounds of water per pound of coal was obtained.* Distilled water is rendered more palatable by aeration and the introduction of a small quantity of salt.

570. **Purification by Addition of Chemicals.** — Chemical substances, such as alum and iron sulfate, are frequently added to water to aid in its purification, but the object of these is to cause flocculation, and the bacteria are removed by subsidence or filtration rather than destroyed. In the main, chemical substances that are sufficiently powerful to destroy organisms in water are likely to injure the potable quality of waters, unless they can be later removed or neutralized.

571. *Ozone.* — Apparently one of the most successful of these methods is in the use of ozone which has already been applied on a commercial scale. The gas is generated electrolytically and then passed through the water. Experiments made by Weyl † on river-water such as the Spree, which contained from 16,000–18,000 bacteria per c.c., showed a reduction to 100–200 organisms per c.c. The water is pumped into a tower and allowed to flow slowly over stones, thus bringing it in contact with the air that is heavily charged with ozone. The gas acts as a powerful disinfectant, destroying the pathogenic organisms with certainty. It is not very readily absorbed by the water, hence water treated in this way does not act easily on pipes.

* *Eng. News*, 1900, XLIII. p. 203.

† 71. Versammlung deutscher Naturforscher u. Aerzte, 1899.

Recently experiments were tried at Lille, Belgium, with the apparatus of Marmier and Abraham. Calmette,* reporting these results, says that its efficiency is higher than any other known process. All pathogenic and other bacteria, with the exception of a few harmless spore-bearing hay bacilli, were destroyed. The ozonization of the water adds no element prejudicial to health.

Experiments made in 1904, by a special committee of investigation, on a plant installed at the Saint-Maur water-works, Paris, showed a bacterial efficiency above 99 per cent, the bacteria remaining in the water consisting only of harmless varieties.

For the successful working of an ozone sterilizing plant it is necessary that most waters be filtered before being ozoned. As most waters can be made satisfactory by filtration, the additional cost of the ozone process in order to secure even complete sterilization will seldom be justified. It is a method which may prove of value in special cases, but it is too expensive for ordinary use. It may also be said that it is hardly out of the experimental stage and that absolute sterility is very difficult to obtain. †

572. *Chlorinated Lime* (*Traube's Method*). — In 1893 Traube ‡ proposed an exceedingly simple and efficient method of rendering water germ-free by the addition of chlorinated lime or bleaching-powder ($CaOCl_2$). This strong disinfectant consists of a mixture of calcium hydroxid, $Ca(OH)_2$, calcium chlorid, $CaCl_2$, and calcium hypochlorite, $Ca(ClO)_2$. By virtue of the active chlorine which it contains it will destroy all bacteria in a few hours even in extremely dilute solutions. The excess of the chlorinated lime may be readily neutralized by the addition of sodium sulfite or calcium bisulfite. Water so treated is perfectly harmless, and does not have its taste or appearance impaired; in fact, remains unchanged except for a slight increase in hardness.

Other methods of utilizing the strong disinfecting action of chlorine have been tried with some success. At Ostend, successful experiments have been made with a process in which a compound of chlorine and oxygen has been used. Chlorid of lime with chlorid of iron is used in the so-called "ferro-chlore" process. This process has been shown to give satisfactory results in experiments in Belgium and also at Paris. §

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* *Rev. Sci.*, 4 Ser. XI. p. 432.

† See report on use of Ozone for the Croton Water-supply. *Eng. News*, 1907, LVIII. p. 561.

‡ *Zeit. f. Hyg.*, 1894, XVI. p. 149.

§ *Eng. Record*, 1906, LIV. p. 94.

572a. *Peroxide of Hydrogen* (H_2O_2). — This disinfecting agent can also be utilized in the sterilization of water. In solutions of 1 : 10,000 the cholera organism is killed in five minutes; the typhoid bacillus in one day in solutions of double this strength. In proportions of 1 : 1000, water is rendered practically germ-free within 24 hours, and these proportions do not affect the taste.

572b. *Copper Sulfate.* — The action of copper sulfate as a germicide is well known and its use for this purpose has been more or less studied, but it has been generally objected to on account of its possible deleterious effect on the human system. Its use to destroy and prevent the growth of objectionable algae and other microscopical organisms in reservoirs is of much more importance and has been successfully applied in many cases.

Attention was directed to the highly toxic effect of copper sulfate upon algae by the studies of Messrs. Moore and Kellerman in the laboratory of Plant Physiology in the U. S. Department of Agriculture, published in 1904.* From these and other studies, and from actual experience in practice, it is found that an amount of copper sulfate of 1 part in 2,000,000 is sufficient to destroy most of the objectionable forms of organisms, some being rapidly destroyed with an application of only 1 part in 20,000,000. In these minute quantities no harmful effect can arise from its use in a drinking water, and considering that very few applications are needed during the season and that a large portion of the copper is precipitated with the organisms, there would seem to be no objection to its use under proper supervision. The method of application which has been frequently employed is to drag sacks containing the copper sulfate back and forth through the reservoir or pond in a more or less systematic manner. With careful manipulation this method will serve to distribute satisfactorily the desired amount of material, but at the best it would appear that some form of spray apparatus using a definite solution would be more satisfactory.

The use of copper sulphate as an algicide has been applied in many cases with good results. At Butte, Mont., a reservoir of 180,000,000 gallons was given two treatments at a cost of 23 cents per million gallons. An amount of copper sulfate equivalent to 1 part in 8,000,000 was sufficient to destroy the *asterionella* and *anabaena* which caused the trouble. At Hanover, N. H., a reservoir of 100,000,000 gallons received a single treatment, using a proportion of 1 part in 4,000,000. The number of micro-organisms was decreased in 24 hours from 600 per c.c. to 60 and wholly eliminated in 60 hours.†

* Bulletin No. 64, Bureau Plant Ind. † See references on p. 549.

The germicidal effect of copper is well established, but in general it is not effective within limits that would be permissible. A promising method of using it for this purpose is in connection with iron and lime as a coagulant in rapid filtration. Tests at Marietta, O., in which a combined sulfate of iron and copper was used containing only 1 per cent of copper sulfate, showed very good results. The average of eleven runs gave an effluent containing but 12 bacteria per c.c., the number in the river water being 1913.*

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C. WORKS FOR THE DISTRIBUTION OF WATER.

CHAPTER XXIV.

PIPES FOR CONVEYING WATER.

573. **Materials Employed.**—A variety of materials may be employed for the construction of water-conduits. If the conduit is not under pressure, the form of construction used may be an open canal dug in the natural earth, or a masonry conduit in "cut and cover," or a tunnel. Where the water flows under pressure the first two types are not suitable and a pipe, or possibly a tunnel, must be employed. The method of construction used in connection with canals, masonry conduits, and tunnels will be described in the next chapter; the present chapter will deal only with the design and manufacture of pipes.

The materials used for water-pipes are cast iron, wrought iron, steel, wood, cement, vitrified clay, lead, and occasionally a few other materials. The important requirements for a water-pipe are strength, durability, and low cost. The relative importance of these requirements will vary under different circumstances, and this will lead to the use of different materials in different cases.

574. **Stresses to be Considered.**—The stresses to be considered in the design of water-pipes are those due to (1) the water-pressure, (2) the pressure of the surrounding earth and the action of other outside forces, (3) changes of temperature, and (4) blows and shocks received in transportation and construction.

575. *Stresses Due to Water-pressure.*—The maximum pressure to be provided for will be the maximum which can occur under normal conditions of operation (usually the maximum possible static pressure), plus an allowance for water-hammer. The former can readily be computed, but the latter is difficult of estimation.

Sometimes pipe-lines are so designed that static pressure can never occur, the valves being so arranged that the water never comes to rest.

In that case the maximum pressure at any point will be the maximum pressure-head which can exist under the assumed conditions of flow. This will be less than the static head by the amount lost in friction from the open end of the pipe to the point in question. (See further discussion in Art. 630.)

The dynamic effect, or the amount of water-hammer to be assumed depends on many circumstances. It was shown in Chapter XII that it varies in general with the length of the pipe, with the velocity of the water, and with the rapidity with which the velocity is changed by the action of valves or otherwise. The amount to be allowed should evidently be varied according to the nature of the pipe-line.

In a distributing system of small pipes where the operation of hydrants and large branches has a relatively great influence on the system, the allowance should be large. In this case the amount added for water-hammer has commonly been about 100 pounds per square inch, which, from the theoretical considerations of Chapter XII would appear to be quite high enough for all ordinary cases. The amount assumed for cast-iron pipes in the new pipe system of the Metropolitan Water-works of Boston is given on page 556.

In the case of a large pipe-line without branches, and carefully protected from excessive pressure by relief-valves and by precautions in operating shut-off valves, the allowance for water-hammer need be very little, especially for pipes of steel or wood. It is true that any reduction whatever in velocity, due to the closing of a valve, will raise the pressure an amount proportional to the length of the pipe affected, the velocity of the water, and inversely as to the time required in closing. For example, if a stop-valve of a 48-inch pipe be closed in 60 seconds, the average pressure with four miles of pipe-line would be 14 pounds per square inch. Large wooden and steel pipe-lines are commonly designed with little or no allowance for hammer, but for those portions under light pressure it would be well to make an allowance of 25 to 50 pounds, depending on the velocity of the water and the length of pipe involved. For those portions of the pipe under heavy pressure the ram would be small in proportion to the static pressure, and the necessity for considering it would be less.

The intensity of the circumferential stress in a circular pipe is

$$s = \frac{pr}{t}, \qquad \qquad (1)$$

where r = radius of pipe,

p = pressure-head, and

t = thickness of shell.

Water-pressure must be specially considered at sharp curves and angles. At such places the pressure tends to displace the pipe-line and force the pipes apart.

576. *Stresses Due to Earth Filling and Other Outside Forces.*— The pressures due to the forces here considered tend to collapse the pipe. The effect of earth filling will be felt seriously only for very deep trenches and for large pipes, while the effect of traffic is of importance only for very shallow filling. To protect pipes from injury due to traffic a minimum depth of covering of 2 to 3 feet will usually be sufficient, since the pipes themselves are able to sustain a very considerable load if it is distributed. The stresses caused by heavy loads of earth need to be more fully considered, and a rough analysis of the problem will be of some assistance.

If we neglect the lateral support of the earth and assume the weight of filling applied as a vertical load, uniformly distributed over a width equal to the diameter of the pipe, and assume also the upward pressure against the pipe to be similarly applied, there will be produced equal bending moments at a and b (Fig. 140), but of opposite sign.* If $W =$ total load and $d =$ diameter of pipe, the bending moment at these points will therefore be

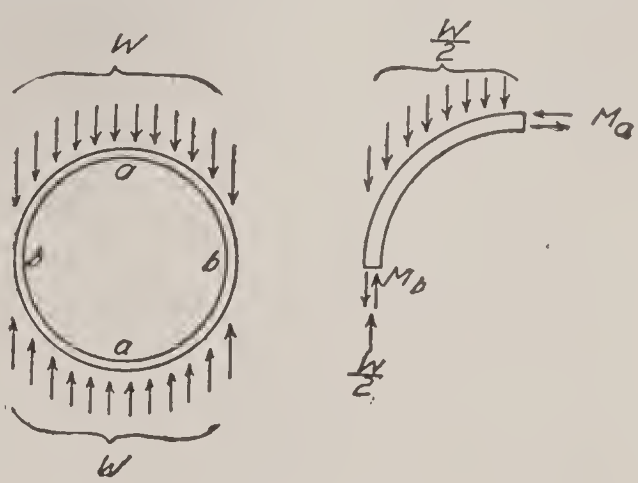

FIG. 140.

$$M = \tfrac{1}{16} Wd.$$

Assuming $h =$ depth of fill in feet; weight of filling $=$ 100 pounds per cubic foot; $f =$ safe fibre-stress in bending for the pipe material; $d =$ diameter of pipe in inches; and $t =$ thickness of pipe in inches, we derive, from the ordinary beam formula, approximately

$$t = \tfrac{1}{2} d \sqrt{\frac{h}{f}}. \qquad (2)$$

If, for example, we assume for cast iron a value of $f = 7000$ pounds per square inch, we will have $t = .006 d \sqrt{h}$. Thus for a 48-inch cast-iron pipe, and a depth of filling of 16 feet $t = 1.15$ inches, and for a

* Because a similar set of horizontally applied forces must reduce the moments at a and b to zero. See also a paper by Wm. H. Searles on "Deflections and Strains in a Flexible Ring under Load." Jour. Assn. Eng. Soc., 1895, xv. p. 124.

depth of 25 feet $t = 1.44$ inches. The smaller of these values is about as small as would be used in any case for this size of pipe.

This analysis is of course very rough, but it serves to give some notion of the maximum stress that is possible from earth pressure. It is to be noted that we have here entirely neglected the lateral pressures involved. In the case of cast-iron pipe the material is so rigid that the lateral support received by the earth may be very little and the load will be supported largely through the bending resistance of the pipe; but if the pipe is relatively flexible, like steel or even a wooden-stave pipe, it will get much aid from this lateral pressure, especially if the earth is well tamped in place. Cases of the breakage of cast-iron pipe under high embankments have occurred, but the above analysis indicates that usually no account need be taken of earth pressures when the depth of filling is less than 10 or 15 feet.

In the case of large steel pipes built of comparatively thin material stiffening-rings are sometimes used to support heavy loads, as, for example, on the large Brooklyn line, where stiffening-rings of $4 \times 4 \times \frac{5}{8}$-inch angles were used under all waterways and wherever the covering exceeded 6 feet. A covering of concrete is also sometimes employed to give additional strength. In most cases, however, no trouble will be had if the back-filling is well tamped, and the pipe perhaps temporarily supported by interior braces. Where the filling is not well done steel pipes have been greatly flattened at the top by the load of earth. At Portland, Oregon, a flattening of 4 inches was caused in this way. Experiments there made showed, however, that a distortion of as much as $8\frac{3}{4}$ inches in a 42-inch steel pipe caused no leaks, although a flattening of only $1\frac{5}{8}$ inches caused a permanent set of $\frac{1}{2}$ inch.*

Experiments on a 61-inch cast-iron pipe, $1\frac{1}{4}$ inches thick, for the Sudbury conduit, showed a difference of from .005 to .01 foot between horizontal and vertical diameters due to deflection from its own weight, and a maximum deflection of .015 foot under a load of 4 feet of gravel.†

Another possible outside force which should be considered in the design is the unbalanced pressure due to the creation of a partial vacuum when emptying the pipe. The capacity of the air-valves should be made such as to preclude dangerous pressures from this source.

577. *Stresses Due to Temperature Changes.*—If no expansion or

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* Trans. Am. Soc. C. E., 1897, XXXVIII. p. 93.

† *Eng. Record*, 1898, XXXVIII. p. 51.

contraction is allowed in a pipe-line, the longitudinal stresses due to changes of temperature will be equal to

$$s = ETc, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

where $s =$ intensity of stress;
$E =$ modulus of elasticity;
$T =$ change of temperature;
$c =$ coefficient of expansion.

Temperature stresses, as a rule, need not be considered except in the case of riveted steel pipe. (See Art. 593.)

578. *Stresses Caused in Transportation and Construction.*—Such materials as cast-iron and vitrified pipe require a considerable thickness to provide for these stresses. The necessary allowance for this purpose has been determined by practical experience, and account of it is taken in the formulas and rules for thickness.

CAST-IRON PIPE.

579. **General.**—Cast iron is the most widely used material for water-pipe. By reason of its moderate cost, its durability, and the convenience with which it may be cast in any desired form it is almost universally employed for the pipes and various special forms of distributing systems. It is also frequently employed for large pipe-lines, and is now easily obtained in any desired diameter up to 6 feet or more. Cast-iron pipes are made in lengths of about 12 feet, which are joined together usually by the bell-and-spigot joint run with lead. Branches and other irregular forms are used for connections. These are called special castings, or simply "specials."

580. **Thickness and Weight of Cast-iron Pipe.**—The material used for pipes is usually required to have a tensile strength of from 16,000 to 18,000 pounds per square inch. A factor of safety of 5 may be assumed where proper allowance is made for water-hammer. In addition to the thickness required to sustain water-pressure a small addition must be made to allow for eccentricity of casting and to provide sufficient strength to bear transportation. One-tenth of an inch should be sufficient for the first allowance. For the second object it would seem that no allowance at all need be made for such sizes and pressures that the thickness required to sustain the water-pressure would be large. However, it is customary to make some allowance

for all sizes. The total amount allowed for both the above-mentioned objects varies in the different formulas from about .25 to .35 inch.

Different formulas are used by different pipe-foundries and by different cities in determining the thickness of pipe. A formula which commends itself as being simple in form and rational in its make-up is that used by the Metropolitan Water-works of Boston. It is

$$t = \frac{(p + p')r}{3300} + 0.25, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (4)$$

where t = thickness in inches;
p = static pressure in pounds per square inch;
p' = allowance for water-hammer in pounds per square inch;
r = radius of pipe in inches;
0.25 = allowance for eccentricity, deterioration, and safety in handling.

The value of p' is to be taken as follows:

Size of Pipe.	Value of p'.
3-in. to 10-in.	120
12-in.	110
16-in.	100
20-in.	90
24-in.	85
30-in.	80
36-in.	75
42-in. to 60-in.	70

This formula assumes a strength of 16,500 pounds per square inch and a factor of safety of 5. It gives pipe somewhat thinner than that formerly used by the Boston Water-works, and about as light as it is advisable to use. It properly varies the allowance for water-hammer according to the size of pipe.

Large cities usually adopt a few standard thicknesses for each size, corresponding to certain pressures, the pipes designed for the different pressures being designated as Class A, Class B, etc. The variations between classes usually correspond to a difference of pressure of about 50 pounds per square inch. The various pipe-foundries have likewise their standard weights for different sizes, which differ more or less among themselves and also differ from the various city standards. For large orders of pipe it is easy to secure any designated weights, but for small orders it will be economy to select from the standards given in the manufacturers' lists that weight which will come nearest to the weight desired.

Standard specifications for water pipe have been adopted by the New England Water-Works Association, which include standards as to weights and dimensions of various classes of pipe and of various specials. These are to be commended as being the result of much careful study and discussion and as aiding greatly in standardizing and improving current practice in this important particular.* Table No. 74 furnishes data in accordance with these standards.

In determining the thickness of various classes of pipe formula (4) has been used and pressures from 50 to 500 feet assumed, although it is not the intent of the specifications to recommend any particular class for a given pressure. Variations in outside diameter are made as few as practicable, the variation in thickness being secured principally by varying the inside diameter. The variations in special castings are fewer than in straight pipe.

581. Joints.—*The Ordinary Bell-and-spigot Joint.*—The joint which is ordinarily employed in this country is the bell-and-spigot joint. The space between bell and spigot is filled with lead, which is calked solidly into place so as to be water-tight. Many forms of bell or socket have been devised, but practice has come to be quite uniform on this point, and is well represented by the standard shown in Fig. 142. The chief requisites of a bell and spigot are: 1st, sufficient space to allow of thorough calking, but no more space than necessary; 2d, sufficient depth of bell to enable a tight joint to be made and to give considerable lateral strength to the pipe; 3d, sufficient strength of bell to resist the bursting-force due to calking. It will be noted from the illustrations that a groove is formed on the interior of the bell. This is for the purpose of holding the lead more firmly in place. The interior surface of the pipe at the joints should be as smooth as possible. In the case of some large pipe recently laid, the joints on the interior of the pipe were filled with Portland-cement mortar in order to give a smooth surface.

In Table No. 75 are given the various dimensions of standard bell and spigot in accordance with the specifications of the New England Water-Works Association (Fig. 142), together with amounts of lead and packing required per joint.

The ordinary bell-and-spigot joint with lead packing will enable pipes to expand and contract under moderate changes of temperature such as occur with buried pipes.

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* These specifications may be had from the Secretary of the New England Water-Works Association, Boston, Mass. They contain full tables of pipe and special castings. See also Jour. New Eng. W. W. Ass'n, December, 1902, March, 1903.

TABLE NO. 74.

STANDARD THICKNESSES AND WEIGHTS OF CAST IRON PIPE ACCORDING TO THE SPECIFICATIONS OF THE NEW ENGLAND WATER-WORKS ASSOCIATION.

Nominal Diameter of Pipe in Ins.	Class A. 50-Ft. Head.			Class B. 100-Ft. Head.			Class C. 150-Ft. Head.			Class D. 200-Ft. Head.			Class E. 250-Ft. Head.		
	Thickness of Shell, Inches.	Weight per 12-Ft. Length, Pounds.	Weight per Ft. Exclusive of Socket, Pounds.	Thickness of Shell, Inches.	Weight per 12-Ft. Length, Pounds.	Weight per Ft. Exclusive of Socket, Pounds.	Thickness of Shell, Inches.	Weight per 12-Ft. Length, Pounds.	Weight per Ft. Exclusive of Socket, Pounds.	Thickness of Shell, Inches.	Weight per 12-Ft. Length, Pounds.	Weight per Ft. Exclusive of Socket, Pounds.	Thickness of Shell, Inches.	Weight per 12-Ft. Length, Pounds.	Weight per Ft. Exclusive of Socket, Pounds.
4	.34	200	14.89	....	....	....	.36	215	15.70	....	....	....	.39	230	16.92
6	.38	330	24.32	....	....	....	.42	350	26.92	....	....	....	.46	380	29.08
8	.42	475	35.58	....	....	....	.48	530	40.38	....	....	....	.53	575	44.33
10	.47	650	49.04	.50	680	52.03	.53	720	54.99	.56	760	57.94	.60	810	63.61
12	.49	810	61.14	.53	855	65.92	.57	910	70.67	.61	970	75.39	.65	1040	81.99
14	.53	1010	76.85	.57	1080	82.41	.61	1150	87.97	.66	1220	94.85	.70	1310	102.73
16	.55	1215	90.98	.60	1300	98.95	.65	1390	106.9	.70	1490	114.8	.75	1610	125.5
18	.57	1400	104.5	.63	1520	115.2	.69	1660	127.4	.75	1780	138.0	.80	1910	148.4
20	.60	1610	121.9	.66	1760	133.7	.72	1920	147.6	.79	2090	161.4	.85	2260	175.6
24	.64	2050	155.6	.72	2290	174.4	.80	2550	196.3	.88	2780	215.3	.95	3000	234.5
30	.71	2860	215.3	.81	3230	244.8	.91	3600	277.7	1.01	3950	307.3	1.10	4340	338.0
36	.79	3800	287.0	.90	4270	326.0	1.02	4840	373.3	1.13	5310	412.3	1.25	5900	459.6
42	.87	4920	368.4	1.00	5560	422.1	1.13	6270	481.1	1.27	6970	538.9	1.40	7720	600.6
48	.95	6130	459.3	1.10	6970	530.2	1.25	7920	608.0	1.40	8780	678.9	1.55	9740	758.5
54	1.03	7510	559.8	1.20	8600	650.3	1.37	98[illegible]0	749.5	1.54	10900	839.9	1.72	12400	946.9
60	1.10	8900	664.0	1.30	10300	782.3	1.50	11900	911.5	1.70	13300	1029.7	1.90	15100	1162.0

Nominal Diameter of Pipe in Ins.	Class F. 300-Ft. Head.			Class G. 350-Ft. Head.			Class H. 400-Ft. Head.			Class I. 450-Ft. Head.			Class K. 500-Ft. Head.		
	Thickness of Shell, Inches.	Weight per 12-Ft. Length, Pounds.	Weight per Ft. Exclusive of Socket, Pounds.	Thickness of Shell, Inches.	Weight per 12-Ft. Length, Pounds.	Weight per Ft. Exclusive of Socket, Pounds.	Thickness of Shell, Inches.	Weight per 12-Ft. Length, Pounds.	Weight per Ft. Exclusive of Socket, Pounds.	Thickness of Shell, Inches.	Weight per 12-Ft. Length, Pounds.	Weight per Ft. Exclusive of Socket, Pounds.	Thickness of Shell, Inches.	Weight per 12-Ft. Length, Pounds.	Weight per Ft. Exclusive of Socket, Pounds.
4	....	....	....	.42	250	18.89	....	....	....	.45	265	20.10	.48	280	21.30
6	....	....	....	.50	420	32.40	....	....	....	.54	445	34.79	....	....	....
8	....	....	....	.58	64	49.65	....	....	....	.63	690	53.62	....	....	....
10	.63	850	66.61	.67	890	70.57	.70	935	73.53	....	....	....	....	....	....
12	.69	1100	86.77	.73	1160	91.51	.77	1220	96.22	....	....	....	....	....	....
14	.75	1390	109.70	.79	1460	115.24	.83	153	120.74	....	....	....	....	....	....
16	.80	1710	133.5	.85	1810	141.4	.90	1900	149.3	....	....	....	....	....	....
18	.86	2040	159.0	....	....	....	....	....	....	....	....	....	....	....	....
20	.92	2420	189.5	....	....	....	....	....	....	....	....	....	....	....	....
24	1.03	3240	253.5	....	....	....	....	....	....	....	....	....	....	....	....
30	1.20	4700	367.5	....	....	....	....	....	....	....	....	....	....	....	....
36	1.37	6400	502.0	....	....	....	....	....	....	....	....	....	....	....	....
42	1.53	8360	654.4	....	....	....	....	....	....	....	....	....	....	....	....
48	1.70	10600	829.4	....	....	....	....	....	....	....	....	....	....	....	....
54	1.90	13500	1042.7	....	....	....	....	....	....	....	....	....	....	....	....
60	2.10	16500	1280.0	....	....	....	....	....	....	....	....	....	....	....	....

Curves of large radius can be constructed with straight pipe by deflecting each length slightly. In this way it is possible, with a reasonable deflection, to lay 4- to 8-inch pipe to a curve of 150-foot radius, a 16-inch pipe to a 250-foot radius and a 36-inch pipe to a 500-foot radius.

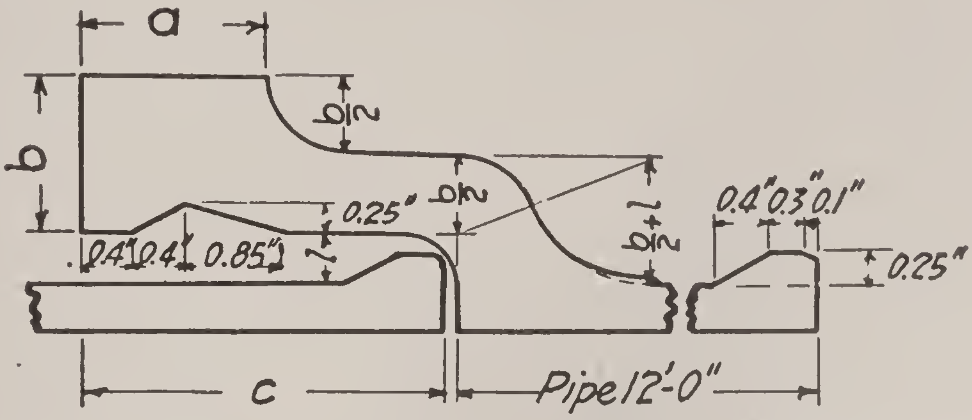


FIG. 142.—STANDARD BELL AND SPIGOT, NEW ENG. W. W. ASS'N STANDARD.

TABLE NO. 75.

GENERAL DIMENSIONS OF STANDARD BELL-AND-SPIGOT PIPE ACCORDING TO THE SPECIFICATIONS OF THE NEW ENGLAND WATER-WORKS ASSOCIATION.

Nominal Diameter, Inches.	Classes.	Dimensions in Inches.				Average Weight of Lead per Joint.		Weight of Jute Gasket per Joint.
		"a"	"b"	"c"	"l"	With Gasket.	Solid Lead.	
4	All	1.50	1.30	3.00	0.40	7.00	9.25	.10
6	"	1.50	1.40	3.00	0.40	9.75	12.75	.15
8	"	1.50	1.50	3.50	0.40	12.50	18.75	.25
10	"	1.50	1.50	3.50	0.40	15.25	23.25	.30
12	"	1.50	1.60	3.50	0.40	18.00	27.00	.35
14	"	1.50	1.70	3.50	0.40	20.50	31.00	.40
16	"	1.75	1.80	4.00	0.50	31.25	50.50	.65
18	"	1.75	1.90	4.00	0.50	34.75	55.50	.70
20	"	1.75	2.00	4.00	0.50	38.50	62.00	.80
24	"	2.00	2.10	4.00	0.50	45.50	74.00	.95
30	"	2.00	2.30	4.50	0.50	56.00	100.50	1.55
36	"	2.00	2.50	4.50	0.50	67.00	120.50	1.85
42	"	2.00	2.80	5.00	0.50	77.50	154.00	2.60
48	"	2.00	3.00	5.00	0.50	88.50	176.00	3.00
54	A, B, C, D	2.75	3.20	5.50	0.50	99.50	215.00	3.95
54	E, F	2.75	3.80	5.50	0.50	100.00	215.50	3.95
60	A, B, C, D	2.75	3.40	5.50	0.50	110.50	239.00	4.40
60	E, F	2.75	4.00	5.50	0.50	111.00	241.00	4.40

582. *Other Forms of Joints.*—In England and at a few points in this country the bored and turned joint has been used. A form of this type of joint is shown in Fig. 143. The inside of the socket and outside of the spigot are turned to an accurate fit, and the joint is made by simply driving the pipes together by means of a wooden ram. Sometimes cement filling is used in addition. In some cases the cost of boring and turning is reported to be less than that of lead joints, while in other cases the opposite is true.

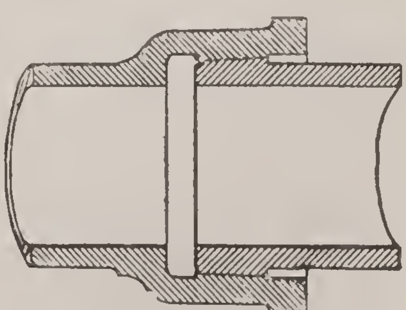


FIG. 143.
TURNED JOINT.

Wooden wedges have been employed to a limited extent in place of lead packing. At Yarmouth, Nova Scotia, such joints have been in use since 1851, and have proven so durable that they have recently been adopted as the standard for that place. The wedges are made from clear, dry, pine staves. The cost is stated to be from 6 to 13 cents per joint for sizes from 8 to 24 inches in diameter. Joints of this sort have been found in perfect condition after a lapse of from forty to forty-five years.* This form of joint would probably be advantageous where electrolysis is to be feared.

A joint that has been used somewhat in Europe, and which is especially suitable for temporary work, is made by means of a solid rubber ring. The ring is inserted in the socket near the outside edge and is rolled back by pushing the pipe into place. For this joint a smooth bell would be preferable. Other forms of rubber joints have been employed occasionally, sometimes for expansion purposes. (Art. 641.)

For inside work and connections in confined locations the flanged joint is more convenient than the bell and spigot. It is also better suited for temporary work. The flanges are faced carefully at right angles to the axis of the pipe, and the joint is bolted together with rubber or other packing between the flanges. Various standards are used for proportioning the flanges, as to thickness, number of bolts, etc., for which reference may be made to the various trade circulars.†

For joining pieces of pipe a sleeve is used, which is essentially a short piece of pipe with two bells. It is illustrated in Fig. 144. When a pipe is cut to make a connection it is usual to shrink a small half-oval or semicircular band on the end to take the place of the rib which forms the ordinary spigot.

583. Special Castings.—The ordinary special castings required are the $\frac{1}{4}$, $\frac{1}{8}$, and $\frac{1}{16}$ bends or curves, T's and crosses, or three-way and four-way branches, Y branches, blow-off branches, offsets, sleeves, caps, and plugs. The various forms are illustrated in Fig. 144. Many of the larger cities have their own standard designs for specials as well as for straight pipe, which differ more or less from the manufacturers' standards. For the smaller cities it will be much the more economical to use either the manufacturers' standards or those of some neighboring large city.‡

The various branches are manufactured either with part bell and part spigot ends, or with all bell ends. The latter form is usually

* *Jour. N. E. Water-works Assn.*, 1900, XV. p. 34.

† See also standard proposed by the Am. Soc. M. E. in Trans. Am. Soc. M. E., 1893, XIV. p. 48.

‡ See standards of the New England W. W. Assn.

preferred for branches, as it enables connections to be readily made by means of pieces of pipe.

In designing special castings consideration should be given to the fact that such castings cost, as a rule, about twice as much per pound as straight pipe. They should therefore be as light and compact as

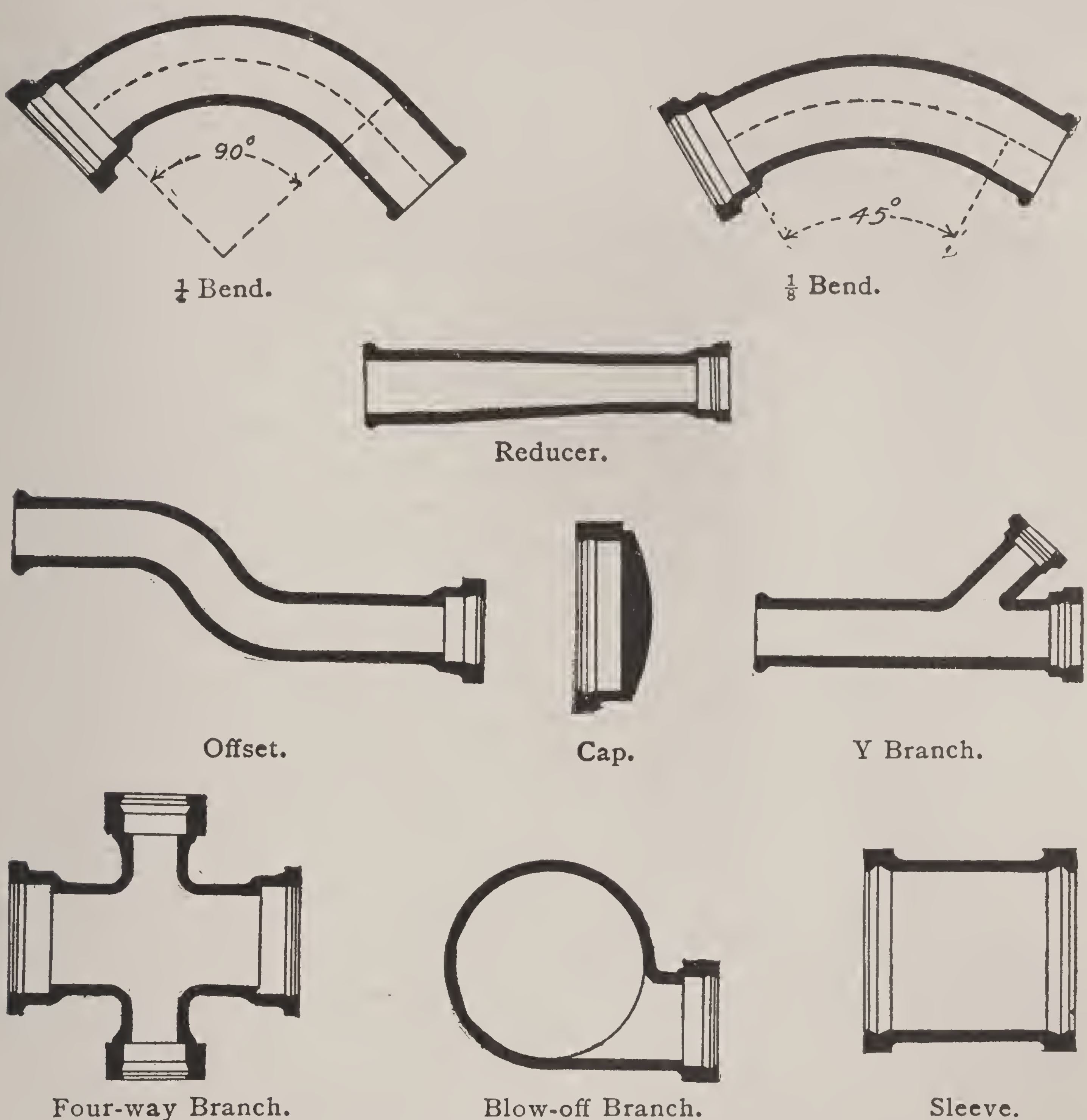
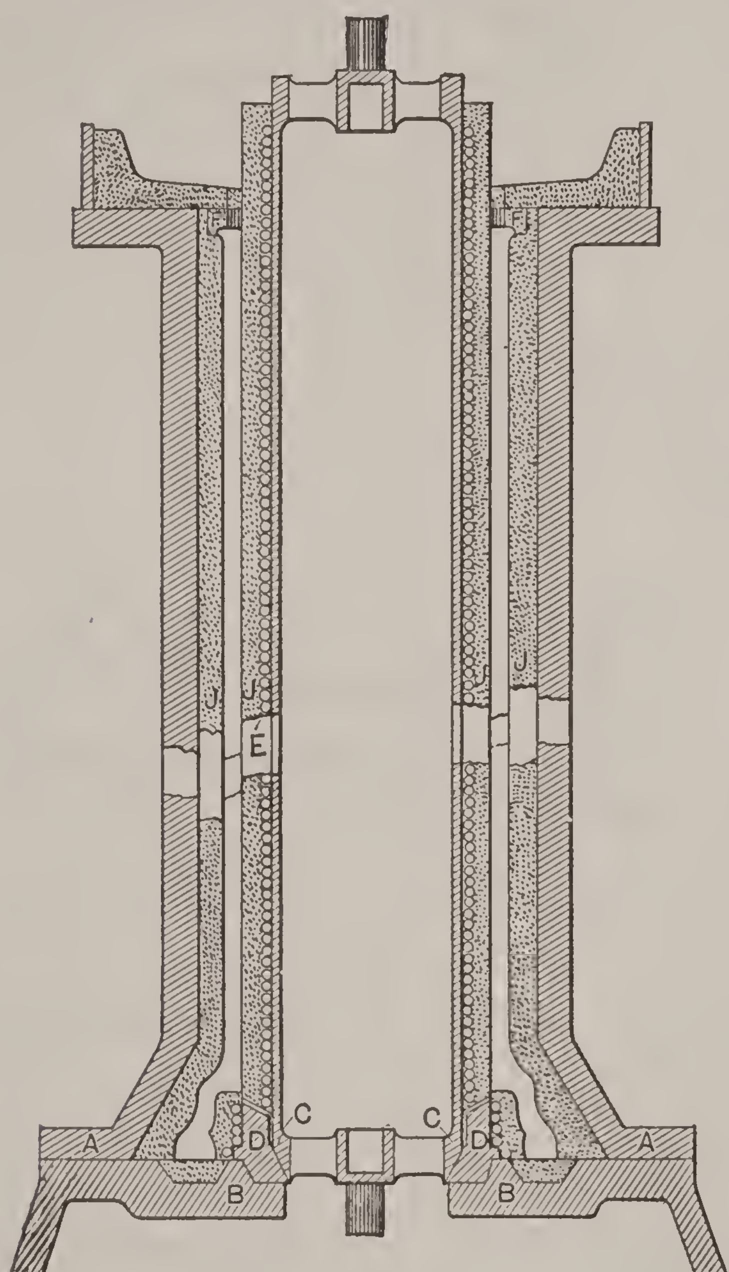


FIG. 144.—SPECIAL CASTINGS.

practicable. They are made of the same thickness as the corresponding straight pipe, but with a less number of variations for the different pressures. The general form of specials should be such as to cause as little disturbance in the water in passing around angles, etc., as practicable. This is of considerable importance where the velocity is high, and hence should be carefully considered in the design of hydrants and hydrant branches.

584. Material and Method of Manufacture.—*Quality of Iron.*—Water-pipe should be made of the best quality of gray iron, uniform in grain and soft enough to be readily worked. The metal should be made without the admixture of cinder-iron or other inferior metal, and no ordinary scrap should be used in the manufacture. A mixture of pig irons is usually required to give the best results, the proper proportions being a matter of experience. An ordinary specification for the strength of the material is that a test-bar 2 inches by 1 inch in cross-section, placed on supports 24 inches apart, shall sustain a load of 1900 pounds at the center, and shall have a deflection of at least $\frac{3}{10}$ inch before fracture. This requirement insures a certain amount of toughness, or resilience, as well as strength. Frequently a tension-test is required, the ultimate strength specified being from 16,000 to 18,000 pounds per square inch. It should be specified that test-bars shall be poured at any time during the day that the inspector desires.



A, Flask. *B*, Base.
C, Spindle. *D*, Socket.
E, Roping. *F*, Bead ring.
J, Sand.

FIG. 145.—PIPE-MOLD.
(From *Cassier's Magazine*, vol. VIII.)

585. *Molding and Casting.*—Water pipes are now always cast in vertical molds, and should be required to be cast with the bell end down, except, perhaps, for the smaller sizes. The form of mold used is shown in Fig. 145. The cores are made by winding the socket-ring *D* and the spindle *C* with hay rope, then coating with damp sand and shaping in a lathe. Much care is required in molding and casting to secure good results, and in spite of the greatest care much pipe will need to be rejected if the inspection is properly done. In the case of a contract of any considerable size the city should always employ a competent inspector to protect its interests.

Cores should be accurately centered so that the shell will be of uniform thickness, and the bells and spigots should be truly circular. The pipe should be free from all surface imperfections that will weaken it or lessen its durability, such as checks, blow-holes, sand-holes, and cold-shuts; and should be smooth and free from lumps, scales, blisters, etc. No plugging of blow-holes or the like should be permitted. Care should be taken to have the core as smooth as possible, and firm enough to support the metal. It frequently happens that ridges are formed on the interior of the pipe, due to the compression of weak cores. The smoothness of the interior is specially important in order that the resistance to the flow of water may be a minimum. Specials should be true to the designated form.

After casting, the pipe should be allowed to cool before being taken from the molds, in order to prevent unequal contraction. As soon as the pipe is uncovered it should be thoroughly cleaned of sand by means of wire and other brushes, and should then be inspected for surface imperfections and for thickness and form. To detect surface defects the inspector uses a light, pointed hammer, and for measuring the thickness calipers are applied to the pipe at several points, an allowance of from $\frac{1}{16}$ to $\frac{1}{10}$ inch being made for variations from the exact specified dimensions. The forms of the sockets and spigots are tested by templates. Defective spigots are often cut off in a lathe and a new spigot made as described on page 560. A small percentage of such defective pipe is usually allowed by the specifications. Each piece of pipe should have cast upon it a serial number to designate the number of the cast and the year, and also letters to designate the manufacturer.

586. *Coating.*—To prevent rapid deterioration, all pipe should receive some sort of protective coating. The first successful process for coating was invented by Dr. Angus Smith in 1849, and was introduced into the United States in 1858 by Mr. Kirkwood. This coating was composed of a varnish of coal-tar and linseed-oil. The ordinary coating as now used is commonly called the Angus Smith coating, but it differs considerably from that originally employed. As now applied in practice it usually consists of ordinary coal-tar, or distilled tar with dead-oil added to give fluidity to the material. Sometimes resin or creosote is added. In the process of coating, the tar is maintained at a temperature of about 300 degrees Fahrenheit. The pipe is also usually heated to about the same temperature before dipping, but is sometimes dipped cold and allowed to remain in the bath until it acquires the same temperature as the tar. Some specifications require the pipe to be removed and then redipped in order to give a thicker

coating. When cool the coating should be hard, tough, and smooth, and should not loosen under the blows of a hammer.

To obtain good results the pipe must be absolutely clean and free from rust before dipping; otherwise the tar will not adhere to the iron. It is supposed that most of the corrosion which appears in the interior of the pipe starts at a point where there is some minute defect in the coating, and it is therefore very important that the coating be continuous. In some recent work done in Boston the interior of the pipe has received an additional coating of paraffine or vulcanite applied with a brush, in the hope that any minute holes in the coating would be filled. Any injury which occurs in handling should be remedied by the application of some kind of asphalt paint or tar varnish.

Asphalt has been tried in various ways as a coating for cast-iron pipe, but without much success. It does not appear to adhere as firmly as tar.

587. *Testing and Weighing.*—After coating, each section of pipe should be subjected to an hydraulic test of from 200 to 300 or more pounds per square inch, according to the pressure for which the pipe is designed, the test pressure being considerably above the actual working pressure. While undergoing this test the pipe should be sharply rapped from end to end with a hand-hammer to detect any weakness. After this test each piece of pipe should be weighed and the weight plainly marked thereon in paint. Inasmuch as the pipe cannot be cast to exact weight, a maximum allowable variation of 3 to 4 per cent. from that specified is usually permitted. Lighter weights will cause the pipe to be rejected; heavier weights will be allowed, but not paid for.

588. **Durability of Cast-iron Pipe.**—The life of well-coated cast-iron pipe is still to be determined. The question of corrosion is an important one, not only with respect to the life of the pipe, but on account of the fact that corrosion will greatly reduce its carrying capacity. Considerable corrosion may indeed take place on the interior of the pipe without greatly impairing its strength.

The rapidity of the corrosion depends largely upon the character of the water, and, generally speaking, those waters containing considerable amounts of free carbonic acid are the worst in this particular. Many instances are reported where well-coated pipes appear to be practically unchanged after forty or fifty years of use. In other cases, especially where the water-supply is soft, some corrosion will take place in a few years. At still other places the coating appears to have been worn away or to have disintegrated. Usually some corrosion will take place in ten or fifteen years even with well-coated pipes. In most cases the corrosion is much less rapid after it has proceeded to a certain extent, and it is probably safe to say that well-coated pipe will last at least fifty years, and probably much longer.

The internal corrosion of pipes occurs in a different way from the ordinary rusting of iron. Bunches or knobs, called tubercles, form on the surface, which consist for the most part of oxid of iron, with some silica, lime, and organic matter. These may increase to a size of $1\frac{1}{2}$ to 2 inches in diameter, and $\frac{1}{2}$ to 1 inch thick. At the base of each tubercle is usually found a spot where the iron is badly corroded and often so soft that it can be cut with a knife. This soft spot may be very small, while the coating around may be well preserved. The tubercle is supposed to start at a point where the original coating is defective, a very small defect being sufficient to allow this to occur. The theory of the corrosion is that the iron is attacked by the carbonic acid in the water, thus forming ferrous carbonate, which is then oxidized to ferric hydrate by the oxygen dissolved in the water. The carbonic acid is thus set free and is capable of further attacks upon the metal. A depression is gradually formed in which the tubercle is built up.

The corrosion of the exterior of a pipe depends largely upon the character of the soil. Ashes, cinders, and the like are to be avoided for filling.

WROUGHT-IRON AND STEEL PIPE.

589. Advantages of Wrought-iron and Steel Pipe.—Wrought iron and steel have been used to a considerable extent for water-pipes, and for large pipe-lines these materials present considerable advantage over cast iron. The question is purely an economical one, and in its consideration several factors enter. Since steel is much stronger than cast iron, the use of it will give a much lighter pipe, an advantage as regards transportation, but a disadvantage as regards durability, especially for small sizes. Special forms are not so readily constructed of steel, so that for distributing-mains cast iron is much preferable. Another disadvantage of steel pipe is that with the ordinary riveted joints a considerably larger pipe is required than if a smooth cast-iron pipe is used. Thus for a diameter of 42 inches the value of c for a riveted steel pipe may be taken at 110 (page 246), while for a new cast-iron pipe it is about 130. The capacity of the steel pipe is therefore only 85 per cent of that of a cast-iron pipe of the same diameter. The discharge being nearly proportional to $d^{\frac{5}{2}}$, the necessary size of steel pipe to equal the cast-iron pipe in capacity would be given by the proportion $\frac{1}{.85} = \left(\frac{x}{42}\right)^{\frac{5}{2}}$, whence $x = 44\frac{1}{2}$ inches, which is about 6 per cent larger than the cast-iron pipe.

Steel pipe is specially adapted to long pipe-lines with few or no branches, also for high pressures, and for resisting other unusual stresses. Several large pipe-lines have been built of steel within the last few years, and it may be predicted that the use of this material will be greatly extended in the future as better means for its protection are devised. Even allowing for its more rapid corrosion, it will prove cheaper in many cases to renew it than to invest the additional money required for the cast-iron pipe. On the other hand, the inconvenience of renewal may be largely against the use of steel.

Wrought iron has been entirely superseded by steel for riveted pipes. Some experiments indicate less corrosion in the case of wrought iron, but the difference is not great enough to be worth much consideration.

590. **Quality of the Material.**—The material used for steel pipes should be soft open-hearth steel of a tensile strength of about 60,000 pounds per square inch, elastic limit one-half of ultimate strength, elongation 22 to 25 per cent, and reduction of area 50 per cent. This material is about the same as now used in the best stand-pipe construction. A good quality of material is required to resist the shocks to which pipe-lines are often subjected, and to withstand safely the work of forging, punching, and calking.

591. **Thickness of Shell.**—If s = allowable stress per square inch on gross section, the required thickness is given by the equation

$$t = \frac{pr}{s},$$

where p = total pressure per square inch, including allowance for water-hammer, and
r = radius of pipe in inches.

The value of s depends upon the method of construction. If the pipe is a riveted pipe, the longitudinal joints are usually double-riveted, and as such have an efficiency of from 60 to 70 per cent. If water-hammer is properly taken into account, the safe stress on net area may be taken at about 15,000 pounds per square inch, whence the stress on gross area will be about 10,000 pounds per square inch, which would be the value of s to be used in the preceding equation. For very large pipes triple-riveted joints may be employed, in which case the efficiency will be about 75 per cent (see also Art. 593).

In order to equalize somewhat the life of pipes of various sizes, and at the same time to prolong it, an allowance of a small amount, such as $\frac{1}{16}$ inch, may well be added to the thickness determined from the formula.

592. Joints.—Small sizes of pipe may be made by means of the lap-welded joint, or the spirally-riveted joint, or the longitudinal lap-riveted joint. Such pipes are made in sections of 12 or 15 feet which are connected in the field in various ways, such as by a screw-coupling, or by means of a cast-iron bell and a spigot consisting of a steel or wrought-iron band, or by riveting, or by merely driving the sections together. For large sizes riveted longitudinal and circular joints are usually employed. Single sheets are bent and riveted to form one section of pipe, which may be made either cylindrical in form, or made with a slight taper and the sections put together stove-pipe fashion. Lap-joints have been commonly used, but this form of joint offers considerable obstruction to the flow of water, so that in some of the later pipes butt-joints have been adopted, and it has been proposed also to employ countersunk rivets. The value of butt-joints and countersunk rivets would be proportionally greater the thicker the plates. Whether they would be economical would depend on the extra cost involved as compared with the saving effected by the reduction in diameter rendered possible.

In the construction of steel pipes several sections are riveted together at the shop, usually enough to make a length of 20 to 30 feet. These sections are then transported to the field and riveted together in place. Special forms of joints, such as described for small pipes, are also sometimes used for large pipes, but probably the safest joint for the circular seams is the well-calked riveted joint.

Riveted joints, both in the shop and in the field, should be thoroughly calked and tested by water-pressure.

593. *Design of the Riveting.*—The design of the riveting follows the same general principles as employed elsewhere, and as more fully discussed in the chapter on stand-pipe design, Art. 720. The size of rivets is usually made about twice the plate thickness up to a maximum diameter of about $1\frac{1}{8}$ inches. The circular joints can usually be made strong enough by single riveting, but economy requires the longitudinal joints to be double- or triple-riveted. For any given size of rivet the spacing is determined by making the shearing strength of the rivet equal to the tensile strength on net section; and this strength divided

by the strength on gross section is the efficiency of the joint. The safe shearing strength of rivets may be taken at about 9000 pounds per square inch. The rivet-spacing for the East Jersey pipe-line was, for example, as follows:

	Inches.	Inches.	Inches.	Inches.
Nominal size of pipe	48	48	48	36
Thickness of sheets	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{1}{4}$
Size of rivets	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	$\frac{5}{8}$
Circular Seams.				
Rivet-pitch	1.5	1.8	2.0	1.5
Lap of sheets	2	$2\frac{3}{8}$	$2\frac{3}{4}$	2
Longitudinal Seams (double-riveted).				
Rivet-pitch	2.277	2.721	3.125	2.277
Distance between rows	$1\frac{1}{16}$	$1\frac{3}{16}$	$1\frac{5}{16}$	$1\frac{1}{16}$
Lap of sheets	3	$3\frac{1}{2}$	4	3

In the 72-inch steel pipe at Ogden, Utah, the longitudinal joints were double-strap butt-joints, triple-riveted, similar to the joints used in marine-boiler practice. The circular joints were double-riveted single-strap butt-joints. The calculated efficiency of longitudinal joints is from 85 to 87 per cent. The stress on net section varies from 13,000 to 14,000 pounds per square inch. The pipe was made in sections 9 feet 2 inches long, each section consisting of a single plate. The field-joints were power-riveted.*

Steel pipe-lines are usually built without expansion-joints. Changes of temperature, therefore, produce certain longitudinal stresses which must be considered in designing the circular joints and in making connections at valves and other points. The stress per square inch on gross section due to temperature changes is given by the formula of Art. 577, page 555. For steel, $c =$ about .0000065 and $E = 30,000,000$ pounds per square inch. If the pipe is buried, the range of temperature will not exceed 40 to 45 degrees, so that, assuming the pipe laid at a temperature equal to the maximum, the greatest stress caused by a reduction of temperature will be

$$s = 45 \times .0000065 \times 30,000,000$$
$$= 8800 \text{ pounds per square inch on gross area.}$$

Considering the self-adjustments which will take place during the con-

* Trans. Am. Soc. C. E., 1897, XXXVIII. p. 258.

struction of the pipe, the stress caused by temperature changes will doubtless be considerably less than that here computed.

If the pipes are exposed for any great distance, expansion-joints become necessary, for the consideration of which reference is made to the next chapter.

594. *The Locking-bar Joint.*—A novel form of longitudinal joint which appears to have much merit is what is known as the locking-bar joint, used recently on some Australian pipe-lines (Fig. 146). In

FIG. 146.—THE LOCKING-BAR JOINT.

making this joint the plates are slightly upset at the edges, then inserted in the grooves of the bar and the bar pressed down upon the plates in an hydraulic press. The pipes are then tested, and if found leaky the joints are usually corrected by additional work in the press. No calking is required. This form of joint has proven cheaper than the riveted joint, and where it has been used specifications have required its efficiency to be as great as that of the plates. Tests by Prof. Unwin have shown that this requirement can be readily met. This joint possesses a very considerable advantage over the riveted joint in that it forms no obstruction to the flow of water beyond causing a slight reduction in the cross-section of the pipe. The figure also illustrates the joint-rings for the circular joints. These joints are made with lead as for cast-iron pipes.*

595. **Special Details.**—Changes in direction are usually made by forming one or more joints at a small bevel. Two or three standard bevels of small angle may be adopted, and any desired curve made by the use of one or more of these bevels. Branches, etc., for the ordinary sizes of pipes, are usually made of cast iron and are riveted or bolted firmly to the steel pipe. Valves are joined to the pipe in a similar manner by means of cast-iron flanges. In connecting large mains Mr. Herschel has adopted the plan of using several small cast-iron flanged connections, an arrangement which allows the use of small castings and small valves. Riveted specials are sometimes used for large pipes.

596. **Coating of Steel Pipe.**—The tar coating employed for cast-iron pipe is not so successful when applied to steel or wrought iron, but the

* *Eng. News*, 1898, XXXIX. p. 373; XL. p. 423. *Eng. Record*, 1900, XLI. p. 178. See also use of this joint at Lynchburg, Va., *Eng. Record*, 1906, LIV. p. 228.

necessity of a perfect coating is even greater in this case on account of the comparative thinness of the metal. Some form of asphalt coating has usually been employed. The ordinary method of applying the coating is to dip the pipe in liquid refined asphalt heated to a temperature of 280 to 350 degrees, as in the coal-tar process. A second dipping to thicken the coat is often used. Various mixtures of asphalt and tar have been tried, but with no better results than with the pure asphalt.

A process of applying asphalt varnish, known as the Sabin process, has been used in some recently constructed pipe-lines with apparently more success than has accompanied the old method. In this process, the pipe, after dipping, is baked for several hours at a temperature of 400 to 600 degrees, thus producing an enamel coating.

In all cases the pipe must be thoroughly cleaned before coating. The specifications for the Coolgardie pipe-line require the pipe to be cleaned by being dipped, first in dilute sulfuric acid, and then in a bath of lime-water. The coating consists of Trinidad asphalt and creosote. The coating of the Bundalier pipe-line is composed of equal parts of asphalt and coal-tar.

Still another form of coating recently used is known as "mineral rubber" asphalt. It is composed of asphalt, but the process is secret. The pipe is dipped but not baked. This process was adopted at Minneapolis in 1897, and has been used in several other important works. The results so far appear to be quite promising.

In transportation and in construction in the field great care must be exercised to avoid injuring the coating. Some protective covering of pieces of old carpet or canvas should be used, and the workmen required to wear rubber shoes. The field-joints and all places where the coating has been injured should be coated by applying with a brush some kind of protective paint. Asphalt dissolved in carbon bisulfide (P. & B. paint) is often used, but the fumes from it are very objectionable to the workmen. Various other asphalt paints or varnishes are also used for this purpose.

597. Durability of Steel Pipe.—Steel pipe coated with asphalt has in some cases been reported to be in perfect condition after a lapse of thirty or forty years. In other cases corrosion has been quite rapid, so that the life of the pipe has been short as compared to that of cast iron. The Rochester wrought-iron pipe built in 1873–5 has in twenty-one years required a little repairing by means of patches placed on the outside. During this time eight plates out of a total of 14,000 have been thus repaired. No leaks have developed at riveted joints. The Rochester pipe was coated with Trinidad asphalt and coal-tar. Mr. Freeman in estimating the cost of steel pipe-lines for New York City assumes their life at fifty years, but provides for their cleaning and painting about every ten years.*

WOODEN PIPE.

598. **Advantages of Wooden Pipe.**—It was noted in the introduction that the use of wood for water-mains was quite universal in the early days of water-works construction, and that this material was subsequently displaced by cast iron. The use of wooden pipe under certain conditions has now again reached considerable proportions in certain parts of the country.

In general, wooden pipe is practically adapted to those locations where transportation of iron is very expensive and where wood is relatively cheap. When properly constructed, wooden pipe is very durable; it is not subject to corrosion by electrolysis nor affected by changes of temperature, and it also furnishes good protection to the water against cold and heat in exposed locations. It possesses another very considerable advantage in the smoothness of the interior and in the fact that the capacity does not become reduced through corrosion. The special field for wooden pipe is for low pressures and moderate sizes, where a metal pipe, if used, would necessarily have excessive strength.

599. **Bored Pipe.**—The manufacture of bored pipe for water-mains has been somewhat revived in recent years, and a considerable amount of such pipe is now manufactured under the name of "improved Wyckoff pipe." The pipe is made from solid logs, but it depends for strength upon spiral bands of flat iron which are wound tightly about it from end to end. The exterior of the pipe is coated with pitch as a protection to the bands. The joints are made by means of wooden thimbles fitting tightly in mortises in the ends of the pipe, and, in laying, the sections are driven together by means of a wooden ram. The interior surface is smooth and continuous. The pipe is made in sections 8 feet long, and in sizes from 2 to 17 inches in diameter. The bands are spaced according to the pressure. Branch connections are made by means of cast-iron specials which have long sockets into which the wooden pipe is driven. A considerable amount of this pipe has been used in recent years. It is very durable and is said to cost some-

* Report on New York's Water-supply, 1900, p. 320.

what less than cast iron where the transportation charges are not excessive.

600. Stave-pipe.—The necessities of water-carriage in the West, and the expense of iron pipe in that region, have developed a very efficient form of wooden-stave pipe. As early as 1874 Mr. J. T. Fanning built such a pipe at Manchester, N. H., which is still in use, but the chief development of this type of construction has taken place in the West since 1883, at which time stave-pipe was first extensively used at Denver.

The pipe is built continuously in the trench. The staves are formed with radial edges, and are bound tightly together by means of round or oval bands of steel or iron, spaced and sized according to the pressure, and fastened by shoes and nuts. The general form of construction and method of building will be clearly understood by reference to Figs. 147 and 148. The latter illustration shows also the method of carrying a wooden pipe across a narrow gorge.

Two types of stave-pipe have been employed. In one of these, the Allen patent, the outside and inside surfaces of the staves are made concentric. The staves are made to break joints, and the end joints are made tight by inserting small steel plates in saw-kerfs in the staves. In the other form, the Durelle patent, polygonal staves 16 to 20 feet long are used which have a slight tongue and groove formed on the edges. The staves do not break joints, but the end joint is made by surrounding the pipe by a layer of staves 4 feet long. The former type has been most frequently used.

Stave-pipe is suited to pressures up to about 100 pounds per square inch. Above this limit it will usually be less economical than steel, as the bands become very heavy and numerous. Stave-pipe has been constructed in sizes from 1 foot up to 9 feet in diameter.*

601. General Requirements for Staves and Bands.—The staves should be made of clear stuff and be somewhat seasoned. California redwood and Oregon fir have been most frequently employed. The staves should be thick enough to prevent percolation and not deflect appreciably between bands. In practice the size varies from about 1 inch by 4 inches to 2½ by 8 inches.

Bands should be made of a good quality of soft steel, and should be upset for the sake of economy. They should be thoroughly coated with asphalt before being used. They must be of such a size and so spaced as to withstand the stresses to which they are subjected, prevent

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* For a full and valuable discussion of the design and construction of stave-pipe see paper by A. L. Adams in Trans. Am. Soc. C. E., 1899, XLI. p. 27.

FIG. 147.—WOODEN-STAVE PIPE.

FIG. 148.—STAVE-PIPE, SANTA ANA CANAL.
(From Trans. Am. Soc. C. E., vol. XXXIII.)

flexure of the staves sufficient to cause leakage, and not injuriously crush the fibers of the wood. To resist the water-pressure large bands and wide spacing will in general be most economical, but the size is limited by the requirement that the band must not crush the wood when fully stressed, and the spacing must not exceed a certain maximum.

In practice the thickness of staves to give durability and prevent percolation will allow a maximum spacing of 10 to 12 inches under light pressures. Under any considerable pressure, other requirements will govern the size of bands and the spacing.

602. Size of Bands.—As the size of a band increases, its strength increases as the square of the diameter, while the safe pressure upon the wood increases only as the first power of the diameter, so that for each case a definite limit exists for the size of band which may be used. Experience shows that the width of contact of round bands with the wood, when the latter is compressed within safe limits, is about equal to the radius of the band. The ultimate crushing strength of the wood is from 1000 to 2000 pounds per square inch, and the safe stress is usually taken at from 600 to 750 pounds. Mr. Adams, in the paper referred to on page 518, uses a value of 650. Adopting this figure and letting r = radius of band, and e = safe pressure per lineal inch of band, we have $e = 650r$. Further, let S = safe strength of band, R = internal radius of pipe, and t = thickness of pipe; then

$$S = (R + t)e = (R + t)650r, \quad \ldots \ldots \quad (5)$$

from which equation the size of band is determined. A factor of safety of about 4 is usually employed.

In case the calculated size of band will, by the formula for spacing given in the next article, correspond to a spacing greater than 10 or 12 inches, then the spacing should be assumed at the maximum allowable value and the size of band calculated by eq. (6). This will occur for light pressures only. Bands less than $\frac{3}{8}$ inch should not be used.

From these considerations Mr. Adams has made up a table, reproduced in Table No. 76, which gives a suitable size of stave and the maximum size of band for different diameters of pipe, using a factor of safety of about 4 for the bands. Oval bands are assumed for pipes 20 inches in diameter or less, in order to secure a greater proportionate area of contact. For the 10-, 12-, 22-, and 30-inch pipes the bands used cannot be stressed to their full working value without crushing the wood. The permissible working stress given is such as will give a value of e equal to $650r$.

603. Spacing of Bands.—The size being determined, the spacing will depend upon the stresses. These are from three sources:

1. The initial tension.
2. The stress due to water-pressure.
3. The stress due to the swelling of the wood.

TABLE NO. 76.

ECONOMIC PROPORTIONS FOR STAVE-PIPE DESIGN (ADAMS).

Nominal Diameter of Pipe. Inches.	Stock Sizes for Staves.	Thickness of Finished Staves.	Economic Sizes of Bands.	Working Stress in Band. S. Pounds.	Factor of Safety in Band.
			Oval.		
10	$1\frac{1}{2}'' \times 4''$	$1\frac{1}{16}''$	$\frac{5}{16}'' \times \frac{7}{16}''$	1255	5.26
12	$1\frac{1}{2} \times 4$	$1\frac{1}{8}$	$\frac{5}{16} \times \frac{7}{16}$	1475	4.47
14	$1\frac{1}{2} \times 4$	$1\frac{3}{16}$	$\frac{5}{16} \times \frac{7}{16}$	1650	4
16	2×6	$1\frac{7}{32}$	$\frac{5}{16} \times \frac{7}{16}$	1650	4
18	2×6	$1\frac{3}{8}$	$\frac{5}{16} \times \frac{7}{16}$	1650	4
20	2×6	$1\frac{3}{8}$	$\frac{5}{16} \times \frac{7}{16}$	1650	4
			Circular.		
22	2×6	$1\frac{3}{8}$	$\frac{3}{8}$	1508	4.4
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
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

If, after a pipe is filled with water, the bands be loosened until the water begins to percolate through the cracks, the stress will then be due to (2) only, but this condition is impracticable of attainment. In actual practice the staves are more or less seasoned and the bands screwed up tightly at first. The wood will readily swell 2 or 3 per cent, which is an amount far beyond the capacity of the bands to allow by virtue of their elasticity and their sinking into the wood; so that the total force on the bands is approximately equal to the swelling-power of the wood (crushing-strength of saturated wood) plus the water-pressure. The swelling-power of the staves appears, from experiments by Mr. A. C. Henny,* to vary from 50 to 200 pounds or more per square inch,—ordinarily from 75 to 150 pounds. Adams assumes 100 pounds, and this is probably a sufficiently high value.

To determine the spacing we have then, if d = spacing of bands in

* Trans. Am. Soc. C. E., 1899, vol. XLI. p. 76.

inches, $p =$ water-pressure per square inch, and $e' =$ swelling-force of wood per square inch, with other notation as on page 523,

$$S = pdR + e'td,$$

whence $$d = \frac{S}{pR + e't} = \frac{S}{pR + 100t}. \quad . \quad . \quad . \quad . \quad . \quad (6)$$

In this formula, S is the safe strength of the band as determined by the application of eq. (5). The size of the band and its working stress may also be taken from Table No. 76. If the spacing as found from eq. (6) is greater than the maximum allowable, then d should be assumed, the value of S computed, and the size of band selected accordingly.

For large sizes and high pressures the term $e't$ is relatively small and a formula for spacing based on water-pressure alone, namely, $d = \frac{S}{pR}$, is sufficiently accurate, and is used by some engineers. For small sizes and low pressures it is desirable, however, to take account of the swelling action.

It is to be noted that no account has been taken of initial tension. It has, however, been assumed that the stress on the band is caused by full water-pressure plus the swelling-power of the staves, and this is the maximum force which can act upon the bands.

604. Coupling-shoes.—The coupling of the bands is made by means of a malleable-iron or steel shoe closely fitting the pipe, and of a strength equal to that of the bands. The design of this shoe is a matter of considerable importance and some difficulty. It should be so made as to strain the bands axially, it should have a good bearing on the staves so as not to cause undue pressure, and it should be convenient and made as light as possible, consistent with strength. Two forms of shoes are illustrated in Fig. 149. The first is of malleable cast iron, while the second is a forged shoe. The first requires a forged head on the band, and the second a loop-eye. The pressure between shoe and pipe is quite uniform in both these forms, which is not true of some that have been used. Other forms are illustrated in Mr. Adams's paper.

605. Specials.—Stave-pipe can readily be built to a curvature of from 200 to 300 feet radius by springing the staves into place. Connections are usually made by means of castings with deep bells, into which the pipe is built and calked with oakum and paint. Variations in diameter are made by the use of tapered staves. Repairs can very readily be made in this kind of pipe.

606. Leakage and Durability of Wooden Pipe.—Tests of pipe-lines have shown in some cases practically no leakage. In others, a slight leakage has been observed which, including evaporation from the

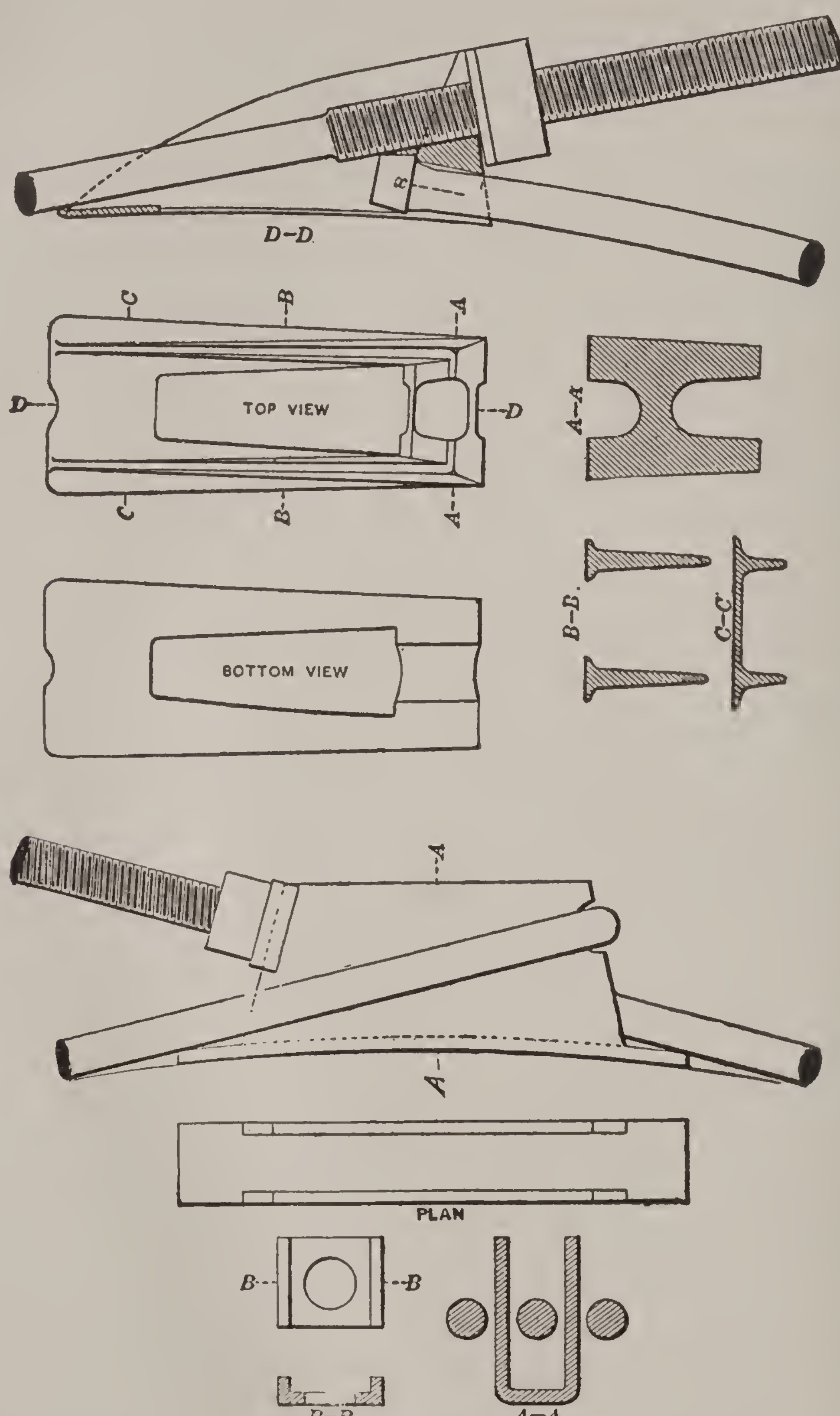

FIG. 149.—COUPLING-SHOES FOR WOOD-STAVE PIPE.
(From Trans. Am. Soc. C. E., vol. XLI.)

surface, has amounted to from .053 to .086 gallon per square foot per day. Mr. Henny considers that a leakage of .05 gallon per square foot per day is a safe allowance for exposed pipes.

The durability of wooden pipe varies greatly under different conditions. Where the pipe is constantly in service and under a considerable pressure the wood is generally kept sufficiently saturated to prevent decay. Many old wooden water mains in various cities have been found perfectly sound after sixty or seventy years of use. The conditions are, however, not so favorable for the usual wooden stave conduit, as the pressures are generally not great and the durability of some such pipe lines has been much less than expected. To some extent this seems to be due to the collection of air along the upper portion of the pipe, thus permitting the wood to become partially dry. The quality of the material is also of much significance. Wood is particularly advantageous where salt water is encountered. The steel bands are subject to some corrosion, but the form of cross-section is favorable and the relative deterioration is generally quite slow.

OTHER MATERIALS EMPLOYED FOR WATER-PIPE.

607. **Cement Pipe.** — Pipe made by lining a core of wrought iron inside and out with cement mortar has been much used in the eastern part of the United States, but is now employed in very few places. It is still reported to give satisfactory service in some cases, but it has generally been abandoned for cast iron. There is great difficulty in maintaining the cement coating intact, and if it is broken the iron core soon corrodes. Its life is in many places not over ten years.

For large conduits, reinforced concrete may often be employed to advantage. It has also been used to some extent for pressure pipes, especially in France, where it has been successfully employed for pressures up to 300 feet. In this country it has been tried only to a limited extent and without great success.* The use of reinforced concrete for conduits is further discussed in the next chapter.

608. **Vitrified-clay Pipe** has been employed in a few places for conduits. It is cheap, indestructible, and when the joints are carefully made the leakage is very small. It is generally used under no pressure, but in one or two instances has been designed to carry considerable pressures. Vitrified pipe has recently been recommended, in a report to the city of Oakland, for salt-water mains to furnish water for street-sprinkling.† It was thought it could easily be made to withstand 100 pounds pressure. The form of joint was to be of strips of burlap dipped in asphalt, which form has been well tested under 40 to 50 pounds pressure. At Florence, Colorado, a 7-mile conduit of 12-inch vitrified pipe has been built. It is not under pressure. Vitrified pipe has also been extensively used at Little Falls and at Amsterdam, N. Y.,

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* See paper by C. W. Smith, Proc. Am. Soc. C. E., August, 1907, p. 581. Also *Eng. News*, 1898, XXXIX. p. 170. † *Eng. News*, 1899, XLII. p. 149.

a length of 5.63 miles having been constructed at the former place. A very considerable saving was thus secured. Deep sockets were used and the joints carefully made by means of jute soaked in Portland cement, with which material the joint was thoroughly filled to within $\frac{1}{2}$ inch of the outside. The remainder of the space was filled with Portland-cement mortar. The cost of the Little Falls conduit of 12- to 20-inch pipe was about $1.50 per foot, which was about one-half the cost of cast-iron pipe.

609. **Materials for Service-pipes.**—Service-pipes, or pipes for conducting water to individual consumers, are made of a considerable variety of materials. Uncoated iron pipe, or pipe coated only with tar, is not serviceable for such small sizes (usually $\frac{3}{4}$ to $1\frac{1}{2}$ inches in diameter), as even a small amount of tuberculation would completely clog up the pipe. Galvanized, tin-lined, lead-lined, and cement-lined iron pipe are widely used, but the most common is lead pipe. Lead pipe is practically indestructible, but rather expensive and heavy for high pressures. In some places it cannot be used with safety on account of the danger of lead poisoning. Certain waters only will attack lead to a sufficient extent to render its use dangerous, but, despite the study that has been put upon the subject, it is not yet fully known, without actual experiment, what effect various classes of waters will have.

Recently the Massachusetts Board of Health has investigated this question by reason of several cases of lead-poisoning that have occurred in that State.* Thirty cases were especially studied in which lead pipe was largely used. In general it was found that waters having the greatest amount of dissolved solids and hardness dissolve the least amount of lead, and that the active agents causing the solution of the lead are oxygen and carbonic acid. The latter is characteristic of soft waters. In fifteen towns with ground-water supplies the average amount of lead ranged from .0055 to .1899 part per 100,000 with pipes in ordinary use, and from .0108 to 8.38 parts when the water had stood in the pipes. Surface-waters in fourteen towns averaged similarly from .0031 to .0788, and from .0099 to .3921 parts respectively. In four cities with ground-water supplies, cases of lead-poisoning were prevalent, and in these four cases the lead averaged 0.2 part, after the water had stood several hours in the pipes. No cases of poisoning occurred with surface supplies. Experiments on galvanized iron and plain iron showed more action than on lead, but with tin the corrosion was very little.

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* Report for 1898, p. 539.

As to the amount of lead which will give trouble it is known that continuous use of water containing .05 part per 100,000 has caused serious injury to the health. Zinc is not dangerous in the amounts likely to be present and galvanized iron pipe is much used.

Cement-lined pipe is quite largely used in the East, but in some places it does not prove to be very durable for the same reason as given in Art. 607. It has, however, given good service in many cities. Tin-lined pipe is now being used to some extent. It is quite expensive, but the experience with it so far indicates that it is very durable.

Statistics relating to the material employed in new services in Massachusetts cities and towns show that lead or lead-lined pipes are used in 26 cities and towns, cement-lined pipes in 43 places, galvanized iron pipes in 77 places and tin-lined pipes in 6 places. Much trouble has been reported from the rusting of galvanized iron.*

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CHAPTER XXV.

CONDUITS AND PIPE-LINES.

610. Where the source of supply is at a considerable distance from the place of consumption the design and construction of the necessary works for conducting the water is a matter of great importance and demands special consideration. Usually a distant source is at a higher elevation than the city to be served, so that it will be possible to convey the water partly or wholly by gravity. In many cases, however, a part or the whole of the water will require pumping, so that the design will also involve a study of possible pumping arrangements. It will usually be necessary to consider several designs based upon different locations and often upon different types of conduits. In determining upon the dimensions of a large conduit the utmost care should be taken in selecting the coefficient to be used in the hydraulic formulas employed.

611. **Classes of Conduits.**—Conduits are divided into two general classes: (1) those in which the water-surface is free and the conduit therefore not under pressure, and (2) those flowing under pressure. To the first class belong open canals, flumes, aqueducts, and usually tunnels, and to the latter belong pipe-lines of iron, steel, wood, or other material capable of resisting hydraulic pressure, and sometimes tunnels. Conduits of the first class must obviously be constructed with a slope equal to that designed for the water-surface, or equal to the hydraulic gradient. This will be a very light and uniform slope, and such conduits will therefore often require in their construction long detours to avoid hills and valleys, or resort must be had to high bridges, embankments, cuttings, or tunnels. Conduits of the second class may be constructed at any elevation below the hydraulic grade-line, but if built above they must be arranged to act as siphons. The selection of the form of conduit is principally a question of economy, and in this respect topography will largely govern, but consideration will also

be given to various advantages, such as are mentioned in the following discussion.

612. Capacity of Conduits.—Where the conduit is long, sufficient storage capacity is usually provided in the vicinity of the city to equalize the demand over several days or weeks, so that the capacity of the conduit may be based on the average monthly or seasonal consumption. The extent to which provision should be made for the future depends much upon the type of conduit. The question must be settled in accordance with the principles laid down in Chapter XI. In the case of a masonry conduit the expense of additional capacity is relatively small, so that it will be economical to provide for a long period in the future, such as thirty or forty years. Very often the capacity should be made equal to that of the watershed drawn upon. In the case of pipe conduits a much less liberal provision should be made for the future, as the expense of additional capacity is proportionately much greater.

613. Single or Double Conduits.—Security against the interruption of the supply demands either that there be two conduits, or that there be sufficient storage capacity at the city end to allow the shutting off of the supply to permit of any repairs which may be needed. The latter method will usually be the cheaper except for very short lines. The storage capacity necessary to allow of repairs will vary from three to four days' consumption in the case of small pipe-lines easy to repair, up to ten or fifteen days' supply for large aqueducts. The amount of storage considered necessary for this contingency should never be drawn upon for other purposes. If a pressure conduit is used and a double line is considered the most economical, or if the second line is built subsequently to provide added capacity, the two lines should be connected at frequent intervals. A section of either can then be shut off and the supply carried for a short distance in one pipe, which will result in but a small increase in the total head consumed or a small decrease in total flow. Where a single conduit is deemed most economical for the greater portion, it may still be advisable to build a double line at certain points where a breakage would be a very serious matter, as at river crossings, etc.

614. Location of Conduits.—The location of a conduit is a matter requiring much skill and judgment. It involves the question of available slope or hydraulic gradient, cost of conduits of different forms and sizes and built of different materials, and frequently the cost of pumping. In the matter of slope there may be sufficient to enable the water to be conducted entirely by gravity, or pumping may be required at one or more points.

In the case of conduits not under pressure, if the total head is closely limited, then the slope must be maintained nearly uniform and a location found, if possible, which will support the aqueduct at the desired elevation. A proper balance must be obtained between a circuitous route avoiding high crossings, and a more direct route which is more expensive per mile. Usually two or more possible routes will need to be examined in detail and comparative estimates made. If the available head is large, then a more economical location can probably be made, as the slope can be varied to a considerable extent in order to best fit the ground, and can also be made steep so as to give small sizes. If the country falls more rapidly than is permissible for the conduit, then the water may be let down at intervals in special forms of construction designed for the purpose.

The location of pressure conduits is comparatively simple. For them a more direct line can be adopted, but at the same time low pressures are to be desired. There should be as few summits and depressions as practicable, and small sags should be avoided. To provide opportunity for easy regulation of the pressure it is desirable that the conduit approach close to the hydraulic grade-line at occasional intervals, as more fully explained in Art. 629.

Tunnels may be constructed at the grade-line and hence flow free, or they may be built at a lower elevation and flow under pressure. Usually the former will give the shorter and cheaper tunnel, but in some cases it is expedient to build tunnels at a greater depth, as in the Croton aqueduct, where 7 miles of tunnel is under a pressure of about 55 pounds, the chief purpose being to avoid interference with valuable property.

Long conduits usually include both masonary aqueducts and pipe-lines, each class being used where most suitable. The former is used as a rule where the ground lies near or above the hydraulic grade-line, and the latter where it lies below for any considerable distance. High and long aqueduct bridges are no longer built, a pressure conduit being substituted, which may follow the ground-profile closely. However, as the transition from open to pressure conduits involves some additional details, it will often be cheaper to support the former on bridges where the height is but moderate. Where pumping is required the expense of raising water must be considered in fixing upon the size and slope of the conduit. In the case of a pipe conduit the slope or head consumed involves only the question of size, and the proper size to make the total cost a minimum can be quite easily determined, as shown in Art. 632. With an open conduit, however, the topography will very largely determine the slope and therefore the size.

The proper location of a conduit requires full and careful surveys, including numerous borings and test-pits to determine the character of the material. The maps of the United States and various State geological surveys are of the utmost help in this connection.

CANALS.

615. **Use of Canals.**—The open canal is not often used for conveying water for city use, but for irrigation purposes it is the common form of conduit. For the former purpose it has several objections, such as loss of water by percolation and evaporation, exposure of water to pollution from surface drainage and otherwise, and exposure to summer heat, which not only warms the water but promotes vegetable growth. In irrigation-canals the seepage often amounts to 1 or 2 vertical feet per day, an amount which would scarcely be permissible in a city water-works conduit, where a large expense has been put upon storage-reservoirs, and, as is often the case, where the total capacity of the watershed is nearly reached. However, where a canal can be constructed with little cutting or embankment, and where the material is nearly impervious, it may be the best form of construction.

If the material is porous, it would probably be better to adopt the covered masonry conduit than to incur a large expense in constructing a puddle or concrete lining to a canal. A very favorable location for an open canal is where a stream-bed can be made into a canal to carry water from one reservoir to another lower down the valley. In this case there will usually be no loss by seepage, but rather a gain by infiltration of ground-water. In side-hill work, or in country of any difficulty, the masonry aqueduct will likely be the cheaper form of construction. For very large quantities of water the economy of canals will be more pronounced. In considering the adoption of a canal the possible pollution of the water should be carefully considered.

616. **Slopes and Velocities.**—If the available head permits, the most economical slope will be such as will give the maximum permissible velocity for the material and therefore the minimum cross-section. Topography may require the use of a much less slope than this, but a greater slope cannot be used without danger of erosion, or an increased cost in protection by paving or otherwise. If the slope of the ground is too great, then the fall may be concentrated at certain points

where special precautions are taken. With a small available head the velocity will be low and the section will have to be made large to correspond with the low velocity.

The allowable velocities for unprotected canals vary from about $1\frac{1}{2}$ to 2 feet average velocity for light sandy soils, $2\frac{1}{2}$ to 3 feet for ordinary firm soils, and 3 to 4 feet for hard clay and gravel. In rock or hardpan 5 to 6 feet may be allowed. A velocity of 2 to 3 feet per second is sufficient to prevent silt deposits and the growth of weeds.

The velocity and discharge for any given slope and cross-section is calculated from Kutter's formula. In using this formula the selection of a proper value of n is a matter of much uncertainty. For unlined channels it is usually taken at .020 to .025 (see Chapter XII). If vegetation is allowed to accumulate in the canal, a large allowance must be made for increased resistance caused thereby.

617. Cross-sections.—The cross-section of a canal is usually trapezoidal in form. For any given side slope the trapezoidal form giving the greatest hydraulic radius, and hence the most economical form as regards slope, is one in which the sides and bottom are tangent to a circle whose center is at the water-surface. The hydraulic mean radius of such a section is one-half the depth of the water. The side slope giving a maximum value for the radius is 60 degrees with the horizontal, and the water-section would be one-half a regular hexagon, but such a section could not be constructed except in very stable ground. For a rectangular section, or one with vertical walls, the width should be twice the height. The hydraulic radius of the rectangular section is $0.355\sqrt{A}$, and of the semihexagonal section is $.38\sqrt{A}$, where A is the area of cross-section. The section having the least water-surface is the triangle, and the best slope of the sides is 45 degrees.

The sections above described will give a minimum of excavation, but are suitable only for small canals. For large canals the material will be more economically handled if the section is made somewhat wider and shallower. Furthermore, if the canal is made partly by excavation and partly by embankment, and if the excavated material is suitable for embankment construction, the amount of excavation will decrease as the width of channel increases. Too wide and shallow a channel is, however, not desirable, as the velocity will be diminished, vegetable growth will be more troublesome, and the reduction of section due to ice will be proportionately greater. Very large canals are sometimes made ten to fifteen times as wide as deep. In side-hill work the amount of excavation will increase with increase in width beyond a certain point. Too deep a channel will also be unsuitable in such situations. Fig. 150 illustrates a section built almost entirely by

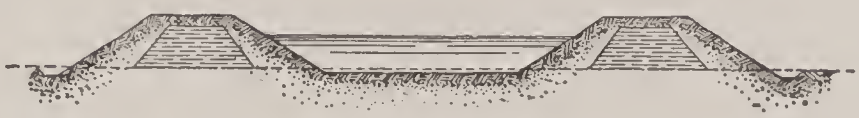

FIG. 150.—CANAL SECTION IN EMBANKMENT.

embankment. The best material is placed in the center of the embankments, and drainage-ditches for surface-water are provided. Fig. 151 illustrates suitable proportions for side-hill work in rock.

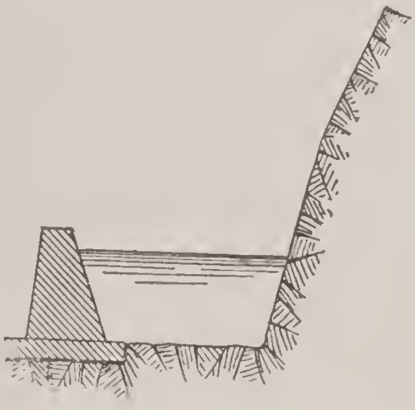

FIG. 151.—CANAL SECTION ON SIDE HILL.

The size of cross-section should be large enough to give the required capacity when the canal is covered with ice to the maximum thickness. If the velocity is low, a considerable allowance should also be made for growth of weeds and grass.

618. Other Details.—The construction of impervious banks follows the same general principles as laid down for reservoir construction. Side slopes in ordinary soils will vary from 1 to 1 for hard clay and gravel, to 3 to 1 or 4 to 1 for fine sand. The tops of the bank should be from 1 to 2 feet above the water-line. It is well to construct a berme just above the water-line, or at the original surface of the ground, above which the slopes of the bank may frequently be made steeper than below. If the soil is very porous, a lining of concrete or puddle may be necessary. Some canals have been lined with a layer of but 2 or 3 inches of concrete, placed on the earth and plastered with Portland-cement mortar. Fig. 152 illus-

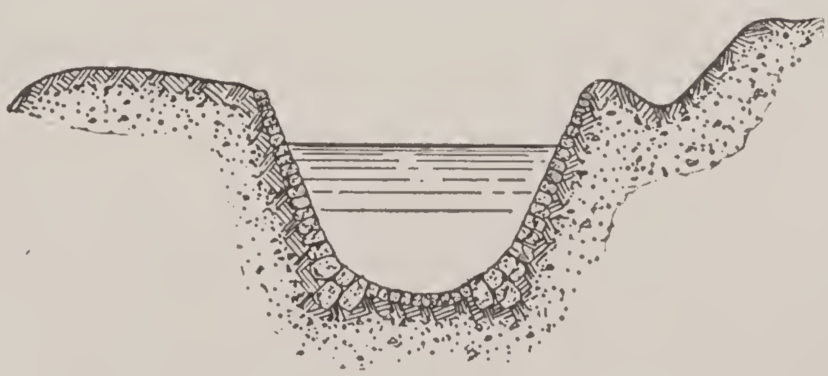

FIG. 152.—SANTA ANA LINED CANAL.

trates a form of section used on the Santa Ana Canal, California. If a very heavy lining is required, it will usually be better to build a covered masonry aqueduct, as this avoids trouble from ice and protects the water from pollution. The presence of clay and silt in the water will tend gradually to reduce percolation.

At sharp bends, and wherever the velocity exceeds the safe velocity for the material, some form of revetment is necessary. This may be merely a layer of gravel, or a paving laid dry or in cement, or a layer of concrete, according to the velocity of the water. If the general slope of the ground is too great for the canal, the fall may be concentrated at a few points by dams, below which the channel must be protected against scour, as described in Chapter XVII. On side-hill work a ditch should be constructed on the upper side to carry off surface drainage. The lower side of the canal at such places will often consist of a masonry wall as shown in Fig. 151.

Waste-weirs and sluice-gates should be provided at intervals along the canal to prevent flooding and to permit of rapid emptying. These wasteways should be located near some natural watercourse into which the waste-water can be conducted by suitable channels. The flow in the canal is regulated for the most part by sluice-gates at the head of the canal. These and other forms of canal gates are supported either by masonry walls, or by timber framework. Stop-planks fitting into grooves in the masonry are suitable for weirs, and for gates which are but seldom operated.

Canals are carried across valleys on trestles or bridges, or, in the case of short crossings, on embankments with a culvert or arched bridge beneath. Under-crossings are made by means of inverted siphons of pipe.

A recently excavated canal for water-supply purposes is one 15,800 feet long, carrying the water from the new Wachusett aqueduct of the Metropolitan Water-supply of Boston, down an old waterway to one of the old reservoirs. It is 20 feet wide on the bottom, with side slopes 3 to 1. To avoid too high a velocity two dams were built giving two moderate falls. Bermes at least 10 feet wide were constructed on each side, and the slopes above were made not steeper than 2 to 1. Where dug through fine sand the canal was faced with gravel or riprap.*

619. Flumes.—Where excavation for a canal is difficult, flumes of wood are often used for temporary works and for irrigation purposes on account of their low first cost. They are usually constructed with horizontal bottoms and vertical sides, but a more advantageous form, in which wooden staves are used for the lower portion, has recently been employed on the Santa Ana Canal in California.† A flume can be made much smaller than a canal on account of the high velocity of 6

* Report Mass. Board of Health, 1895, on the Metropolitan Water-supply.
† See Trans. Am. Soc. C. E., 1895, XXXIII. p. 61.

or 8 feet per second permissible. There is also much less resistance to flow, thus giving much less loss of head for like capacity.

MASONRY AQUEDUCTS.

620. Advantages of Masonry Aqueducts.—For conveying relatively large quantities of water over territory where the conduit can readily follow the hydraulic grade-line, the masonry conduit in cut and cover is a preferable form of construction. If properly constructed it is very durable, requires little attention, and if the topography is favorable it is much cheaper than large pipe conduits of iron or steel. Masonry is unsuited to withstand tensile stresses, hence it is not used to convey water under pressure. Combinations of steel and concrete may, however, be used for this purpose. Masonry conduits would not often be employed for cross-sections less than 10 or 15 square feet, for, unless the location be very favorable, their cost for such small sizes is likely to be greater than that of steel or iron pipes.

621. Size of Cross-section, Velocity, and Slope.—The size of cross-section, the velocity, and the slope are interdependent, one of the last two elements being usually the determining factor. The velocity should preferably be such as to prevent deposit of sediment, which requires $2\frac{1}{2}$ to 3 feet per second average rate; and for brick or concrete masonry it should not exceed 6 or 7 feet per second. Higher velocities may be allowed if stone masonry of hard material is employed, or if a lining of iron or steel is used. The question is usually determined by the available head between the terminal points of the conduit, or by the topography of the locality. If sufficient head is available, a smaller conduit will result if the velocity is made as large as the material will stand without danger of excessive wear.

The circular form of cross-section gives the greatest hydraulic mean radius and therefore the minimum area of section, but this form is not the most economical in construction. For large aqueducts the form which experience has shown to be the best is that illustrated in Figs. 157 and 158. The hydraulic radius of this section is but little less than that of a circular section.

Whatever the section adopted, the values of the hydraulic radius, velocity, and discharge for different depths of water should be tabulated for convenient use in computations. It will usually be the case that a conduit will flow only part full for the first few years, and the design should be made with reference to the fact that as the flow increases the velocity will increase. Kutter's formula is usually employed in calculations. The value of n to be used will vary with the character of the masonry about as given on page 256. The resistance to flow may be very greatly increased in a few years after the conduit has been put into use, by the formation of deposits or by organic growths. The capacity of the New Croton Aqueduct has diminished from such causes about 14 per cent in $9\frac{1}{2}$ years.

622. **Materials Employed.** — Up to about 1895 brick and rubble masonry were the materials generally employed for aqueduct construction, the lining and, frequently, the arch-crown being of brick. Concrete was first used in place of the rubble in foundations and side walls, brick being still used for the arch, or as a mere lining, as in the Massachusetts aqueduct illustrated in Fig. 158. In still later work concrete has almost entirely superseded other material, the brick lining being generally replaced by a lining of cement mortar, or no special lining at all being used. Reinforced concrete may be used to advantage in some cases, especially where the foundation is soft and where special forms of cross-section are required. In compact ground the advantage of reinforced concrete is, however, doubtful, as not much material can be saved by its use over that required in a properly proportioned plain concrete structure. The general change which has taken place in the past 15 or 20 years is well illustrated by the designs represented in Figs. 157–159a. In the use of concrete, Portland cement has almost entirely replaced natural cement for all purposes.*

623. **Form and Stability of Section.** — The forces to be considered in designing a section are the pressure of the water, the earth-pressure,

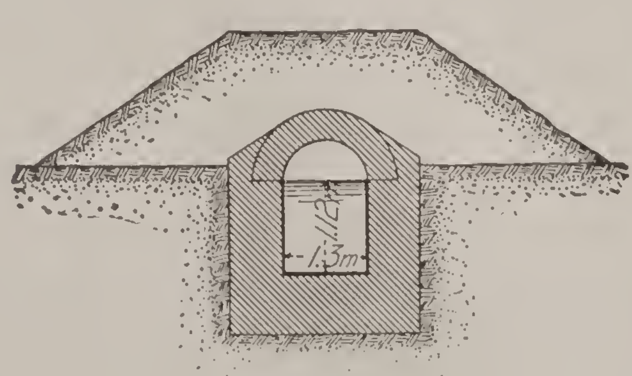

FIG. 153.
GALLERY, VIENNA WATER-WORKS.

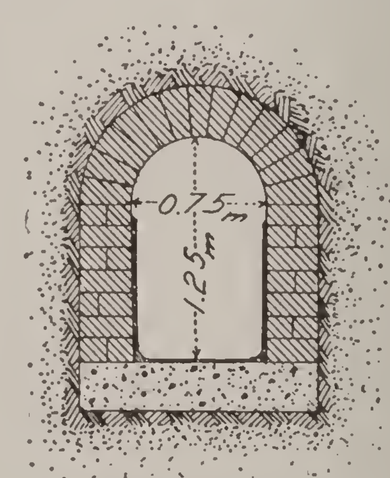

FIG. 154.
SMALL FRENCH AQUEDUCT.

and the weight of the masonry. Besides being of sufficient strength the section must be of convenient form for construction and inspection, and it must be economical. For small aqueducts a rectangular form

* See also Art. 759.

has often been used, as in Figs. 153 and 154, the cover being of stone slabs or of arches. The Manchester aqueduct is also of this general form (Fig. 155). Sharp angles are, however, objectionable, and these can be avoided and an increased capacity obtained at little cost by building the bottom as an inverted arch. Such a form is also better suited to resist any upward pressures. If there is lateral earth pressure, the sides will also be strengthened by curving them. These

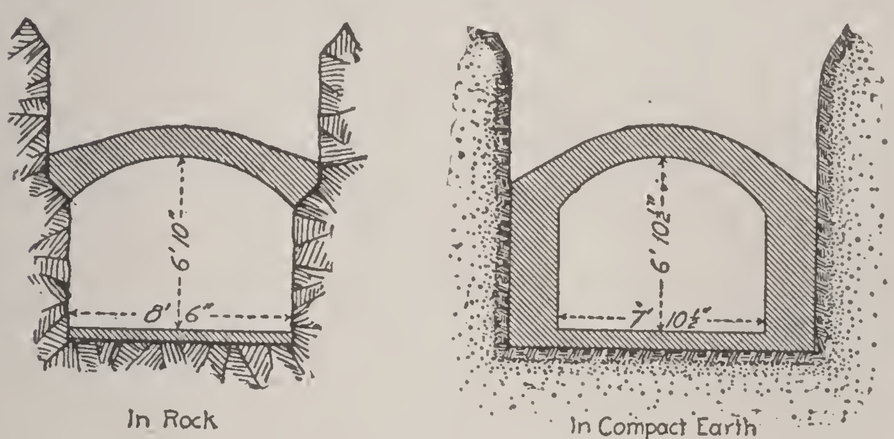

FIG. 155.
MANCHESTER AQUEDUCT.

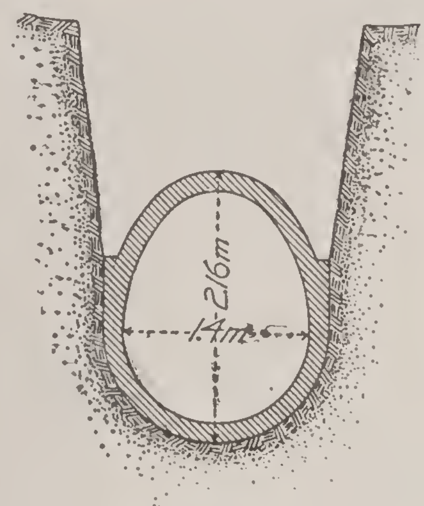

FIG. 156.
DHUIS AQUEDUCT, PARIS WATER-WORKS.

modifications give rise to the horseshoe shape as commonly used for large conduits. To make the bottom of short radius, giving a circular or elliptical section, is not so convenient in practical construction, although to give head-room in small conduits the elliptical or oval section has been used (Fig. 156).

In the case of small conduits built in compact earth, the water-pressure and the arch-thrust may be considered as largely resisted by the earth, but to insure this the back-filling up to the springing-line should be entirely of concrete (Figs. 155 and 158). In rock, a lining of one or two rings of brick, or of brick and concrete, is all that is necessary. In loose earth, and especially on embankments, the side walls should be heavy and have broad foundations. Little dependence can be placed upon the lateral thrust of the earth in an embankment, and experience indicates that in the settling of embankments there is a tendency for the walls to spread, and large longitudinal openings or cracks have been formed in aqueducts in this way. Inverts of reinforced concrete are very effective under such conditions. Several sections of modern aqueducts are shown in Figs. 157–159a. Fig. 159a illustrates a reinforced concrete design. Where not reinforced the arch ring must be of sufficient thickness to avoid tensile stresses. By carefully proportioning the side walls and arch with reference to the pressures acting, a comparatively small thickness of crown will be sufficient. In

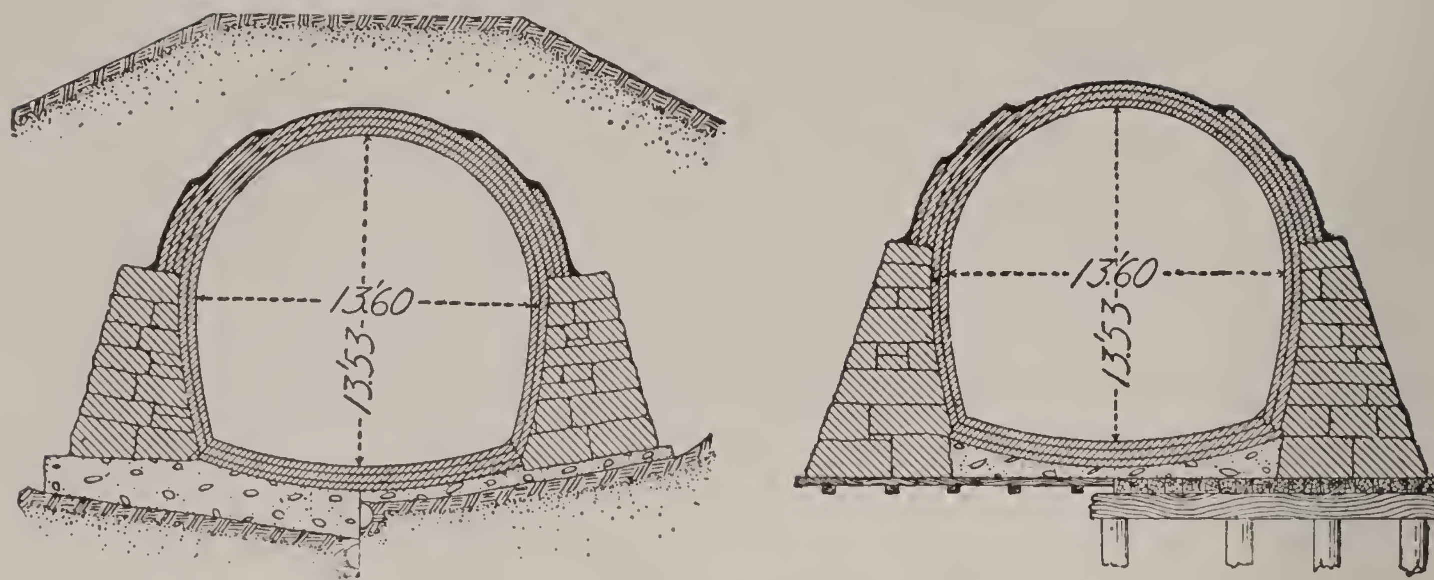

FIG. 157.—SECTIONS OF THE NEW CROTON AQUEDUCT, (1885).

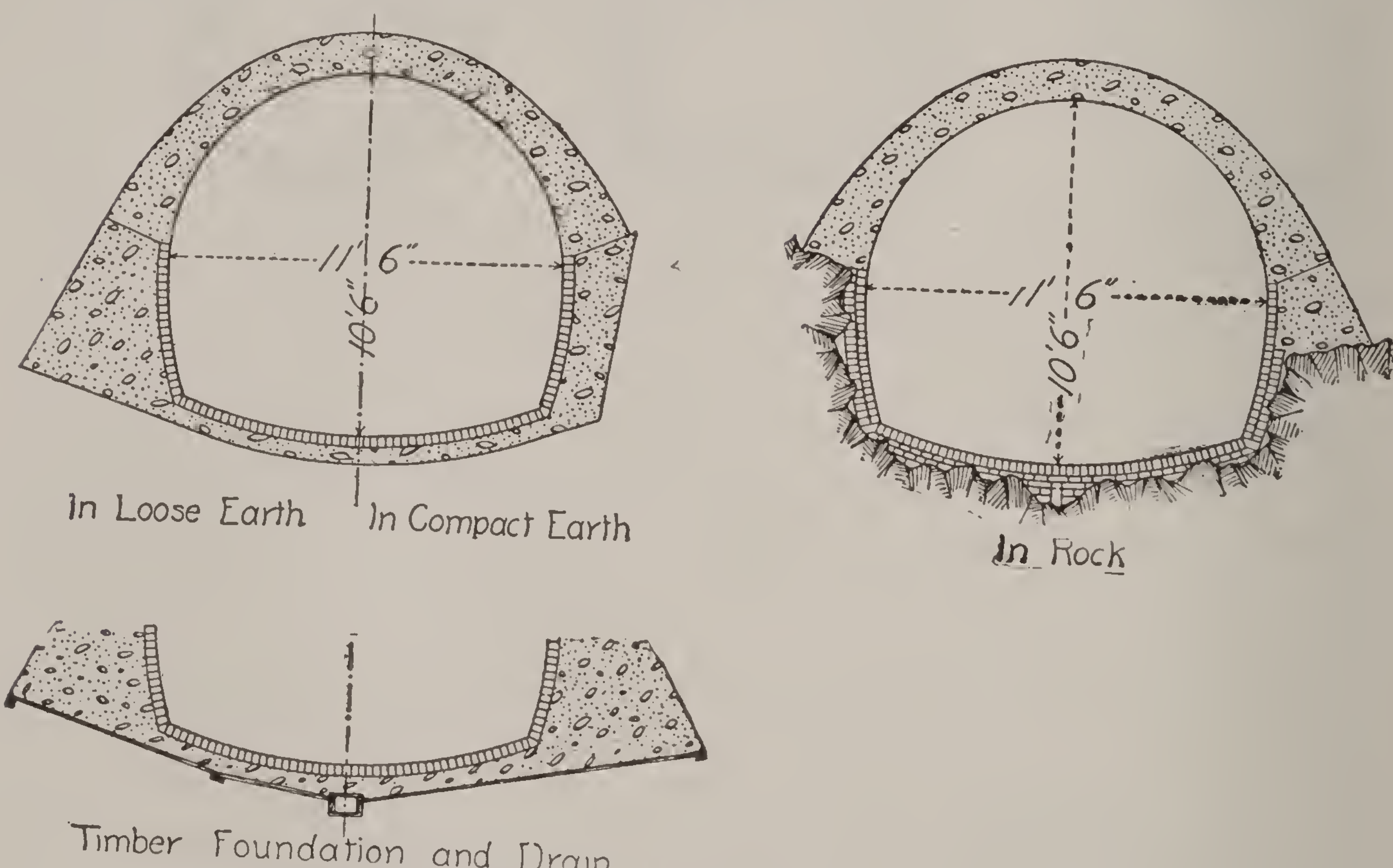

FIG. 158.—SECTIONS OF THE WACHUSETT AQUEDUCT, BOSTON, (1895).

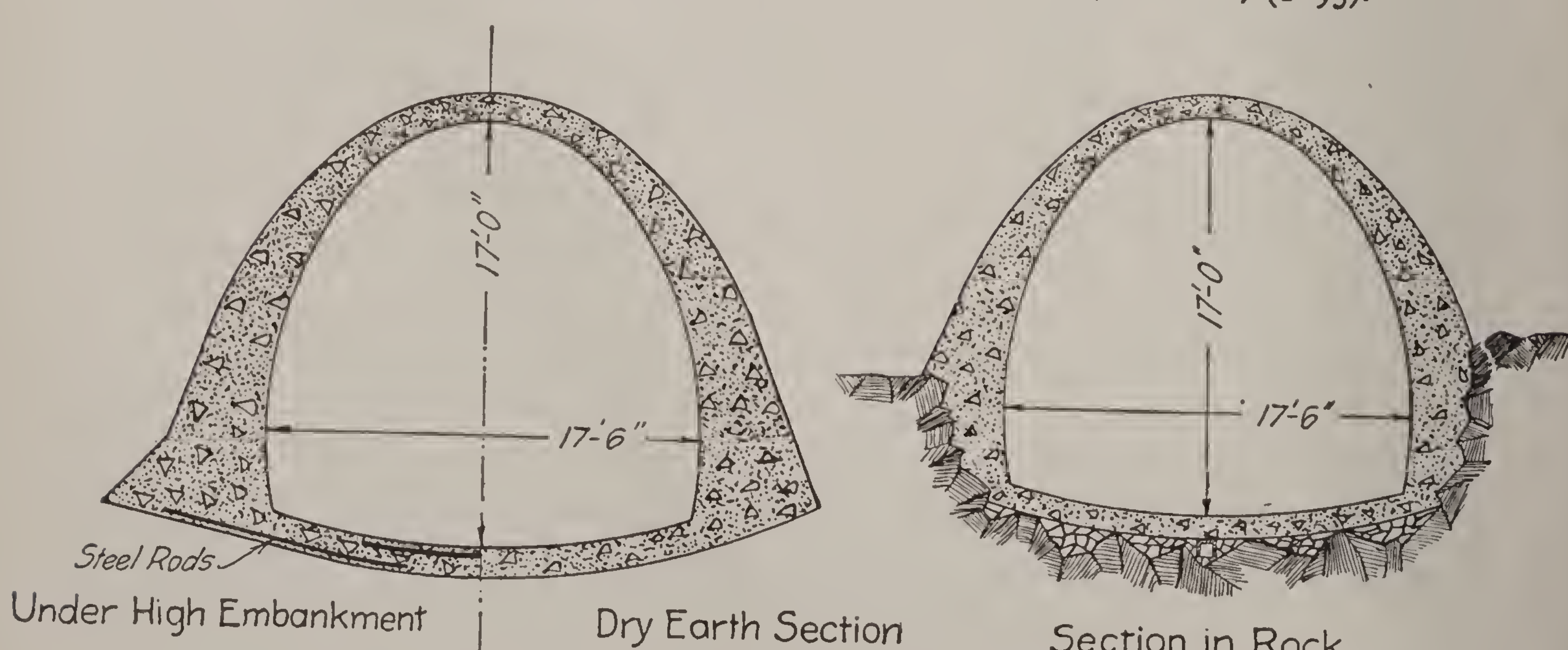

FIG. 159.—SECTIONS OF THE CATSKILL AQUEDUCT, (1906).

this respect the later designs are notably more economical than the earlier ones. (Compare Figs. 159 with 157 and 158.) If reinforced concrete is used a somewhat lighter section may be employed, especially near the base of the side walls.

The invert, in compact ground, is made only thick enough to secure a firm, impervious bottom. Even where the excavation is made through impervious rock, an invert of brick or concrete is desirable as giving a smoother bottom for cleaning and inspection and one offering less resistance to flow. In soft foundations, or on embankments, the invert should be made thick and strong, preferably of reinforced concrete, in order to be able to act as a beam and so aid in distributing the weight of the side walls. (See Fig. 159.) Timber or pile foundations may be required on soft soils. Settlement must be reduced to very low limits or cracks and leakage will result.

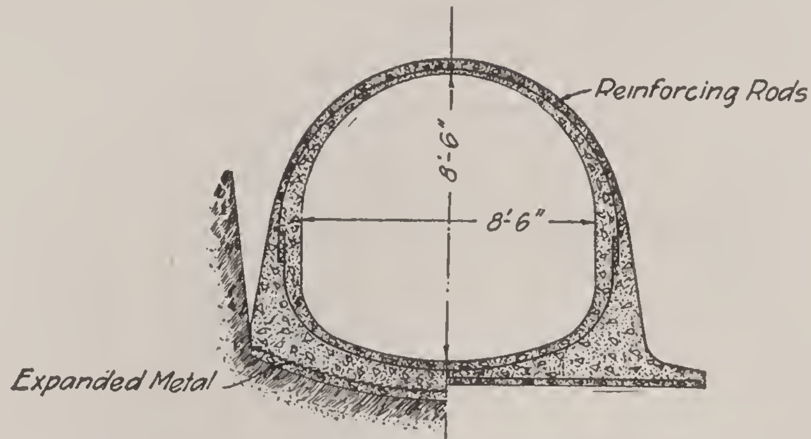

FIG. 159a. — JERSEY CITY CONDUIT.

624. Constructive Features. — It is unnecessary to state that in work of this kind the masonry must be constructed with the most careful supervision. In wet soils, drains should be built beneath the invert to enable the masonry to be laid without trouble from water. The drains may be led into the conduit at a point lower down and the water permitted to flow through the completed portion. Concrete and stone masonry should be given one or two finishing coats of thin, neat cement to secure imperviousness, the last coat to be finished as smooth as practicable. If carefully done, and no settlement occurs, the leakage will be slight. Successive sections of concrete construction should be connected by deep key joints and in order to permit some contraction without leakage, it is desirable to insert tongues of lead or plate iron every 50 or 75 feet.

Where built on embankment the greatest care must be used in constructing the earthwork in order to avoid settlement. The following is an extract from the specifications for the Wachusett aqueduct relating to embankment construction:

"The central portion of the bank, beneath the level of the highest part of the base of the aqueduct, to a width 8 feet greater than that of the base at such level, and to an added width of 1 foot for each foot below such level, is to be built with extreme care and with carefully selected earth; all stones larger than 2 inches in diameter are to be thrown out. The material is to be deposited and spread in horizontal layers not exceeding 3 inches in thickness, each layer to be sufficiently watered and very thoroughly rolled with a heavy grooved roller. From time to time during the construction of this portion of the embankment, and, if so required, three times after its completion, this portion shall be so thoroughly saturated with water that it will stand upon the surface. The building of the aqueduct upon such embankments shall not be begun until they have stood six weeks after completion, unless otherwise directed."

The requirements for the remainder of the earthwork were somewhat less severe. The embankments have in general a top width of 14 feet, with side slopes of $1\frac{3}{4}$ to 1. They start from a base from which all soil and all other perishable matter are removed, and on sloping ground the base is stepped. If founded on soft material, such material must be removed or piles be used. Experience proves that with good material and careful work the settlement of embankments will be very slight.

Trenches should not be back-filled until the cement has had time to harden considerably, and then it should be carefully done from both sides simultaneously. The evils of too hasty loading of the arch have been well shown by Mr. A. Fteley, by means of a device for measuring the deformations of the cross-section. The use of this during the progress of the construction of a large aqueduct showed a considerable settlement of the crown, due to too early loading. The diagrams also showed the insufficient strength of invert in yielding ground.*

The aqueduct should be covered to a depth of 3 or 4 feet to prevent the formation of ice and to protect the masonry. Embankments should be given a slope of $1\frac{1}{2}$ to 2 horizontal to 1 vertical, according to the nature of the material. They should be trimmed to a rounded outline and then sodded.

625. Special Details.—Masonry aqueducts, like canals, should be provided with gates, wasteways, and overflow-weirs at intervals, to maintain the water-level, and to enable the aqueduct to be emptied in parts. Masonry aqueducts are not designed to flow under pressure, and to insure safety in this respect, long aqueducts will require the construction of waste-weirs. Gates should be constructed at the junction of aqueduct with pipe-lines or siphons, and at terminal points. Intermediate wasteways or blow-offs are located near some natural watercourse, and should have a capacity, if possible, equal to that of the aqueduct. Fig. 160 shows one of the wasteways of the New Croton

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* Jour. Assn. Eng. Soc., 1883, II. p. 123.

Aqueduct and a stream-crossing at the same place, and Fig. 161 illustrates an undercrossing of the Brooklyn conduit.*

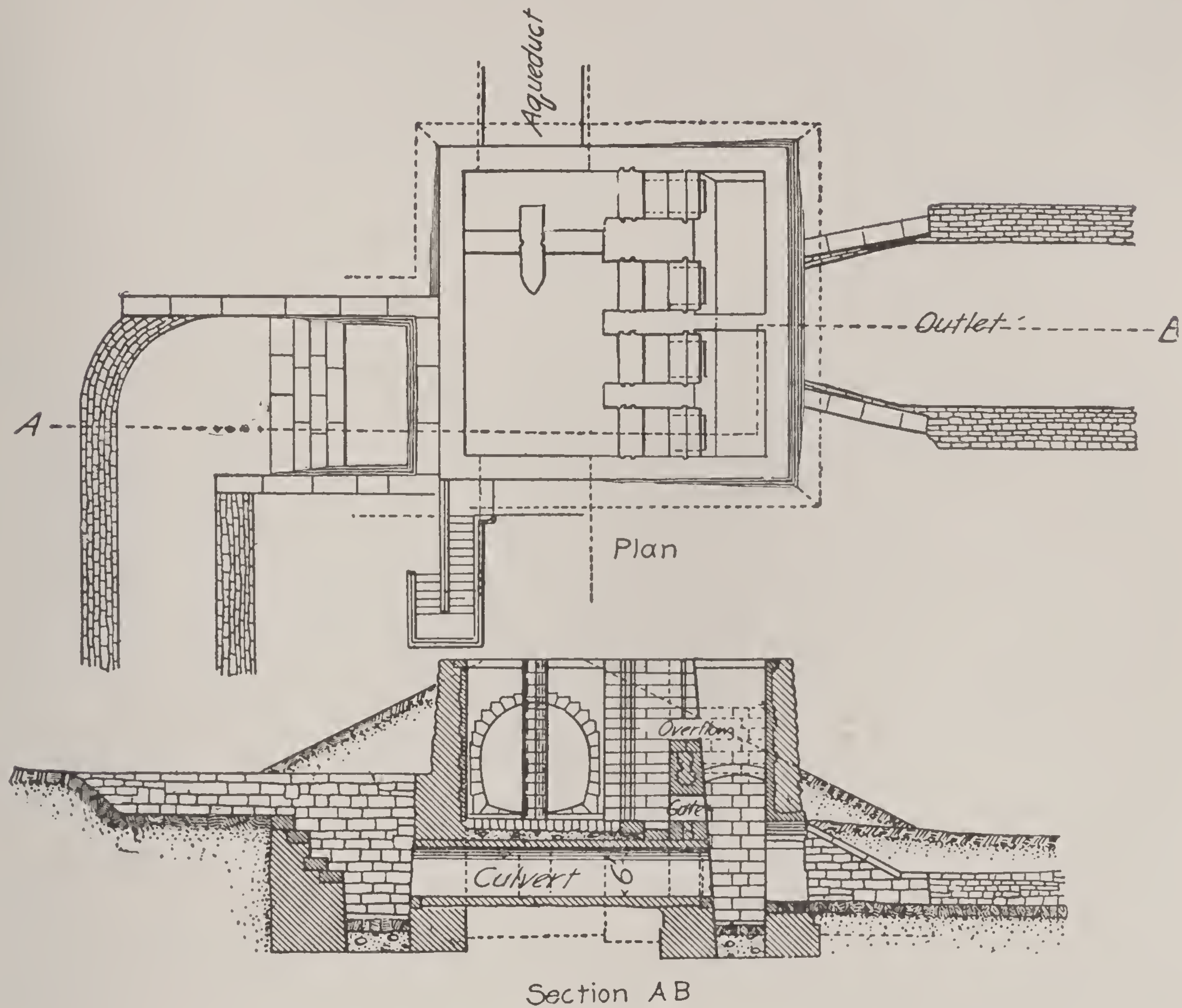

FIG. 160.—BLOW-OFF AND CULVERT, NEW CROTON AQUEDUCT.

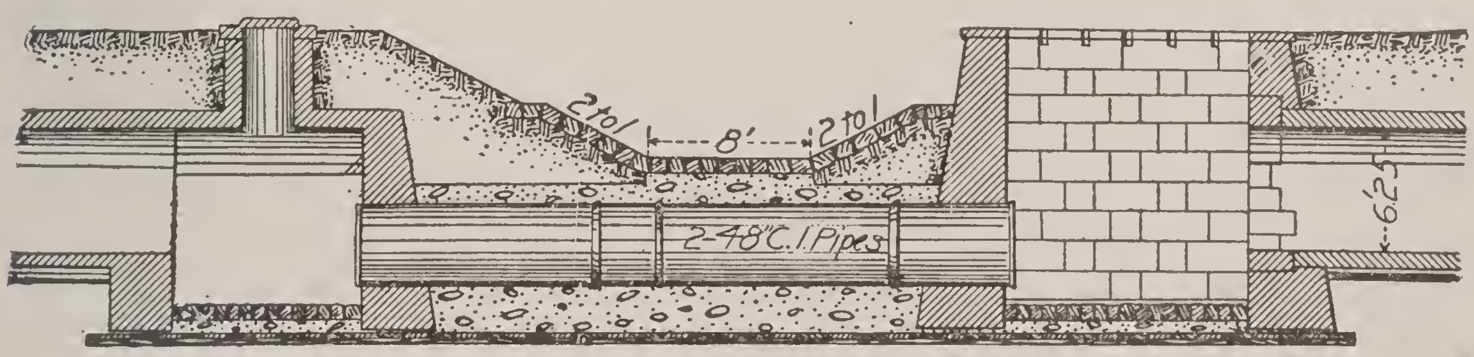

FIG. 161.—UNDERCROSSING, BROOKLYN CONDUIT.

Culverts for crossing small streams, and bridges for larger ones, are a part of the design. Some of the most monumental works of history are the bridges which have been built for carrying aqueducts. Large aqueduct bridges are now seldom constructed, pipe-lines being substituted, but bridges of moderate size will still often be the more economical design. These are usually masonry structures, and, as in the case of

* *Eng. News*, 1891, xxv. p. 225.

embankments, special pracautions must be taken to prevent settlement. Experience with other structures of a similar character led the engineers of the Wachusett aqueduct to adopt certain special precautions in the construction of the Assabet bridge. This is a masonry structure 389 feet long and of seven spans. The brick lining of the aqueduct was first covered with a coat of cement mortar, which was then painted. Sheets of lead weighing 5 pounds per square foot were then carefully cemented in place, coated with asphalt, and the interior lined with 8 inches of brick. The roof is of brick arches on I beams, and is covered with cement and asphalt.

Small streams are led under aqueducts through culverts, or through inverted siphon-pipes with gratings at entrance.

626. Tunnels.—Tunnels frequently form a part of an aqueduct. The section adopted is usually the same as for the masonry portion, but the circular form may here be used. In unstable material a brick lining will be required. If a tunnel is unlined, the section should be increased by 15 to 20 per cent to allow for increased resistance due to the roughness of the surface. The unlined portion of the Wachusett tunnel is thus made about 21 per cent larger than the lined portion; likewise in the case of the Manchester aqueduct the increase in section is about 16 per cent. Mr. J. R. Freeman adopts for certain proposed aqueducts for New York City a value of n in Kutter's formula of .028 for unlined tunnel and .014 for lined tunnel. He estimates that for large sizes the lined tunnel is actually cheaper for a given capacity than the unlined.*

Tunnels are usually built to flow free, but sometimes are operated under pressure. Thus the Croton aqueduct tunnel is under about 125 feet pressure for 7 miles, and under the Harlem River the head is about 425 feet, the hydraulic grade-line being there 120 feet above the river. The actual unbalanced water-pressure on the aqueduct lining would be the difference between the inside pressure and the pressure of the ground-water, which, at the river-crossing, would probably be measured by the level of the water in the river. If the ground-water pressure is in excess, there would be filtration into the tunnel.

It appears to be difficult to secure good work in placing the backing of tunnels, and the defects in this respect are notorious in one or two large aqueducts. Recent experiments by Col. A. M. Miller, U. S. Engineer, have shown that cavities which are not easily filled with masonry in cement, can be filled dry and successfully cemented by forcing in grout under pressure.†

* Report on New York's Water-supply, 1900, p. 318. † *Eng. News*, 1899, XLII. p. 410.

627. Aqueducts of Vitrified Pipe. — As already described in the last chapter (page 581), vitrified pipe can well be used for small aqueducts not under pressure. This pipe is considerably smoother than brick masonry, and for any given capacity will be more economical, at least for sizes up to 24 to 30 inches, and possibly for the 36-inch size.

PIPE-LINES.

The General Design.

628. Material to be Employed.—The advantages of various materials have been considered in the last chapter. Summarizing briefly, it may be said that cast iron is especially suited for conduits of small or moderate size, and for places where frequent use of branches and specials is called for. Steel is especially suited for very large sizes, and for heavy pressures, and for lines in those situations where a light pipe is especially desirable. Wooden pipe is adapted for use in remote regions, for low pressures, and where the pipe is to be exposed. Vitrified pipe may be used for small sizes and very low pressures.

629. The Profile.—The question of location has already been touched upon in a general way in Art. 614. A pipe-line must follow in general the variations of the ground-surface, and such a location should be selected as will enable it to do so and at the same time give low pressures, that is, it should be kept as near the hydraulic grade-line as possible. If the pipe is made of uniform size, the hydraulic grade-line will be a straight line from one end to the other; but if it is not practicable to keep the pipe-line below a continuous grade-line at all points, an intermediate reservoir may be placed at the high point and the sections on either side designed independently. Several such breaks may evidently be advisable in some cases. If the intermediate points are too high for this arrangement, then a deep cut or a tunnel will probably be desirable. Small elevations above the hydraulic gradient may be overcome by siphonage, but this will require special provision for the removal of air. In other cases pumping may be resorted to.

If the pipe-line dips too far below a straight grade-line, it may save expense to break the grade in this case also, by means of an intermediate reservoir so located as to give sufficient fall in the lower part of the conduit. Thus, in Fig. 162, AB is the grade-line with a pipe of uniform size, and ACB the gradient when a reservoir is inserted at C. The latter arrangement gives much the lower pressures.

Overflows and equalizing reservoirs are advantageous for regulating the pressure in the pipe; and to permit of their economical construction it is desirable to have the pipe-line approach or cut the hydraulic grade-line occasionally. Plan and profile should be so laid out as to avoid sharp curves as much as possible, and the curves used should conform to certain adopted standards. In deep valleys and gorges it will often

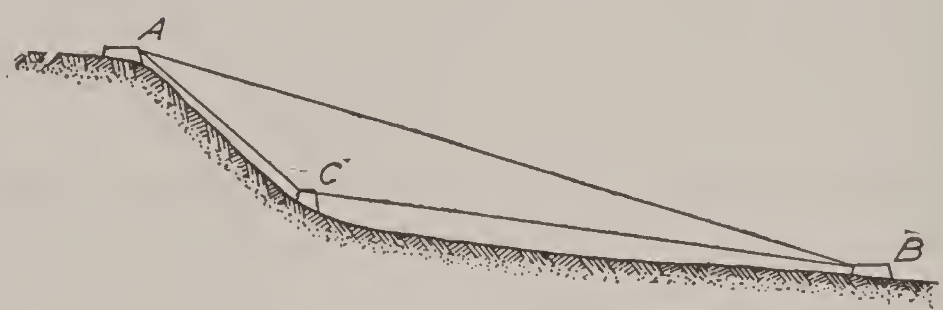

Fig. 162.

be best to carry the pipe for a short distance on a trestle or bridge, thus avoiding sharp curves and at the same time shortening the line.

630. Pressures to be Assumed.—The water-pressures in a pipe-line are measured by the ordinates to the hydraulic grade-line, which has a slope depending upon the frictional loss. When the water is stationary the hydraulic grade-line is horizontal and the pressures will be the same at all points at the same level. When flowing, the pressures will be much reduced at certain points. If a pipe-line is so designed that the lower end is closed at times, the pressures must be assumed to be static, i.e., measured from a horizontal hydraulic grade-line; but if it is so arranged that the water will always have free egress, then the pressures will be measured from the sloping hydraulic grade-line, and much will be saved in cost of pipe. In practice, the second condition is often practically obtained by placing small reservoirs and overflows on the hydraulic grade-line at intervals where the pipe-line rises close to this elevation. Each section of pipe between consecutive reservoirs may then be operated in the ordinary way and designed for the static pressure. This gives a pressure-line consisting of a series of horizontal lines. The method of designing for a sloping hydraulic grade-line requires that no part of the line can possibly be closed except at the extreme upper end. In this case also it is well to have overflows at various points along the pipe-line. Obviously both methods of designing may be employed for different sections of a conduit.

An example of a combination of both methods is the Rochester conduit, the profile of which is shown in Fig. 163. The pipe is a 38-inch steel pipe $26\frac{1}{2}$ miles long. In the middle of a $17\frac{1}{2}$-mile section an overflow-tower is connected to the pipe, and is always kept open, thus limiting the pressures on the upper half to those due to the hydraulic grade-line. A waste-pipe leads to a near-by creek. The lower half is designed for static pressure and is provided with gates in the usual manner.

In the case of large pipe-lines not connected with distributing systems the pressure to be considered in the design need not be much

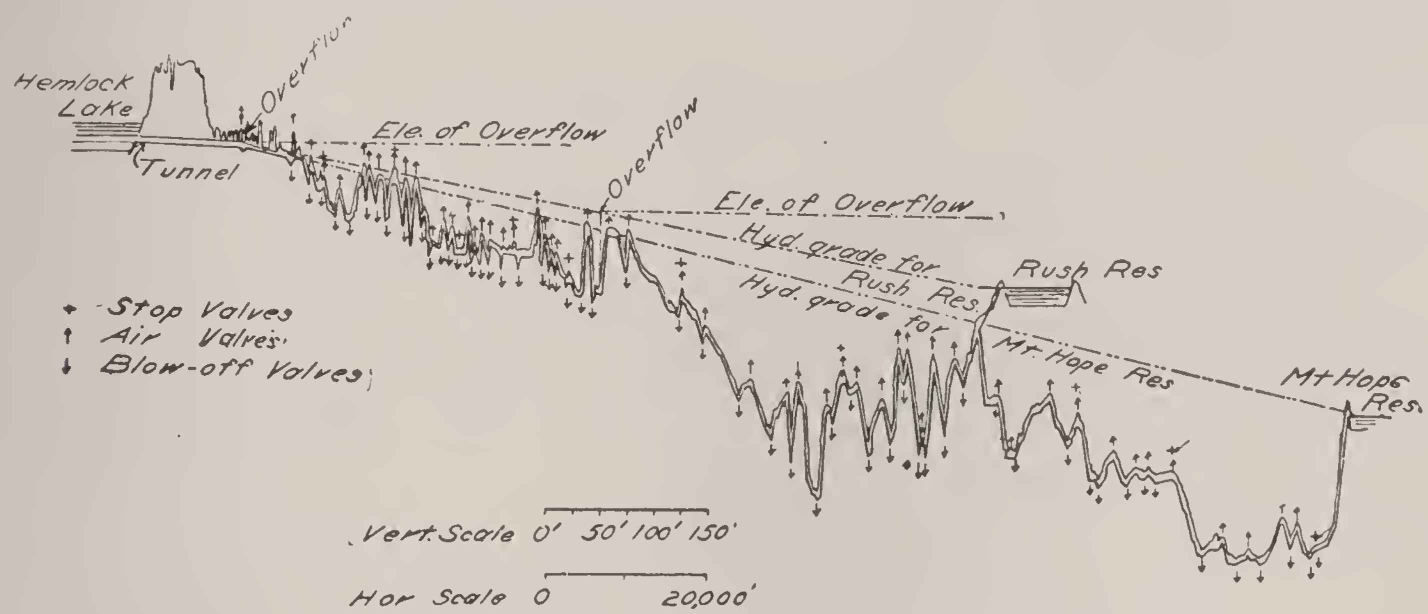

FIG. 163.—PROFILE OF ROCHESTER PIPE-LINE.

increased for the item of water-hammer. This is especially true of pipes ending in reservoirs and operated with open ends.

631. Calculation of Size of Pipe.—Where the total available head is fixed, the size required for any given capacity is readily determined. If the head is very small, the size required will be relatively large, and it may be more economical to use pumps, with a smaller pipe-line, designed as explained below. In case the water contains suspended matter, it is desirable to maintain a self-cleansing velocity of 2 to $2\frac{1}{2}$ feet per second, otherwise the sediment must be blown out at frequent intervals. If the line is divided into sections by reservoirs or overflows, the size of each section is determined independently of the others.

632. *Economical Size of Pipes where Pumping is Required.*—If the loss of head is not fixed, as is the case where the pressure is supplied by pumps, the size of pipe should be such as to make the total yearly expense a minimum. If the cost of various sizes of pipe is known, and the cost of pumping per unit of work done, the problem can readily be solved by a few trials.

In most cases the possible variation in pipe would not seriously affect the design of the pumps, and the cost of fuel would be about the only item affected by a small change in head. The additional cost would then be small. In the case of very long force-mains, however, or pipe-lines with several pumping-stations, nearly all items of expense would be affected by a change in size of pipe.

As this problem is of common occurrence in connection with distributing systems as well as pipe-lines, an approximate general solution for cast-iron pipe will be given, from which a good notion of the economical velocities for various sizes of pipes can be had.

From an analysis of Weston's tables and other data relating to the cost of cast-iron pipe, it is found that the cost of pipe, laid, is approximately given by the formula

$$c = 20 + 2ad^{1.55}, \quad \ldots \ldots \ldots \quad (1)$$

in which c = cost per foot in cents,
a = cost of iron in cents per pound,
and d = diameter of pipe in inches.

If in eq. (31), page 230, we express d in inches instead of feet, we have, for the velocity of flow in pipes,

$$v = 76.28\left(\frac{d}{12}\right)^{\frac{5}{7}} s^{\frac{4}{7}} = 13d^{\frac{5}{7}}s^{\frac{4}{7}}, \quad \ldots \ldots \quad (2)$$

in which v = velocity in feet per second, and
s = slope of hydraulic grade line,
= loss of head in feet per foot.

From eq. (2) we get

$$s = 0.011\,\frac{v^{\frac{7}{4}}}{d^{\frac{5}{4}}}. \quad \ldots \ldots \ldots \quad (3)$$

We also have the general relation

$$Q = v \cdot \tfrac{1}{4}\pi\left(\frac{d}{12}\right)^2. \quad \ldots \ldots \ldots \quad (4)$$

Then from eqs. (3) and (4) we have

$$s = 100\frac{Q^{\frac{7}{4}}}{d^{\frac{19}{4}}}. \quad \ldots \ldots \ldots \quad (5)$$

Let b = yearly cost of pumping 1 cubic foot per second 1 foot high, and Q = volume pumped per second. Furthermore, let r = rate of interest plus rate of depreciation of pipe-line. The total yearly cost of pipe and pumping, per foot of pipe, will then be

$$A = bsQ + cr = bsQ + 20r + 2ard^{1.55}. \quad \ldots \quad (6)$$

Substituting the value of s from eq. (5), we have

$$A = 100b\frac{Q^{2.75}}{d^{4.75}} + 20r + 2ard^{1.55}. \quad . \quad . \quad . \quad . \quad (7)$$

Differentiating with respect to d, etc., we find that for a minimum value of A

$$d = 2.22\left(\frac{b}{ar}\right)^{.16} Q^{.44}; \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

that is, for any given values of b and a the diameter should vary with $Q^{.44}$.

To express this relation in terms of velocity, which is a more convenient form, we may substitute from (4); *whence we have, for the economical velocity,*

$$v = 30\left(\frac{ar}{b}\right)^{.36} d^{.27}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

The cost of pumping is ordinarily expressed in terms of cost per 1,000,000 gallons lifted 1 foot high. This will vary largely in different plants, but the cost of additional lift will seldom exceed 3 to 4 cents per million-gallon foot, and in large plants will not exceed 2 cents. The total cost of pumping in large plants is usually from 3 to 5 cents.

Table No. 77 gives various values of v as computed from eq. (9) for various costs of pumping. The cost of pipe is taken at 1 cent per pound, interest-rate 4 per cent, and a depreciation of pipe-line of 1 per cent per year. For other values of the cost of pipe, (a), or of interest plus depreciation, (r), multiply the value of v given in the table by $a^{.36}$ or by $\left(\frac{r}{5}\right)^{.36}$.

If the pumping is done at a variable rate, then the maximum velocity should be made somewhat greater than the value given in the table. If, for example, the pumps are operated for half the time at a rate Q, then the value of b will be equal to the total yearly cost divided by the rate Q and by the head, and will hence be less than if a rate $\frac{Q}{2}$ were maintained for the entire year at nearly the same total cost. The resulting value of v will therefore be greater; in the assumed case it will be equal to the value given by the table multiplied by $2^{.36}$ or by 1.3.

For other than cast-iron pipe the actual cost will be different from the cost here assumed, but the variation in cost of pipe with size will be proportionately about the same, so that an approximate value for velocity can be found by using the table with such a cost of cast iron as will give the correct cost of some one size of conduit. As in all cases of maximum and minimum, a considerable change in the value of the variable when near to the correct value will affect the result but slightly. Another factor which usually enters is the gradual increase in the quantity of water which is to be pumped, so that the pipe must at first be made too large for economy.

TABLE NO. 77.

ECONOMIC VELOCITIES IN CAST-IRON MAINS WHEN THE COST OF PIPE IS 1 CENT PER POUND, AND THE COST OF ADDITIONAL LIFT IS 2, 4, AND 6 CENTS PER MILLION-GALLON-FOOT. INTEREST RATE PLUS DEPRECIATION = 5 PER CENT.

Cost of Pumping 1,000,000 gal. 1 foot high.	Size of Pipe.									
	4-in.	6-in.	8-in.	12-in.	16-in.	24-in.	30-in.	36-in.	48-in.	60-in.
	Velocity in Feet per Second.									
2 cents........	1.61	1.82	1.95	2.19	2.37	2.65	2.82	2.98	3.23	3.43
4 cents........	1.25	1.41	1.52	1.70	1.85	2.07	2.20	2.32	2.51	2.67
6 cents........	1.08	1.22	1.31	1.47	1.60	1.78	1.90	2.00	2.17	2.31

If pipe costs $1\frac{1}{4}$ cents, multiply above values by 1.08.
" " " $1\frac{1}{2}$ " " " " " 1.15.

If the pressures in a pipe-line vary greatly, the most economical size will not be the same for all sections, but will vary a little, being the smallest under the heaviest pressures.

Construction.

633. **Plan and Profile.**—In preparing a design, an accurate map and profile should be made to a large scale, on which should be shown the exact location of the pipe, the radius and length of each curve, location and amount of angles or bevels, and the position and size of valves and other appurtenances. The various sections of pipe and the special forms can then be numbered to correspond with the location on the map so that they can be readily sent to their proper places.

634. **Trenching.**—Trenches for water-pipe are not usually deep enough to require much bracing or sheeting, the depth being ordinarily only sufficient to give the necessary covering. Deep trenches will, however, be required occasionally, as where the pipe-line crosses a high ridge extending above the hydraulic gradient. The methods of

sheeting and bracing, and of trenching, are the same as used for sewer work, and will be found fully described in works on sewerage. In wet ground, only a short section of trench should be opened at once, in order to keep the inflow of water as low as may be. In rock, the trench must be carried 3 to 6 inches below the proper grade and the space refilled with sand or fine material to give a proper bedding for the pipe. Bell-holes for cast-iron pipe must be excavated wide enough to give plenty of room for making a good joint. All existing pipe-lines and other structures must be carefully supported or removed.

635. Foundations.—Where the material is too soft to give a good bearing, it may be necessary to use artificial foundations. These may consist of blocks placed on stringers, or of piling, with caps on which the pipe may rest. When full of water the pipe will weigh but little, if any, more than the soil displaced, so that there is little tendency for it to settle after the back-filling has become compact. A foundation is necessary, however, to keep the pipe in place during construction and to hold it rigid against unequal pressures. For large valves and other heavy parts, special foundations of concrete are likely to be needed. To assist in getting large pipe to grade, it is convenient to support it on wooden blocking and wedges, two blocks being used under each section. After laying, the trench should be well filled underneath the pipe by suitable material. At sharp curves and angles, buttresses of concrete or stone masonry should be built to prevent distortion of the line by the water-pressure. Anchorage masonry is also desirable at intervals in case the grade is very steep.

636. Laying of Pipe.—*Cast-iron Pipe.*—The laying of cast-iron pipe is usually begun at a valve or special. Small pipe up to 6 or 8 inches in diameter is easily handled without a derrick, the sections being lowered into the trench by two or three men. In laying, care should be taken to enter the pipe to its full depth and to see that there is sufficient joint-space all around. The pipe should have been inspected for eccentricity, and the joint-room should not vary more than $\frac{1}{16}$ inch from the required dimensions. The spigots should be adjusted by wedges to give a uniform joint-space. The packing of jute or other material is inserted and thoroughly packed with a thin yarning-iron. If special strength is not required, this packing may nearly fill the space back of the enlargement or V-shaped space in the bell. The remaining space is filled with molten lead. In pouring the joint the lead is guided into the space by a jointer, commonly made of clay formed around a length of rope. This is placed about the pipe so as to press against the hub, except at the top, where an opening is made for pouring. Patent jointers are better for large pipe and difficult work. After pouring, the lead is loosened somewhat from the pipe by means of a chisel and set up by calking-iron and hammer. To do good work there should be plenty of room around and under the pipe. In wet trenches and with small pipe, two or three sections may be joined before lowering. To handle large pipe, various forms of derricks are employed, the three-legged form being commonly used. Fig. 164

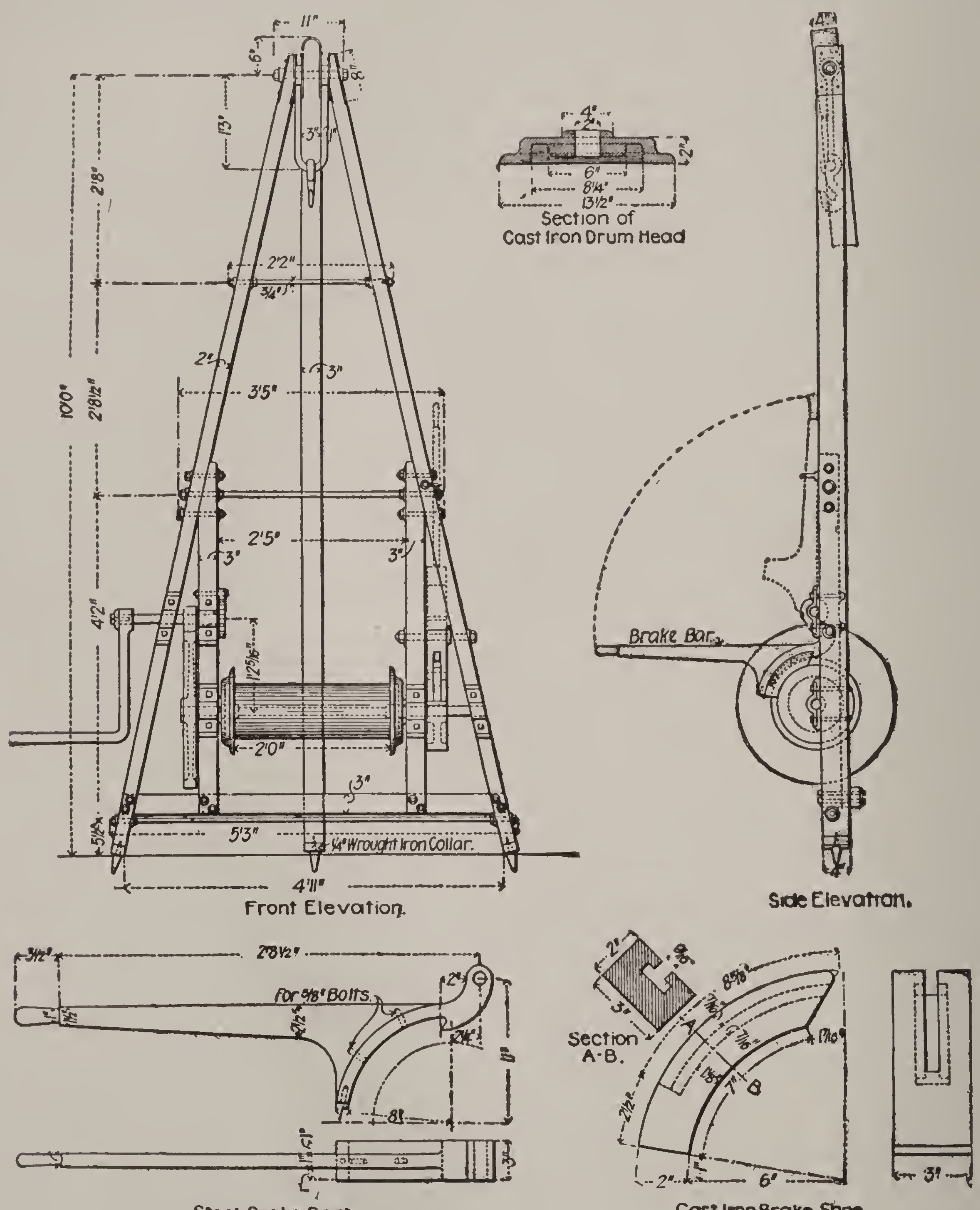

FIG. 164.—PIPE-DERRICK, BALTIMORE, MD.
(From *Engineering Record*, vol. XXXVII.)

illustrates a specially designed derrick of the Baltimore Water-works, made for handling pipe up to 16 inches in diameter.*

* See also *Eng. News*, 1896, XXXV. p. 339, for illustration of another form.

637. *Steel Pipe.*—Riveted pipe should be connected up in as long sections as practicable before being transported to the trench, so that as much of the riveting may be done by power-riveters as possible. For this reason it will be desirable on large works to establish a riveting and dipping shop not far from the pipe-line. In transportation and construction the greatest care should be taken to avoid injuring the coating. When placed in the trench the pipe should have an even bearing on firm soil or on blocking, and should be well supported while the joints are being riveted. The riveting is usually done by hand, but power-riveters have been used in a few cases. These are made in two parts: (1) a ring which fits around the pipe and forms the support; and (2) the power appliances, which are placed on the inside. Two rivets on opposite sides may be driven at the same time. Power-riveters require a much larger excavation to enable them to operate, but they are desirable where the rivets are large.* The percussion pneumatic riveter, which is largely used in ship-building and similar work, is well adapted for this work.

After riveting, all field-joints should be calked, and these and all other abraded places painted. Some recalking may be needed after the pipe is tested.

638. *Wooden Pipe.*—Points to be specially observed in the construction of wooden-pipe lines are care in the selection of the timber, proper coating of the bands and spacing of same, and proper cinching. Spacing of bands should be fully indicated on the profile. In making the pipe, the staves for the lower half are laid in a cradle of circular form, and those for the upper portion are supported on rings. The pipe can be built in sections, which may readily be connected by cutting the closing staves slightly too long and springing them into place. Sharp angles cannot be followed, and even to make easy curves the pipe has to be forced out of line by means of jacks and braced in place until the construction has progressed considerably.

639. **Testing and Inspection.**—The pipe as completed should be tested in sections of 1000 feet, or thereabouts, by hydraulic pressure. For this purpose the ends are closed with specially-made blanks, which are provided with pipe-fittings, valves, and gauge attachments. The pressure used should be somewhat higher than that which will obtain in regular service. It may be inconvenient to leave a steel pipe entirely uncovered until the test is made, on account of trouble due to

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* See *Eng. News*, 1898, XXXIX. p. 170, and Trans. Am. Soc. C. E., 1897, XXXVIII. p. 264.

temperature changes, but the field-joints at least should be left open to inspection. Wherever leaks are found the pipe should be recalked.

After the construction is completed a pipe-line should, if possible, be inspected in the interior throughout its entire length. By suitable means comparatively small pipes can be thus inspected. A 30-inch pipe-line at Syracuse was inspected by a man passing through the pipe by the aid of a special car.

640. Covering of Pipes.—Except in mild climates a conduit will need to be covered to prevent the water from freezing; and even in warm climates it will usually be desirable to cover conduits of iron or steel to protect them from extreme variations of temperature. The recently constructed Coolgardie pipe-line, as first proposed, was to be left exposed, on account of the objectionable character of the soil, but later it was decided to cover it. Iron and steel would be more durable if exposed, as they could then be kept painted, but much expense would be involved in the construction of expansion-joints, and there would also be more danger of interruption of the supply. Not so much objection is to be made against exposed wooden conduits, and some have been so constructed. In a wooden pipe there is no trouble from expansion, and the water is not so greatly affected by temperature changes.

The depth of covering to protect pipes against freezing, in the case of the large conduits under discussion, need not be more than 3 or 4 feet in the northern part of the United States. (See also Art. 754.) A covering of 2 or 3 feet is sufficient to protect them from injury by ordinary traffic. The maximum allowable depth of covering will seldom be reached, but for very large pipes additional strength should be given when the lightest pipe would otherwise be used, if the depth of filling exceeds 15 or 20 feet. For the reasons pointed out in Art. 576 the back-filling should be done with great care up to the top of the pipe, the material being placed and tamped in 4- to 6-inch layers. This is especially important for light steel pipe. If the pipe is located in paved streets, the back-filling must be thoroughly tamped throughout (Art. 756).

Appurtenances and Special Details.

641. Provision for Expansion and Contraction.—In the case of cast-iron pipes, expansion and contraction are sufficiently provided for by the flexibility of the lead joints, unless the pipe be exposed for long distances. In riveted steel pipe, ordinarily no provision for expansion is made, the pipe being therefore stressed accordingly. To resist the forces developed will require heavy anchorages at the ends of pipes and at junctions with masonry portions, or else expansion-joints must be used at those places, as has been done in some instances. Valves and special castings must also be made strong enough to resist this force of expansion. (See page 555.) Exposed sections of pipe, if of any considerable length, should be provided with expansion-joints, but as these are, for large pipe, somewhat expensive and difficult to make operate satisfactorily, they should be avoided if possible. For small pipe, and at points readily inspected, the ordinary stuffing-box with gland, etc., answers the purpose. For larger pipes various other forms have been devised, some of which are illustrated in Fig. 165. Fig. 165*a* illustrates a joint which has been used to a considerable extent in

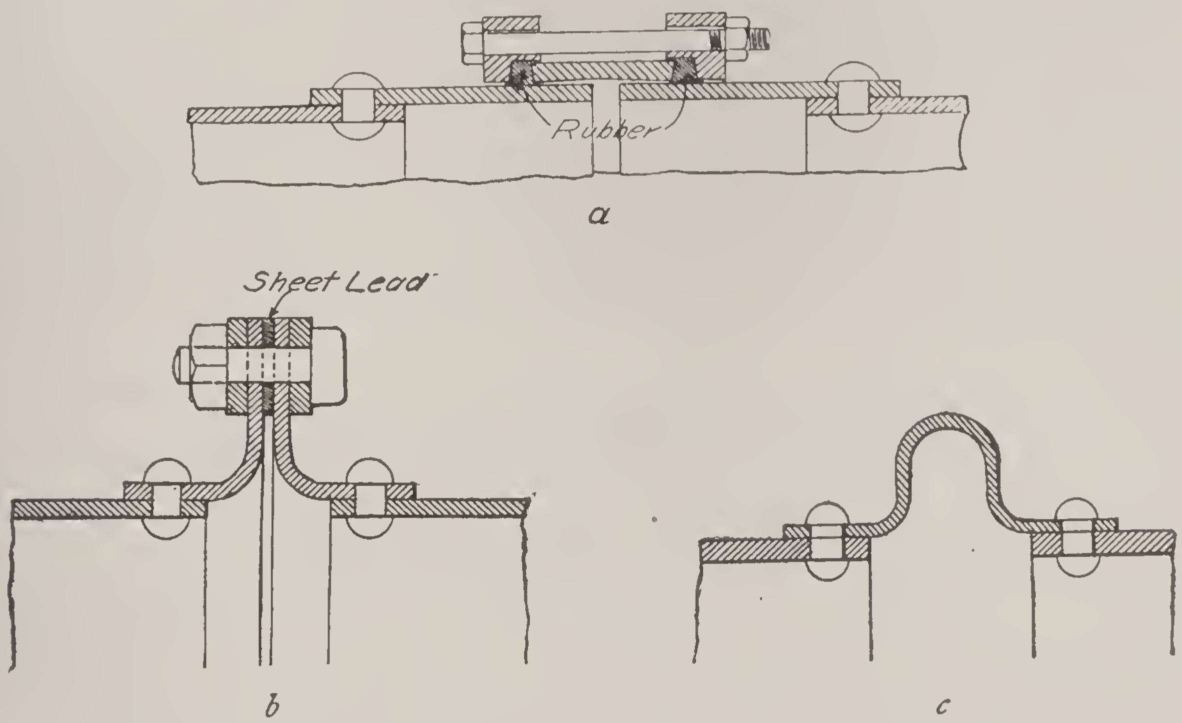

FIG. 165.—FORMS OF EXPANSION-JOINTS.

Paris with good satisfaction. Figs. *b* and *c* illustrate joints wholly of metal which have been employed to some extent.* All these are designed for large pipes.

642. **Manholes.**—Manholes should be provided in large pipe-lines at intervals of 500 to 2000 feet, and particularly at depressions and near valves. They are usually of cast iron of oval form, about 18 or 20 inches long by 12 or 14 inches wide. They are bolted to cast-iron flanges, which are cast with or bolted to the pipe. Where it is likely that mechanical scrapers may be used to clean a pipe, long removable covers or hatch-boxes should be built at intervals to admit the scraping-machine. (See Chapter XXIX for description of such machines.)

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* *Eng. News*, 1899, XLI. p. 406. See also large expansion-joint in *Eng. Record*, 1899, XL. p. 156.

643. Stop-valves.—To enable a pipe-line to be readily inspected and repaired, stop-valves should be inserted at intervals of 1 or 2 miles, and especially at important depressions and summits. Otherwise to empty and refill a long conduit would require several days. In the case of breakage, the water can be shut off at the nearest valve, and any considerable waste or serious damage be prevented. Large valves are expensive, and just as an increase in the cost of pipe will decrease the economical size of pipe, so in the case of valves, a size considerably smaller than the pipe can often be used with good economy. The cost decreases rapidly as the size decreases, while the loss of head due to the contraction, if made with suitable reducers, is not large. The best size can readily be calculated from cost of valves, cost of reducers, increased friction in smaller pipe, and cost of pumping or value of head. An advantage of the small valve is that it is much more easily manipulated, and in some cases it may be desirable to hold to such sizes as can be operated by one man without the use of special gearing. An example of such an arrangement is in the connections of the large pipes of the East Jersey conduit, where a number of 16-inch pipes with 16-inch valves were used.

Valves up to about 16 inches in size are usually operated direct. Larger valves are operated by gearing, or by hydraulic power, the cylinder for the latter being constructed as a part of the valve. Large valves are usually provided with by-passes which are opened first, so that the pressures on the main valves are more nearly balanced. The force required to move a valve can be roughly calculated if we know the pressure and weight of valve-disks. The necessary gearing for manual operation can then be calculated. Very large valves are sometimes divided into two or more parts to give easier handling, and some are so arranged that when operated a small secondary valve is first opened which acts like the by-pass to reduce the amount of unbalanced pressure.

Valves of all kinds and designs are furnished by various special manufacturing concerns. Fig. 166 shows an ordinary single-disk valve. Fig. 167 shows a large valve with gearing and by-pass such as is used on the Boston water-works. Valves for water-works should have double-faced disks, which should seat readily and accurately. Many forms are made with two disks which adjust themselves to the seats. All sliding surfaces should be faced with bronze, and the stems should also be of this material, and carefully proportioned as to strength. The waterway should not be obstructed when the valve is opened. All parts should be readily removable. Valves should be

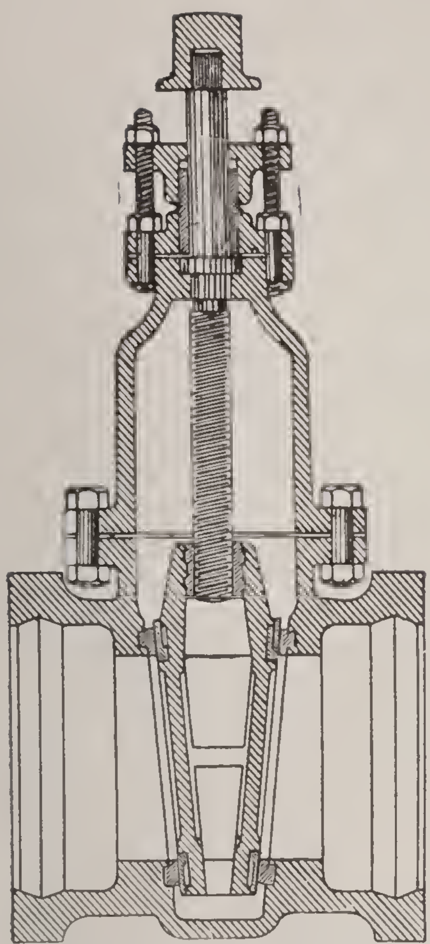


FIG. 166.—GATE-VALVE WITH SINGLE DISK.

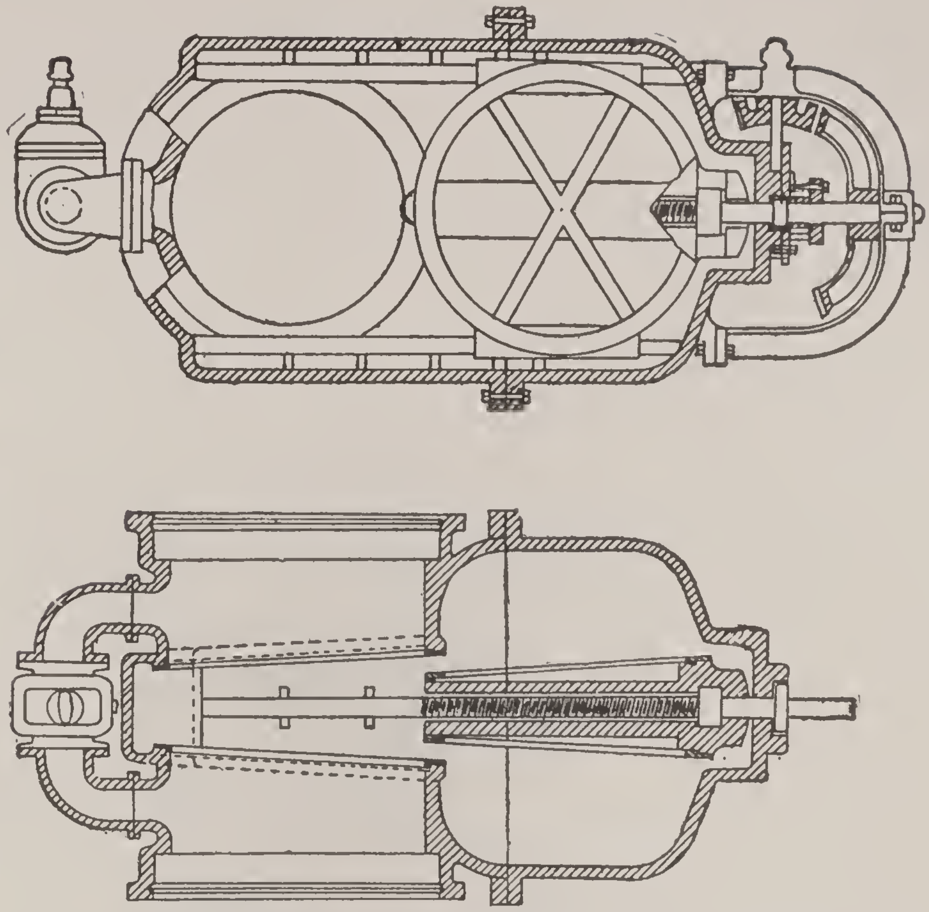


FIG. 167.—VALVE WITH GEARING AND BY-PASS.

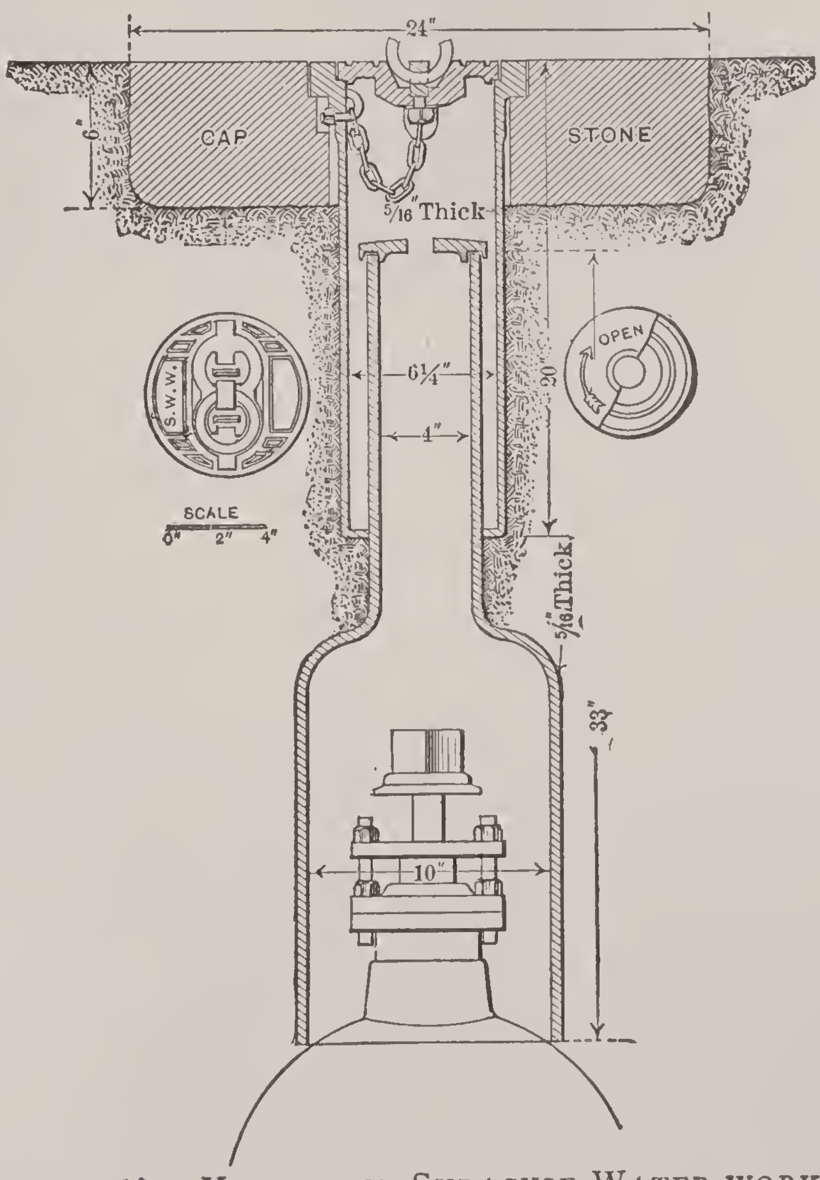
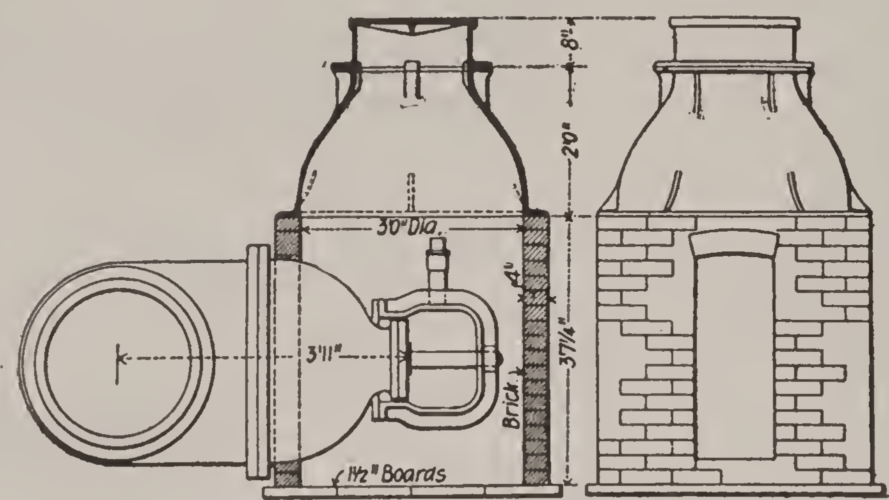


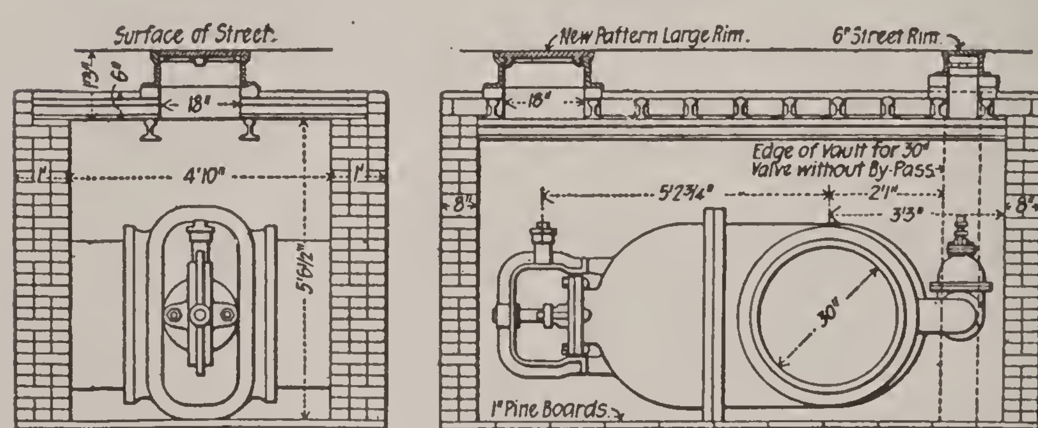
FIG. 168.—VALVE-BOX, SYRACUSE WATER-WORKS.
(From Trans. Am. Soc. C. E., vol. XXXIV.)

thoroughly tested for leakage from each side with valve closed, and again tested with the valve open.

Small valves (up to 16 or 20 inches) are placed vertical, with stems protected by cast-iron valve-boxes or by masonry vaults. Large ones are placed horizontal, with the operating mechanism surrounded



Vault for 16- to 24-inch Valves.



Vault for 30- to 40-inch Valves.

FIG. 169.—VALVE-VAULTS, BALTIMORE WATER-WORKS.
(From *Engineering Record*, vol. XXXVII.)

by a masonry vault or manhole. The standard valve-box used at Syracuse is illustrated in Fig. 168, and in Fig. 169 the valve-vaults used at Baltimore.*

644. Air-valves.—At every summit of a pipe-line and at shut-off valves there should be placed an air-valve to permit the escape of air on filling, the entrance of air on emptying, and frequently the escape of air which may gradually accumulate at summits. The first and second objects are readily obtained automatically, and the third often is. Air-valves are of various design, a form known as the Brooks Automatic Valve being illustrated in Fig. 170. This form consists of a brass disk-valve supported on a spindle and opening inwards. When there is no water in the pipe the valve remains open, but when the water reaches the valve as the pipe is filled, it closes quickly by reason

* *Eng. Record*, 1898, XXXVII. p. 143.

of the buoyant effect and the velocity of the escaping water. A form is often used in which a brass ball constitutes the valve.* For large pipes a cluster of small valves is employed, and it is well to have them so arranged that they will not close simultaneously. The area of air-valves is determined from considerations of quick filling, and sometimes also is calculated to be sufficiently large to admit air fast enough to prevent excessive vacuum in case the pipe should be broken. In distributing systems, hydrants at summits can usually be used as air-valves.

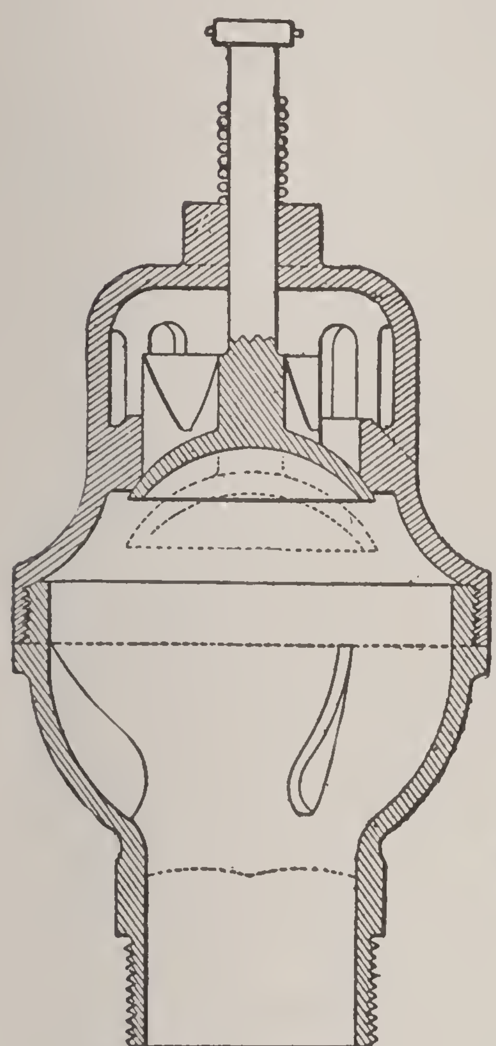


FIG. 170.—AIR-VALVE.

At sharp summits, and with low velocities and pressures, air will be apt to accumulate and give trouble unless removed, especially in the case of force-mains. Air can be removed by hand-operation of valves of the form already described, or by automatic valves. A common form of this type of valve is illustrated in Fig. 171. It is so proportioned that when air collects in the chamber and the float is no longer supported by water, the valve opens and permits air to escape till the water again rises to the float. It is necessarily a very small valve, and not well suited for the other purposes already mentioned.† The Engineering Commission of the Coolgardie pipe-line recommended that at important summits the pipe should be made of twice the ordinary size, so as to facilitate the collection of air.

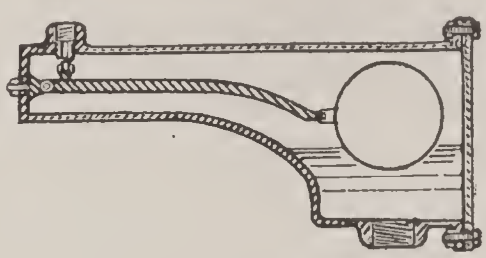


FIG. 171.
AUTOMATIC AIR-ESCAPE VALVE.

An air-valve is usually connected to the main pipe by means of a short branch, which is provided with an ordinary gate-valve so as to permit the removal of the air-valve for repairs. Air-valves must be well encased and protected from frost.

645. Blow-off Valves.—At all depressions, blow-off valves should be provided, the waste-pipes from which should be led to a sewer, stream, or drainage-channel. These valves need be only about one-third the size of the main pipe.

* For other designs see references 3, 6, and 7, p. 627.

† See *Eng. Record*, 1899, XXXIX. p. 493, for description of large automatic air-valve.

646. Self-acting Shut-off Valves.—Several English pipe-lines have been provided with valves so arranged that in case of accident to the pipe they will gradually close and so prevent loss of water and the destruction of property by flooding. In the device used, a lever carries at one end a small disk placed in the centre of the pipe. If the velocity of the water exceeds a certain amount, the pressure on this disk moves the lever, thus releasing a weight which in turn operates a butterfly valve in the main pipe.*

647. Check-valves.—These are introduced at points where a breakage would permit a large loss of water by backward flow, such as at the entrance to reservoirs, at the foot of long upward inclines, and in force-mains just beyond the pumps. Their use in connection with the circulation in reservoirs is mentioned in Art. 707. Fig. 172 illustrates an ordinary check-valve for small pipes. For pipes larger than 24 to 30 inches a diaphragm or valve-plate is cast in an enlarged section of the pipe, and a number of small valves attached to this plate, the total area of valves usually exceeding that of the pipe. A small by-pass is also provided to avoid heavy water-hammer.

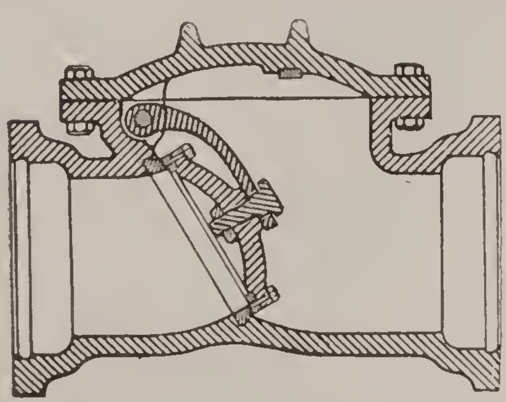


FIG. 172.—CHECK-VALVE.

648. Pressure-regulating Devices.—Various methods of automatically regulating the pressure are employed in different places. One very desirable method of regulating the pressure in a long conduit is by means of reservoirs and open stand-pipes, as already noted (Art. 629). These structures must be provided with overflows, and if the demand is quite irregular and the stand-pipe small, much water is likely to be wasted. This waste can be avoided and the flow adjusted to the demand by the use of the balanced float-valve, described on page 481.† By this means the level in the reservoir may be kept constant by varying the opening in the preceding section of pipe. This method is applicable where a pipe-line is divided into several levels, or where a low-level district is served from a high-level source, but the section of pipe-line leading to such valve must be designed for static pressure.

By suitable arrangements the balanced valve may be used also as a pressure-regulator, or pressure-reducer, without the interposition of a reservoir. To accomplish this the valve (see Fig. 135, page 486) may

* Proc. Inst. C. E., vol. CXXVI. p. 2.

† See description of special arrangement of such a valve, in *Eng. News*, 1898, XL. p. 158.

be operated by a small piston, *d*, acting in a closed cylinder. On one side of the piston this cylinder is arranged to communicate with the main pipe at whatever point the pressure is to be regulated. Upon the other side of the piston a spring acts in opposition to the water-pressure, which spring may be adjusted to any given tension. So long, then, as the pressures of the water and spring are equal no movement takes place, but as soon as the water-pressure exceeds that of the spring the valve is moved, which either increases or decreases the discharge as the case may be, and again brings the pressure down to the normal amount. A flexible diaphragm of thin metal may be used in place of the piston.

Safety-valves, or pressure-relief valves, are occasionally used at the ends of long pipe-lines or wherever water-hammer is especially to be feared. They are simple disk valves opening outwards and held in place by springs which are adjusted to the water-pressure. They should be of large section and designed with reference to the principles discussed in Art. 279, page 252. They take the place of air-chambers, and are more convenient at points where air-chambers could not readily be kept full of air. The water which passes through them at times of excessive pressure is of course wasted. The balanced valve can also be readily used as a safety-valve by the method described in the preceding paragraph.*

649. Terminal Arrangements.—The upper end of a gravity pipe-line is usually enclosed in masonry and provided with a sluice-gate or valve. At this point it is also desirable to have a weir or measuring-sluice. If pumps are employed, then a Venturi meter is a valuable device for measuring the flow. The lower end of a pipe-line usually terminates in a reservoir, where again valves are provided and where connections may also be made directly with the pipe system. In case the pipe-line is designed according to the hydraulic grade-line, no valves should be placed here, or if so placed, should be interlocked with waste-valves, so that the latter must be open before the former are closed. Such interlocked valves were used on the East Jersey pipe-line.

Intermediate stand-pipes and reservoirs at the hydraulic grade-line may be merely short open pipes placed vertically, or laid up an adjacent hillside till they reach the proper elevation and where provision is made for overflow; or they may be larger or smaller reservoirs, according to the necessity for storage. It may be better and more

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* See details of such a valve used on the East Jersey pipe-line. *Eng. News*, 1893, XXX. p. 24.

convenient to store water in three or four reservoirs than in a single one. Rochester has two such reservoirs. Liverpool has six reservoirs on a 67-mile pipe-line, of capacities ranging from $2\frac{1}{2}$ million to 650 million gallons. One advantage of large reservoirs is that the pressures are kept more constant without overflow, or with a less frequent adjustment of valves. Automatic valves may be used as described in Art. 648.

650. Crossings.—In crossing under other structures, such as railways, buildings, sewers, etc., special precautions should be taken to avoid all danger of future breakage. Pipe of extra strength may be used, or added strength given by a bed and covering of concrete. Large pipe-lines should be divided into two smaller ones for safety, with valve-connections at the ends. In very important cases the pipe may be laid in a subway so as to permit of repairs as readily as elsewhere. Streams are crossed either on bridges, or by laying the pipe beneath the stream-bed, or by the use of a subway as above mentioned. At Cleveland, Ohio, several crossings of narrow navigable streams have been changed to tunnel-crossings so as to permit of repairs. These tunnels are about 600 feet long and $9\frac{1}{2}$ feet in diameter, and are located 78 feet below the street-surface. They end in vertical shafts provided with manholes. The pipes are of steel, 48 inches in diameter, the vertical sections of which are supported upon I beams built in the shaft. Expansion-bearings are provided at the bottom, and the horizontal portions are supported on saddles 6 feet apart.* Many examples of this form of crossing exist in European works, such as the Mersey crossing of the Liverpool aqueduct, and the crossing of the Seine at Paris.

In this country the common practice in crossing a stream is to lay a cast-iron or steel pipe below the stream-bed, or else to employ a bridge-crossing. Where no bridge already exists the former will ordinarily be the cheaper, and in many cases, as in navigable channels, a bridge could not be permitted. In other cases it may be cheaper to build a bridge especially for this purpose, as in rocky canyons and narrow gorges. At the angles at ends of bridge- and submerged crossings special care is necessary to keep the pipe from separating at the joints.

651. Bridges.—If the pipe-line crosses an existing bridge, it will usually be convenient to support it beneath the flooring. Where a bridge is built for the purpose, no floor-system is put in, but merely suitable straps or stirrups to support the pipe. Steel or wood-stave pipe may be used for short spans without other support than that fur-

* *Eng. Record*, 1898, XXXVIII. p. 449.

nished by the pipe itself. A steel pipe $\frac{1}{4}$ inch thick, full of water, will, at a fibre-stress on gross section of 10,000 pounds per square inch, span a length of about 65 feet, if the tendency to buckle is not taken into account. This span-length is nearly independent of the diameter of the pipe, varying directly with the thickness of the material; but with large diameters the allowable stress would have to be much reduced, or else provision made to stiffen the upper plates of the pipe. The pipe can readily be stiffened by the use of angles riveted along the upper surface, and also by placing the longitudinal seams near the top. If a pipe is used as a bridge, the circular seams must be designed for the extra stress involved. Expansion can be provided for in such a bridge by resting the pipe at one end in a saddle which is supported on rollers. An expansion-joint is then placed just back of this saddle. A pipe bridge would be cheaper than a separate structure even if the metal had to be much thickened to give the necessary strength. The pipe can also be advantageously curved so as to constitute an arch bridge. This is a common practice in Europe, where spans of more than 150 feet have been built in this way. The method has also been used in the new Weston aqueduct, where an arch span of 80 feet has been made of a 90-inch pipe.* Wood-stave pipe has also been used in this way for spans of over 100 feet.†

652. Protection of Exposed Pipes. — The amount of protection required to prevent freezing on bridges, or at other exposed places, depends upon the size of pipe, the amount of circulation during periods of minimum flow, the temperature of the air and the water, and upon the length of the exposed portion. No general rule can be given. Usually the water is from surface sources, and its temperature in winter will be but little above the freezing-point, unless it should pass for long distances in deep trenches. The temperature of the water will change very slowly in large pipe-lines, and in the case of pipes 2 feet or more in diameter special protection would seldom be needed at crossings of ordinary length, if the water has at all times some movement. A wooden pipe possesses much advantage in this respect over pipes of iron.

Small lines, especially distributing-mains, require protection. This is usually furnished by placing the pipe in a wooden box and filling around it with some non-conducting substance, such as sawdust,

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* *Eng. Record*, 1904, XLIX. p. 480.

† Trans. Am. Soc. C. E., 1894, XXXI. p. 135.

mineral wool, asbestos, hair-felt, and the like. A mixture of plaster of Paris and sawdust has been used with good results. Any packing to be effective should be kept dry. The packing is often arranged to give one or more dead-air spaces around the pipe to aid in preventing radiation.

Materials such as above mentioned act to retain the heat of the water; but if the water is already near the freezing-point, they are not very efficient. Some method of applying heat may be desirable. Mr. S. E. Babcock has successfully solved this problem in the case of an exposed pipe at Little Falls, N. Y., by the use of wool waste as packing. This material contains a small amount of oil and gradually decomposes, thus giving off a small amount of heat. It was found necessary to renew this packing in five years, but the expense was small.*

653. Submerged Pipes.—Various methods are employed in laying pipes beneath watercourses. In the case of small streams the usual method is to employ a coffer-dam and lay the pipe as on dry land. Where the water cannot readily be excluded in this way the pipe must either be put together before lowering in place or must be laid by divers. Submerged pipe should, as a rule, be laid in a trench and carefully covered to prevent injury by waves, drift, ice, boats, etc. The trenching is done by dredging, and any drilling and blasting which may be necessary can be done from platforms or from anchored barges. In the case of at least one pipe-line the trench was made by the scouring action of the water, which was forced to flow beneath the pipe as it was gradually lowered into place.† The trench should be formed to line and grade, or at least an accurate profile of the same should be taken.

Various special details are used in submerged-pipe laying, such as the various forms of flexible joints to enable the pipe to conform to the grade of the trench, and special joints for easy connection where divers are employed. Submerged pipe should be thoroughly tested either in sections before laying, or better, after the line is completed, in which case compressed air can be used for the purpose. Leakage of air will be indicated by the appearance of bubbles, and the imperfect joints can then be calked by divers. The various methods of laying submerged pipe will now be described together with some of the special details used in this work.

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* Proc. Am. W. W. Assn., 1892, p. 110. See also description of boxing at Duluth, in *Eng. Record*, 1899, XXXIX. p. 162.

† *Eng. News*, 1892, XXVII. p. 424.

1. Where the stream is shallow, a common method of laying is first to connect the entire pipe, or large sections of it, on platforms extending across the stream, and to lower the portion so connected by means of screws. Ordinary joints can usually be employed and the pipe put together to fit the profile of the trench. Pipes can very conveniently be laid in this way from the ice during winter.

Two cases of this method of laying will be briefly noted. At Cedar Rapids, Ia., 600 feet of 16 inch pipe was laid in this way in a depth of 2½ feet of water. A trench 2 feet deep was first excavated, and framed trestle-bents set up 12 feet apart. A barge was then run between the legs of the trestles, the pipe put together on the barge, and then slung by straps fastened to 1¼-inch threaded rods suspended from the trestles. When the entire pipe-line was connected, it was all lowered together, electric-bell signals being used to secure simultaneous action among the several men stationed at the screws. The cost of laying was $1.25 per foot.*

At Escanaba, Mich., 2000 feet of 12-inch wrought-iron pipe was lowered through ice at a cost of $200. The trench was excavated in sand by means of the water-jet, after the pipe was laid.†

Where the pipe cannot readily be built to conform to the trench, or where settlement is feared, a certain number of flexible joints may be used. The most common form is the Ward joint, designed by Mr. J. F. Ward many years ago. It is illustrated in Fig. 173. To make the joint tight requires that some tension be put upon the pipe after the joint is in place. Other forms of flexible joints are illustrated on the following pages.

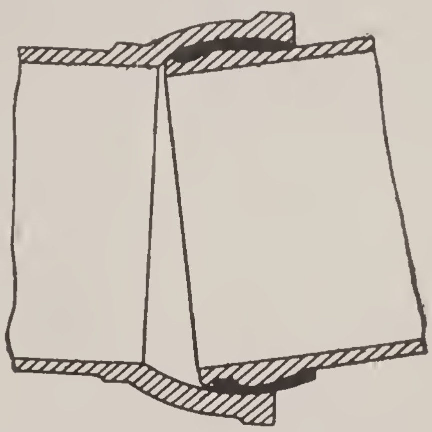

FIG. 173.—WARD FLEXIBLE JOINT.

2. Instead of connecting the entire pipe-line and lowering all together, it may be lowered in sections by the aid of flexible joints, each section consisting of several lengths of pipe connected in the usual manner. The pipe can thus be laid and lowered from a short piece of trestle or from a barge. This method is especially suitable for deep water where trestles cannot readily be used.

At Portland, Oregon, a 28-inch cast-iron pipe was laid from barges and trestles, the former being used in deep water and the latter in shallow water. The pipe was lowered from the barge by sliding it down a cradle extending from the barge to the river bottom. Flexible joints were used throughout.‡

At Columbus, Ga., an 18-inch main was laid by the use of twenty-four flexible joints. The pipe was put together on shore in 204-foot sections,

* *Eng. Record*, 1898, XXXVII. p. 97.

† *Eng. Record*, 1899, XL. p. 72.

‡ Trans. Am. Soc. C. E., 1895, XXXIII. p. 257. Several flexible joints are here described.

each terminating in one-half of a flexible joint. The sections were floated into place one by one, connected to the end of the previously laid portion and then sunk. All leaky joints were afterwards calked by divers.*

At Rochester, N. Y., a 60-inch steel intake-pipe was laid in sections 100 feet long, joined by means of flexible joints. The pipe was connected above water and lowered joint by joint by means of winches supported on pile platforms. The joint used was similar to that shown in Fig. 174.†

3. Many lines of submerged pipe have been laid by joining several lengths on shore, towing them into position, sinking them and connecting them by divers. This method is especially applicable for large pipe-lines. It has been used for large intakes at Syracuse and at Milwaukee; also at Galveston, Nashville, Boston, and many other places.

The method of laying the Syracuse intake was as follows: ‡ The 52-inch pipe was first riveted together in sections 116 feet long. The ends of these sections were then closed with oiled canvas bulkheads, rolled into the water, and floated to a position between the sections of a catamaran stationed over the pipe-trench and held in place by spud-piles. Ropes were then attached to the pipe, the bulkheads removed, and the pipe lowered to rest upon small timber foundations secured to the pipe before sinking. The joining was done by a diver. The special joint used in connecting the sections is illustrated in the right-hand portion of Fig. 174. A cast-iron hub is riveted to the end of

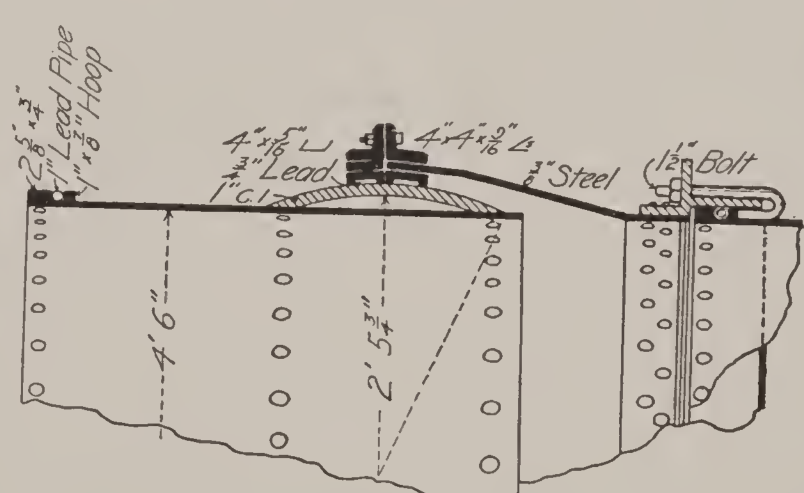

FIG. 174.—FLEXIBLE AND RIGID JOINTS, SYRACUSE INTAKE.

one section, and through this pass twenty hook-bolts. After guiding the pipes into place these hooks are brought to bear against a loose hoop of wrought iron placed on the end of the other section, and the nuts screwed up, thus closing up a lead-pipe gasket and forming a tight joint. Several flexible joints were used at changes of grade, and one of these is also illustrated in the figure. It is made by joining short pieces of pipe by a very broad lead joint run into the space between the two 4-inch channels and a cast-iron spigot. A 12-degree deflection is permitted. The cost for the pipe delivered was

* *Eng. Record*, 1899, XL. p. 97.

† *Eng. News*, 1895, XXXIII. p. 234.

‡ Trans. Am. Soc. C. E., 1895, XXXIV. p. 23.

$8.80 per foot, including seven flexible joints. The laying (exclusive of trenching) cost $2.50 per foot.

Similar joints were used on the Duluth intake at a cost of $82.00 each for the rigid joints, and $398.25 for the flexible joints. The 60-inch pipe there used cost $9.11 per foot delivered. Flexible joints very similar in design have also been used at Toronto and at Rochester, N. Y., as already noted.

The 60-inch cast-iron intake at Milwaukee was laid in 50-foot lengths and joined by a diver. The spigot end of each section was fitted with a temporary hub and poured with lead before it left shore. After entering it into the hub of the previously laid section it was then pulled tight by means of clamps as illustrated in Fig. 175. As much as 200 feet of pipe per day was laid by this method.*

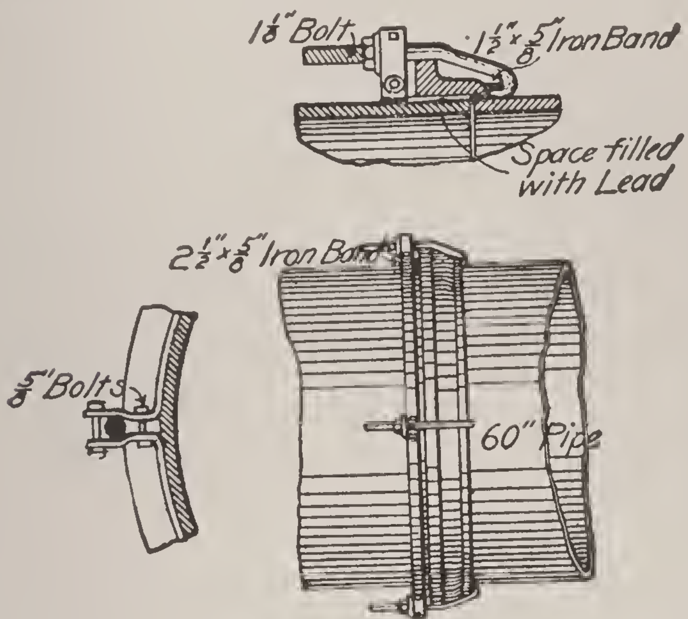

FIG. 175.—JOINT FOR CAST-IRON INTAKE, MILWAUKEE.

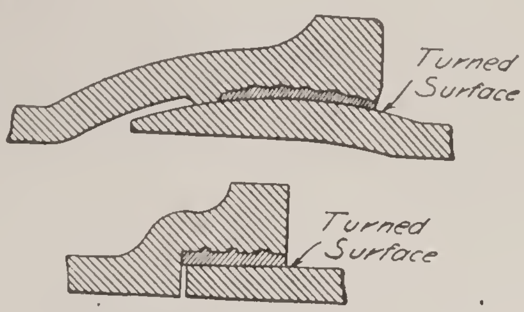

FIG. 176.—FLEXIBLE AND TAPER JOINTS, BOSTON.

In laying submerged pipe under the Charles River, Boston, three types of joints were used: (1) the ordinary joint with three turned grooves in the bell instead of one; (2) a taper joint for making subaqueous connections; and (3) a flexible joint. The last two are illustrated in Fig. 176. In making the taper joint the sections are put together, the joint run with lead, then the sections drawn apart, leaving the lead in place. A similar form of joint has been used in many places. In the flexible joint the spigot is turned to a spherical surface and cut off so as to permit a deflection of 1 in 10 without projecting into the waterway. It comes in contact with a rib on the bell turned to a close fit. In laying, the pipes were put together on a platform on shore in sections of 6 or 7 lengths, then towed in place and sunk. They were adjusted according to the direction of a diver, and the sections drawn together by a hydraulic cylinder attached to the pipe already laid. The joints were calked by the diver. Flexible joints were used where there were vertical deflections, or where settlement was to be feared.†

4. A method of laying submerged pipes sometimes used is to connect the entire pipe, or sections of it, on shore in a line in the direction of the proposed main, and to haul the pipe into the stream by a winch

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* *Eng. News*, 1895, XXXIV. p. 187.
† *Eng. Record*, 1898, XXXVII. p. 518.

on the opposite side, the pipe being at the same time floated by lashing it to empty barrels. At the Mersey River crossing of the Liverpool line a temporary pipe was laid by riveting up complete a 12-inch steel pipe and hauling the entire main across at one operation. In this method flexible joints should be used in sufficient number to permit of all necessary deflections from a straight line.

5. Very short crossings of deep channels may often be conveniently made by riveting up the pipe and sinking the entire structure at one operation. This method avoids obstructing the channel for any considerable time, and has been used in the case of several narrow navigable streams.

COST OF AQUEDUCTS AND PIPE-LINES.

654. Canals and Masonry Aqueducts.—The cost of conduits of this class can be quite closely estimated, if constructed in ordinary ground, from an itemized estimate of quantities, the unit prices being about the same as noted on pages 371 and 409. The unit prices used by Freeman in the report already referred to were based on the actual cost of the large Wachusett aqueduct of Boston. They were:

For Aqueducts in Cut-and-cover:	
Earth excavation	\$0.35 per cubic yard.
Earth borrow	0.30 " " "
Rock excavation	1.50 " " "
Portland-cement concrete	6.00 " " "
Natural-cement concrete	5.00 " " "
Brick lining	13.00 " " "
For Tunnels:	
Heading excavation	7.00 " " "
Bench excavation	4.00 " " "
Brick lining	13.00 " " "
Brick backing	6.50 " " "
Rubble backing	4.00 " " "
Dry backing	2.00 " " "
Portland-cement concrete in side lining	8.00 " " "
Natural-cement concrete in side lining	6.50 " " "

The Sudbury aqueduct of a cross-section equivalent in area to a circle $8\frac{1}{2}$ feet in diameter cost \$23.86 per foot, excluding special structures. The Wachusett aqueduct cost about \$24.00 per foot. Its cross-section is equal to a circle of 11.33 feet in diameter.

655. Pipe-lines.—The cost of pipe-lines will vary greatly according to the cost of the material used. This element can readily be ascertained at any time by reference to current price-lists, and the item of transportation can also be quite easily determined. For a good detailed analysis of amount and cost of labor, reference may be made to Weston's "Tables for Estimating the Cost of Laying Cast-iron Water-pipe," also to Billing's "Details of Water-works Construction," and to contract prices in the current periodicals. The data here given are intended only to give a general notion of the relative cost of work of this character. The formula already given (page 604) for the approximate cost of cast-iron pipe, furnished and laid, is

$$c = 20 + 2ad^{1.55},$$

where c = cost per foot in cents, a = cost of pipe per pound, d = diameter of pipe in inches. This formula has reference to work done under average conditions, and for earth-excavation, and refers only to ordinary sizes of pipe. From this formula the cost of various sizes is as follows:

Size of Pipe.	Cost per Foot.	
	Cost of Pipe = 1 cent per lb.	Cost of Pipe = $1\frac{1}{2}$ cents per lb.
4-inch........	$0.37	$0.46
6- "	.52	.68
8- "	.70	.95
10- "	.91	1.26
12- "	1.14	1.61
16- "	1.67	2.40
20- "	2.28	3.32
24- "	2.96	4.33

These figures are not intended to include the items of contractors' profit and of engineering.

The actual cost per foot of pipe, for distributing pipes, valves, etc., at Plainfield, N. J., is given as follows:*

	6-in.	8-in.	12-in.	16-in.
Pipes and specials..................	$0.394	$0.561	$0.965	$1.580
Lead and yarn......................	.04	.050	.087	.097
Valves and boxes....................	.041	.050	.054	.072
Tools and labor.....................	.124	.165	.177	.260
Contractor and engineering	.034	.063	.061	.089
Total..........................	$0.633	$0.889	$1.344	$2.098

* Trans. Am. Soc. C. E., 1894, XXXI. p. 375.

A similar statement for Alliance, Ohio, shows very nearly the same cost.*

In large cities where pavements are disturbed and the price of labor is high, the cost of pipe-laying is likely to be considerably more than given above.

For very large cast-iron pipes the cost of the iron, lead, and transportation forms such a large proportion of the total cost that a relatively close estimate can be made when these items are once known.

Table No. 78, compiled by Adams,† aims to give comparative figures for the cost of wood-stave, steel, and cast-iron pipe. The figures are based on a cost of cast-iron pipe of $19.00 per ton, and of steel of 1.6 cents for No. 14 B. W. G. plate to 1.25 cents for No. 8 and thicker. These prices are exceptionally low. They are now (1901) much higher, and the figures of the table should be correspondingly raised. The costs as given do not include hauling nor contractors' profits; they are intended to be used for comparative purposes only.

TABLE NO. 78.

COMPARATIVE COST OF PIPE AT CHICAGO (ADAMS).

(Including Laying, but not Hauling.)

Nominal Diameter.	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.	$\frac{1}{4}$-inch.	$\frac{5}{16}$-inch.	$\frac{3}{8}$-inch.	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	8.27	....	....	....	....	...	4.52	5.66	6.83	11.32	12.00	13.26	16.00

Regarding the actual cost of steel pipe-lines the following data are given:

* *Eng. News*, 1894, XXXI. p. 490.

† Trans. Am. Soc. C. E., 1899, XLI. p. 58.

The Rochester 38-inch steel conduit, $26\frac{1}{2}$ miles long, built in 1894, cost about $8.10 per foot, ready for use. The pipe was composed of $\frac{1}{4}$-, $\frac{5}{16}$-, and $\frac{3}{8}$-inch plates with lap-joints. The 40-inch steel pipe for Cambridge, Mass, built in 1895, 4.6 miles long, cost $4.81 per foot, or 3.13 cents per pound. The pipe thickness was $\frac{5}{16}$ inch, and lap-joints were used. The contract price of the Allegheny 60-inch steel pipe was about $8.50 per foot. The plates were $\frac{1}{2}$-inch thick. The pipe was built in 1896. Bids for the New Bedford 48-inch steel-pipe line in 1896 were, for the pipe alone, $5.10 per foot for lap-joints and $5.65 for butt-joints and countersunk rivets. The plates were $\frac{5}{16}$-inch thick, and length of line 8 miles. For the conduit complete the corresponding prices were $7.55 and $8.10 respectively.

Freeman estimates the cost of steel-pipe conduits of $\frac{3}{8}$-inch metal for New York City as follows: 4-foot pipe, $13.96 per foot; 5-foot-pipe, $16.70; 6-foot pipe, $19.50; 7-foot pipe, $22.50, etc.* These prices cover conduit complete, made with butt-joints and countersunk rivets, specially well coated, 10 per cent of length of trench in rock ledge, and assumes cost of steel at $2\frac{1}{2}$ cents per pound.

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CHAPTER XXVI.

PUMPING-MACHINERY.

656. Introductory.—The cost of pumping water is usually the greatest operating expense of a water-works system, and a city which can secure an adequate gravity supply has at once eliminated a most expensive and troublesome feature from the system. Pipe-lines, reservoirs, dams, and like structures, if well designed and properly constructed, are permanent and require but little attention and entail little expense for maintenance. All operating mechanisms, of whatever kind, require more or less constant attention and, however well designed and constructed, must, of necessity, be subjected to more or less wear and possible disarrangement and breakage.

The best results of intellect are secured by concentrated rather than by continuous effort. Proper consideration and supervision will secure well-designed and well-constructed works, but no care in the original design or in the construction will assure intelligent operation. In the course of time, intelligently designed works may come into the hands of unintelligent operatives and poor results follow. Poor designs, poor construction, and poor operation frequently entail large and unnecessary expense in the operation of pumping-plants.

Where water cannot be obtained at an elevation sufficient to admit of satisfactory gravity pressure at the points where it is to be used, pumping-machinery becomes necessary. It then becomes the duty of the engineer to design a pumping-plant which shall be the most economical for the condition under which the plant is to be established and operated.

This design involves the selection of—

1st. The best source of energy available for power purposes;

2d. The best means of generating such energy from a potential form, and for converting it into a form in which it can be utilized for power;

3d. The most economical means of transmission of the power so developed from the point of generation to the point of application; and

4th. The form of pump best adapted for the conditions under which it is to be operated.

These factors are often largely modified by the nature of the source of water-supply, and by various other features of a water-works system. All of these must be considered in connection with the selection of the pumping-plant, for many of them exert an important influence on the conditions under which the plant must be operated, and, therefore, often determine the type of the plant available for any particular purpose. (See Art. 688.)

In the generation, conversion, transmission, and application of power to pumping purposes, there are many losses of energy which add greatly to the expense of pumping. Many of these cannot be obviated; others can be removed, or at least reduced, by careful design. Careful analysis of each detail of the plant will determine the points at which a saving may be effected and will enable the engineer to reduce these losses to a minimum.

It is the purpose of this chapter to furnish an outline of the points to be examined in making such an investigation, and the methods to be used and factors to be considered in the selection of pumping-machinery; also, to describe briefly the various types of pumping-machinery which may be utilized, and their adaptability to different conditions.

657. Energy Expended in Pumping Water. — An expenditure of energy is entailed whenever motion is transmitted to a body. A portion of this energy is used in producing the velocity, a portion in overcoming frictional resistances ("lost" work), and a portion in overcoming other resistances involved in doing "useful" work, such as raising a body to a higher elevation, etc.

In hydraulic problems the energy expended in producing velocity may be readily transformed to pressure-energy by the familiar expression

$$h = \frac{v^2}{2g},$$

in which h = pressure-head,
v = velocity,
g = acceleration of gravity.

If q = volume of water moved, and w = weight of a unit of volume, then the work performed in producing the velocity v will be

$$\text{work} = qw\frac{v^2}{2g} = qwh.$$

Thus if $q = 10$ gallons, $w = 8\frac{1}{3}$ pounds, $v = 4$ feet per second, the work will be equal to $10 \times 8\frac{1}{3} \times \dfrac{4^2}{2g} = 20.69$ foot-pounds. If 10 gallons of water is moved each second, then the *rate* of work is 20.69 foot-pounds per second $= .37$ H.P.

The greatest expenditure of energy, however, is usually incurred in overcoming the resistance, or pressure, against which the motion is produced.

If $h_1 =$ the feet in height to which water is raised, then the useful work performed will be: Work (in foot-pounds) $= qwh_1$.

If $h_1 = 100$ feet, $q = 10$ gallons per second, $w = 8\frac{1}{3}$ pounds, then

$$\text{Rate of work} = qwh = 8330 \text{ foot-pounds per second} = 15.14 \text{ H.P.}$$

Again, if $h_2 =$ the friction-head, the work lost in friction will be, in foot-pounds, equal to qwh_2.

If $h_2 = 5$, with q and w as above, then the work lost to overcome friction will equal 416.5 foot-pounds per second, or about .75 H.P., and the total work done by the pump will be

$$\text{Work (in foot-pounds)} = qw(h + h_1 + h_2).$$

In pumping water the largest expenditure should be and usually is due (except in the case of very long pipe-lines) to overcoming the resistance of gravity, or in useful work, although the energy used in acquiring velocity and in overcoming the frictional resistance of passages and conduits through which water passes may be considerable, especially in poorly designed machinery or ill-devised pipe systems.

In overcoming gravity or other resistance, the quantity of water raised and the resistance overcome are the measures of energy expended. In certain cases the energy used in producing velocity may be returned in work done without loss. In other cases such energy cannot be utilized.

Table No. 79 shows the equivalence of various units of quantity,

TABLE NO. 79.

EQUIVALENT MEASURES AND WEIGHTS OF WATER AT 4° CENT. = 39.2° FAHR.

U. S. Gallons.	Cubic Feet.	Cubic Inches.	Imperial Gallons.	Liters.	Cubic Meters.	Pounds.
1	.13368	231	.83321	3.7853	.0037853	8.34112
7.48055	1	1728	6.23287	28.3161	.0283161	62.3961
.004329	.000579	1	.003607	.016387	.0000164	.03611
1.20017	.160439	277.274	1	4.54303	.0045303	10.0108
.264179	.035316	61.0254	.22012	1	.001	2.20355
264.179	35.31563	61025.4	220.117	1000	1	2203.55
.119888	.016027	27.694	.099892	.453813	.0004538	1

and Table No. 80 shows the equivalence of various pressure

TABLE NO. 80.

PRESSURE EQUIVALENTS.

Feet Head.	Pounds per Sq. In.	Pounds per Sq. Foot.	Pounds per Circular Inch.	Inches of Mercury, 32° Fahr.	Kilograms per Sq. Centimeter.	Atmosphere.
1	.4335	62.425	.3413	.882	.03047	.02945
2.307	1	144	.7854	2.0379	.07029	.06794
.01602	.006939	1	.00545	.01414	.000487	.00047
2.937	1.273	183.3	1	2.594	.08952	.08649
1.133	.4912	70.73	.3858	1	.03453	.03334
32.821	14.225	2047.8	11.174	28.992	1	.96652
33.95	14.72	2119.7	11.562	29.921	1.035	1

resistances, which, in any particular case, may be due to gravity or to other causes, such as the resistance of the spring of a relief-valve, or the friction through pipe, hose, nozzle, etc. In pumping water these two tables include the elements of the useful work done.

658. Work and Power Equivalents.—Work is a phenomenon of energy; it is the overcoming of a resistance by a force acting through space. Power is the rate of work and involves the idea of force, space, and time.

In pumping water any form of energy may produce a force which, when properly applied, will overcome a given resistance, such as a given head or pressure. In doing this, work is performed. If a definite weight or quantity of water is pumped against a definite head or pressure in a given time, certain work is done each unit of time. Thus a rate of work is established and certain power is expended. Both work and power in pumping are therefore measured by the quantity of water pumped and the resistance pumped against. For considering power, the second, minute, hour, or day are the units of time used.

Table No. 81 gives the equivalence of units of energy or work, i.e., the idea of time is not involved.

Besides the units of work given in Table No. 81 the various power-units, when limited to a definite time, may also be used to designate a definite amount of work, in which the unit of time is used to limit the quantity of work to be performed, but does not necessarily involve the idea of the time in which the said work is performed. Various fuels and other forms of potential energy may also be expressed directly in foot-pounds. Table No. 82 shows the relation or equivalence of such units in foot-pounds.

TABLE NO. 81.

EQUIVALENT UNITS OF ENERGY.

Work.	Heat, B.T.U.	Electricity.	Hydraulic Energy.			
Foot-pounds.	Degree-pounds.	Volt-coulombs.	Foot-gallons.	Foot-cubic-feet.	Pound-gallons.	Pound-cubic feet.
1	.001285	.0003766	.12	.016	.0519	.0069
778	1	.2929	93.28	12.448	40.394	5.368
2655.4	3.414	1	318.39	42.486	137.87	183.23
8.341	.01072	.003140	1	.1334	.433	.05754
62.39	.08033	.02353	7.48	1	3.245	.4312
19.259	.02476	.007255	2.309	.30816	1	.1329
144.92	.18630	.05457	17.37	2.318	7.524	1

TABLE NO. 82.

WORK EQUIVALENTS.

Unit.	Equivalent in Foot-pounds.
Horse-power hour	1,980,000
Kilo-watt hour	2,654,150
Pound of steam (approximate)	778,000
British thermal unit	778
Pound of carbon (approximate)	11,500,000
Pound of hard coal (approximate)	11,400,000
Pound of soft coal (approximate)	9,910,000
1,000,000 gallons 1 foot high (water)	8,341,000
1000 gallons 100 feet high (water)	834,100
100 cubic feet 1 foot high (water)	62,396

In many problems of pumping, questions of power, rather than work, are involved. That is, a definite rate of pumping must be considered. Table No. 83 gives the equivalence of common power units used for such problems.

TABLE NO. 83.

EQUIVALENT UNITS OF POWER.

Work.		Heat.	Electricity.	Water-power.			
Foot-lbs. per Minute.	Horse-power.	Thermal Units per Minute, B.T.U.	Watts.	Foot-gallons per Minute.	Foot-cubic-feet per Minute.	Pound-gallons, per Minute,	Pound cubic-feet, per Minute.
1	.0000303	.001285	.0226	.12	.016	.0519	.0069
33,000	1	42.416	746	3960	528	1713.4	229.05
778	.02357	1	17.58	93.28	12.444	40.394	5.388
44.24	.00134	.0568	1	5.308	.70895	2.296	.307
8.34	.00025	.0107	.18856	1	.1337	.433	.0579
62.396	.00189	.0802	1.4105	7.48	1	3.24	.433
19.26	.00058	.0247	.435	2.31	.309	1	.1337
144.92	.00436	.1851	.0326	17.37	2.31	7.48	1

659. Classification of Energy Losses.—If it were possible to perform work or to utilize power without loss, the amount of energy which would be necessary to raise any given quantity of water in any given case could be ascertained from the preceding tables. Energy loss is, however, concomitant with the use of all machinery, and differs with the class and complication of the machinery through or in which the energy is generated, transformed, transmitted, and utilized.

In general the operation of applying energy to pumping water consists of the generation of energy from a potential source, the conversion

TABLE NO. 84.

CLASSIFICATION OF ENERGY LOSSES IN PUMPING.

			Losses.
Generation.	Fuel	Internal-combustion engine. (Gas; Oil.)	Engine losses.
		Steam	Furnace. Boiler. Piping.
	Water-power	Direct (ram).	Ram losses.
		Indirect (wheels).	Velocity losses. Wheel losses.
	Minor sources	Electric (primary batteries). Wind (mills). Waves (motors). Sun heat (solar engines).	Various mechanical and other losses due to method used.
Conversion.	Internal-combustion Engine		Included in engine losses.
	Steam		Engine and connection losses.
	Electrical		Dynamo losses.
	Hydraulic		Pump losses.
	Pneumatic		Compressor losses.
Transmission.	Mechanical	Direct-connected shaft. Belt; Rope; Chain; Combination.	Various losses, due to the method used.
	Hydraulic		Pipe friction. Motor losses. Connections.
	Electrical		Transformer losses. Wire losses. Motor losses. Connections.
	Pneumatic		Pipe friction. Air cooling. Motor losses. Connections.
Application.	Pumping	Inlet-pipe	Influx. Velocity. Friction.
		Pump	Friction in valves and water passages. Mechanical friction.
		Discharge-pipe	Pipe friction.
	Ram		Pipe losses.
	Steam		Radiation. Condensation. Pipe losses.
	Air		Air-pipe losses.

of such energy into a kinetic form, its transmission to the location of the pump, and its application to pumping the water by means of the pumping machinery used.

The economical application of pumping-machinery depends on the reduction of all energy losses to the lowest practical amount. These losses should always be examined in detail, and in order that they shall in no case be overlooked it is well to examine the losses under the following heads:

Generation Losses.
Conversion Losses.
Transmission Losses.
Application Losses.

These principal divisions should be further subdivided for detailed consideration as shown in Table No. 84, the items of which will next be considered.

SOURCES OF POTENTIAL ENERGY.

660. Available Sources.—The sources of potential energy available for power purposes are fuel, water-power, wind, solar energy, and chemical energy (which properly also includes the energy of fuel which is rendered kinetic through chemical union). Fuel is most generally utilized by means of heat and heat-engines, water-power by means of various forms of water-motors, the wind by means of windmills,

TABLE NO. 85.

FUELS: CALORIFIC VALUE AND EQUIVALENTS.

Fuel.	Average Heat-units.		Equivalent Evaporation from and at 212° Fahr. Pounds.	Equivalent Horse-power Hours.	Equivalent Million Foot-gallons.
	Per Pound.	Per 1000 Cubic Feet.			
Equivalents for Calculations	10,000	....	10.35	3.87	.921
Coke	14,880	....	15.40	5.84	1.384
Anthracite coal	14,660	....	15.17	5.76	1.365
Bituminous coal	12,740	....	13.18	5.00	1.185
Wood	7,740	....	8.01	3.04	.720
* Petroleum	19,150	....	19.82	7.52	1.782
Natural gas	....	885,880	917.06	348.08	82.494
Coal-gas	....	570,900	599 93	224.32	53.164
Water-gas	....	253,100	262.00	99.45	23.569
Producer-gas	....	111,190	115.10	43.69	10.354
Equivalents for Calculations	....	100,000	103.50	38.90	9.219

* Gasoline, the form of petroleum used in the gasoline engine, weighs about 5.8 lbs. to the gallon. Each gallon contains about 110,000 B. T. U.

chemical energy by means of electric batteries, and solar energy by means of solar engines. In a commercial way, fuel and water-power only are of great importance or need to be here considered.

661. Fuel.—Fuel is the source of potential energy most widely used commercially. From wood, coal, petroleum, natural gas, and other fuels, energy is developed in the form of heat by combustion.

The average value of various fuels is approximately as shown in Table No. 85.

662. Water-power.—Water-power is readily convertible by easy calculations into water pumped. Without loss, 1 foot-pound of water for power purposes should give 1 foot-pound of water pumped. For example, 10 pounds of water falling 10 feet possess 100 pounds of energy and would, if utilized without loss, raise 1 pound of water 100 feet, 2 pounds 50 feet, 4 pounds 25 feet, etc.

If utilized without loss, we have said; but it has already been stated that utilization of energy without loss is impossible, hence the above proportions of work done are in practice materially reduced.

GENERATION AND CONVERSION OF ENERGY.

663. Ordinary Efficiency of Generators and Motors.—The proportion of energy which can be realized in useful work will depend on the efficiency of the machines by means of which energy is being utilized. By efficiency is meant the ratio of energy utilized in useful work done to energy applied for power purposes.

Any prime mover may be utilized by proper connection with a suitable pump for the purpose of pumping water. The loss of energy in any case will depend both on the type of machine used and the design, construction, and operation of the particular machine in question. The results usually attained in good practice with various generators and motors which may be utilized for pumping are shown in Table No. 86.

664. The Steam-boiler.—Fuel energy is most commonly converted into a form in which it can be applied for power purposes by means of the steam-boiler, although the internal-combustion engine has also now become an important factor for power installations.

On account of heat lost in the waste gases from the boiler-furnace, only about 83 per cent of the calorific value of the fuel can theoretically be made available without the use of economizers and forced draft. The best boilers will utilize about 90 per cent of this available energy, or about 75 per cent of the full calorific power of the fuel used.

TABLE NO. 86.

ORDINARY EFFICIENCIES OF GENERATORS AND MOTORS.

Type of Machine.		Ordinary Efficiency at Full Load. Minimum.	Maximum.
Water-wheels:			
Overshot wheels		65	75
Breast wheel		60	65
Undershot wheel		25	40
Turbines		60	85
Impulse-wheel		75	85
Steam generators: Boilers		50	75
Steam-engines:			
Condensing	Triple-expansion Corliss	15	18
	Compound Corliss	12	15
	Simple Corliss	10	12
	Compound high-speed	10	12
Non-condensing	Compound Corliss	10	12
	Simple Corliss	7	9
	Compound high-speed	7	9
	Simple high-speed	6	7
	Simple slide-valve	5	7
Heat engines:			
Gas- or oil-engines		16	20
Diesel motor		25	30
Steam air-compressors:			
Compound condensing Corliss		10	12
Simple condensing Corliss		7	9
Simple Corliss		5	6
High-pressure		3	4.5
Small straight-line		2	3
Electrical machinery:			
Dynamos		80	92
Motor (large)		80	90
Motor (small)		75	85
Transformer		50	95

The greatest care is necessary in the design and construction of furnace, boiler, and accessories in order to develop the maximum efficiency and secure the most economical results in the utilization of fuels. Radiation and condensation are important factors in boiler losses and should be rendered as small as possible by properly protecting the boiler. The same losses are also important in the steam-pipes which transmit the steam from generator to motor and must be kept at a minimum by proper precautions.

665. **The Steam-engine.**—Of the energy delivered to the engine, the proportion actually utilized depends upon the character of the engine used, its design, and the condition in which it is maintained.

A perfect engine, on account of the nature of steam, could utilize only about 25 per cent of the energy of the steam delivered to it. In actual practice, however, the best engines utilize only about 17 per cent, and poor engines in poor condition frequently utilize less than 1 per cent of the energy of the steam.

If the steam-consumption per actual horse-power per hour for any engine is known, the efficiency of the engine can be readily determined from Table No. 82.

666. *Use of Steam Expansively.*—In the simplest form of steam-engines and of steam-pumps, the steam at full pressure follows the piston for the full length of every stroke and the expansive force of the steam is not utilized. This is the case in direct-acting, reciprocating, high-pressure steam-pumps. In higher types of steam-pumps and in almost all types of steam-engines the steam is cut off after a portion of the stroke is completed, and the steam is allowed to expand for the balance of the stroke.

In Fig. 177 let ab represent the pressure on the steam-piston, and

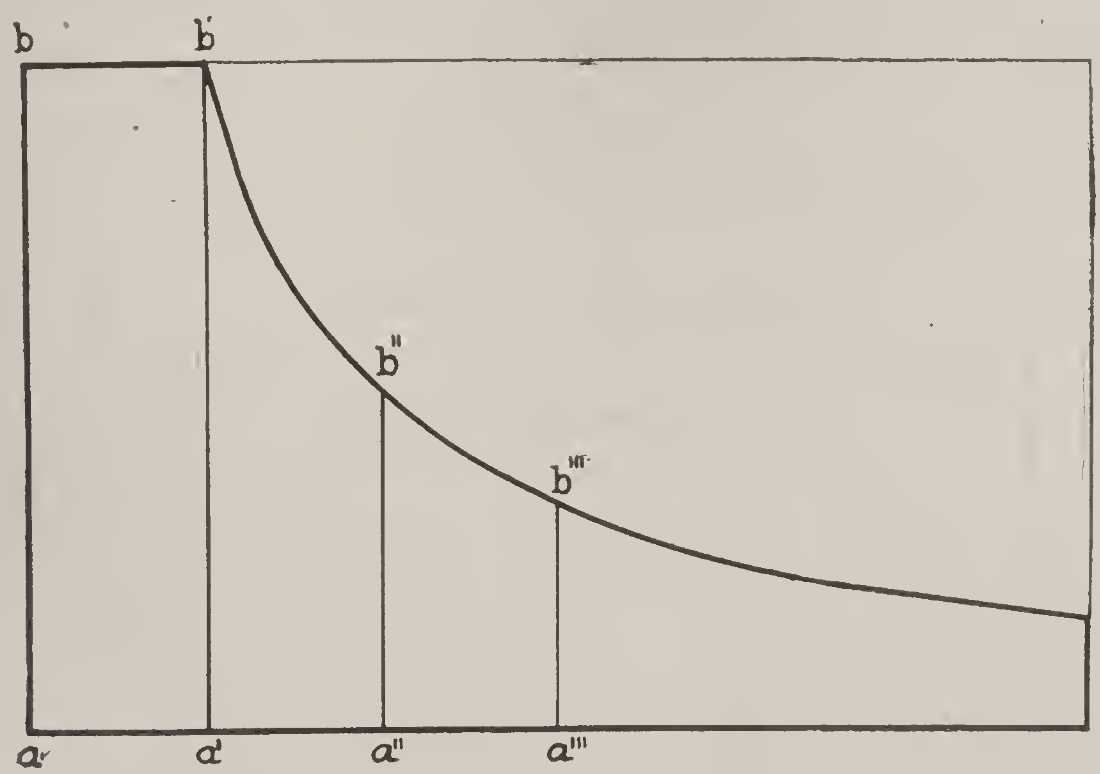


FIG 177.

aa' the space passed through by the piston; then their product, represented by area $abb'a'$, equals the work done by a unit of steam with pressure ab, and which follows the piston for a space aa'. Now if at $a'b'$ the steam-supply is cut off and the piston still advances to position $a''b''$ or $a'''b'''$, etc., the expansive force of the steam will still cause a pressure to be exerted against the piston which will decrease in amount as the piston advances, but which is nevertheless adding constantly to the work done, as shown by the area which represents in the diagram the work of a unit of steam. This additional work is obtained, it should be noted, with no additional expenditure of steam. The additional work done by each steam-unit depends on the degree of expansion obtained, which in turns depends on the type of engine or pump used, and various other considerations which cannot be discussed here.

From Fig. 177 it is seen that if the steam is allowed to expand and the pistons to increase its stroke, the power obtained will be increased. The power obtained from a cylinder of given size will, however, be greater when the steam is carried for the full length of the stroke.

When the steam-supply is cut off at a fraction of the stroke and allowed to expand for the remainder of the distance, the average or mean effective pressure (M.E.P.) decreases and the horse-power of the engine will likewise decrease unless the pressure of the steam is increased sufficiently to offset the loss in M.E.P. thus occasioned. In Table No. 87 is shown the initial pressure (column 3) needed to maintain a M.E.P. of 75 pounds, or, in other words, the necessary initial pressure needed with various cut-offs to obtain the same horse-power from the same size steam-cylinder.

TABLE NO. 87.

ECONOMY SECURED BY USING STEAM EXPANSIVELY.

Point of Cut-off.	Boiler-pressure required to give Same Power of Engine.			Per Cent of Saving in Fuel.
	When Used Full Stroke.	When Cut Off.	Ratio.	
1	75	75	1	0
$\frac{7}{8}$	75	76	1.01	12
$\frac{3}{4}$	75	77	1.03	22
$\frac{5}{8}$	75	82	1.09	32
$\frac{1}{2}$	75	88.5	1.18	41
$\frac{3}{8}$	75	99	1.32	50
$\frac{1}{4}$	75	125.5	1.67	58
$\frac{1}{8}$	75	195	2.60	68

From this it is seen that high rates of expansion require either high initial steam-pressure or large engines. And the limit is soon reached at which the economy of a greater degree of expansion is offset by the extra cost of the engine necessary to obtain it. Cylinder condensation and various questions of construction also enter into the question and become important factors.

The relative theoretical saving effected by different degrees of expansion are shown in the fifth column of Table No. 87.

667. *Use of Condensers.*—If the steam, after being used in the engine or pump-cylinder, is exhausted into the atmosphere, the piston

will work against a back pressure equal to atmospheric pressure (about 14.7 pounds) plus the friction in exhaust-passages (usually from 2 to 3 pounds). If the exhaust passes into a condenser, the back pressure is relieved in proportion to the vacuum carried.

Usually a condenser will add about 10 pounds to the mean effective pressure in the engine-cylinder. The percentage of saving by it will be the ratio of pressure added to mean effective pressure which would otherwise be developed in the cylinder, or

$$\frac{10}{\text{M.E.P.}} = \text{percentage saved.}$$

Dr. C. E. Emery* gives Table No. 88, showing the estimated

TABLE NO. 88.

PERCENTAGE OF GAIN BY USE OF CONDENSER.

Type of Engine.	Pounds Steam per Indicated H.P. per Hour.				Per Cent Gained.
	Non-condensing.		Condensing.		
	Probable Limits.	Assumed for Comparison.	Probable Limits.	Assumed for Comparison.	
Simple high-speed..........	35 to 26	....	25 to 19	22	33
Simple low-speed............	32 to 24	29	24 to 18	20	31
Compound high-speed	30 to 22	26	24 to 16	20	23
Triple high-speed...........	27 to 21	24	23 to 14	17	29

steam consumption of various engines and the saving effected by the use of condensers.

668. *Average Steam Consumption.* — The approximate average steam consumption per indicated horse-power for various classes of engines is shown in Table No. 89.

TABLE NO. 89.

APPROXIMATE STEAM CONSUMPTION PER I.H.P. OF VARIOUS TYPES OF STEAM-ENGINES.

Pound of Steam per Indicated Horse-power per Hour at Full Load.

Triple-expansion condensing Corliss..........	12 to 14 pounds.
Compound condensing Corliss................	14 to 18 "
Simple condensing Corliss....................	18 to 21 "
Compound Corliss..........................	18 to 21 "
Compound condensing automatic............	20 to 24 "
Simple Corliss..............................	24 to 30 "
Simple high-speed..........................	30 to 36 "
Simple slide-valve..........................	33 to 45 "

* Trans. Am. Inst. E. E., March, 1893.

669. *Effect of Operating at Partial Load.*—From 6 to 15 per cent of the indicated horse-power of an engine is lost in friction in well-designed engines at full load. At partial load the percentage of loss is much greater.

All machinery gives its greatest efficiency when operated at or near its maximum capacity. This is due to the fact that the friction of most machinery is practically constant at all loads, or nearly so. With a 100 horse-power engine the friction of the engine will be perhaps about 10 horse-power, hence the following condition will result:

Useful Load.	Friction Load.	Mechanical Efficiency.
With no load	10 H.P.	0%
With 10 H.P	10 H.P.	50%
With 30 H.P	10 H.P.	75%
With 70 H.P	10 H.P.	87½%
With 100 H.P.	10 H.P.	91%

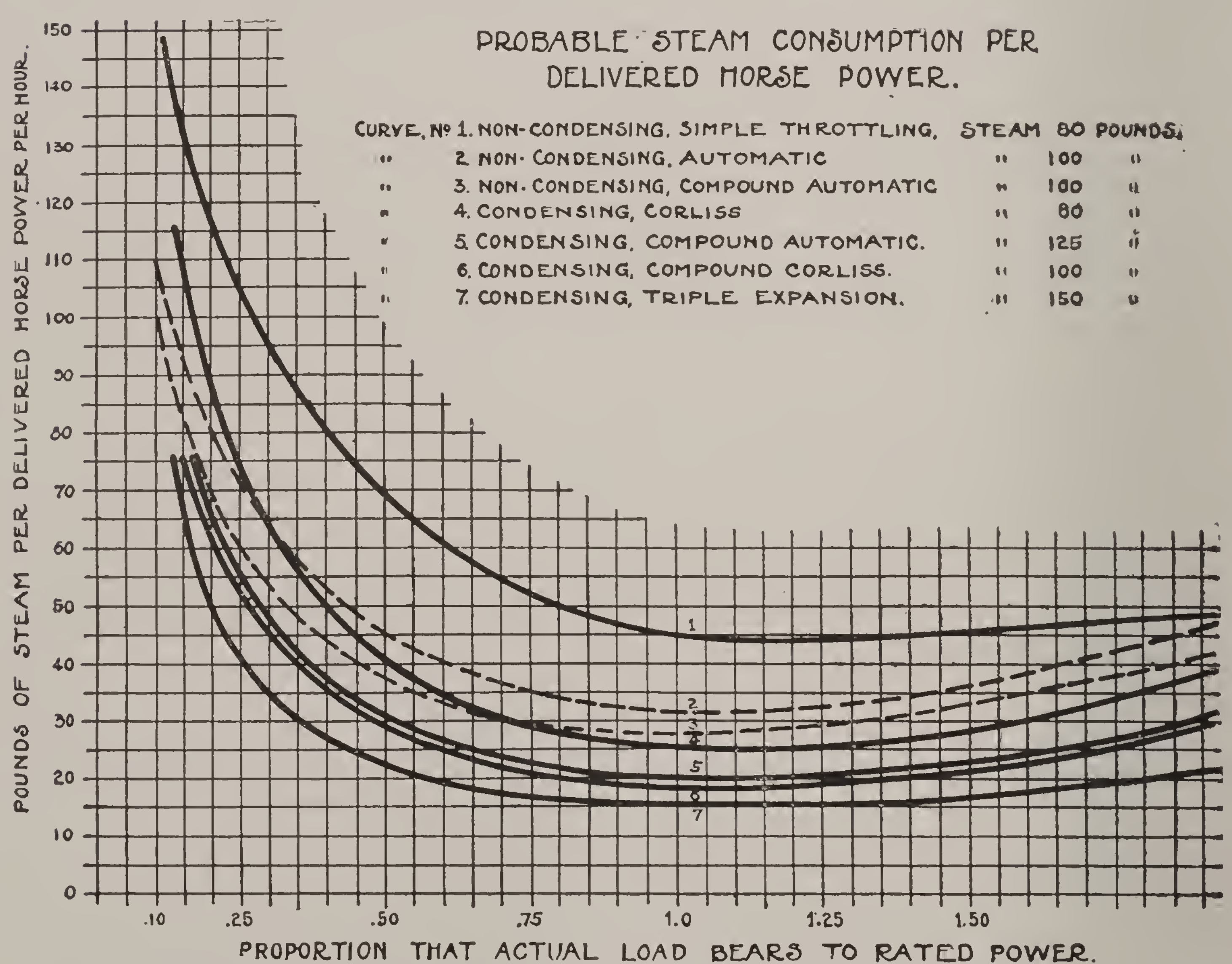


FIG. 178.—STEAM CONSUMPTION FOR VARIOUS CLASSES OF ENGINES.

The steam consumption of various classes of engines at various loads is well illustrated by the diagram of Fig. 178, modified slightly from a table by Prof. R. C. Carpenter.

670. Heat-engines.—Only about 12 per cent of the fuel energy is utilized in the indicated horse-power of the best steam-engines, while in ordinary practice only from 1 to 3 per cent is so utilized. As the loss is largely due to the nature of steam, it has resulted in attracting the attention of inventors to other forms of heat-engines for power purposes. The best-known forms of these are the various gas- and gasoline-engines in which a mixture of air and gas or vapor is ignited or exploded in the engine-cylinder itself, without the interposition of a boiler. These engines utilize from 16 to 20 per cent of the calorific value of the fuel used, and in some special forms 30 per cent has been utilized. These engines are also available for power without the slow process of getting up steam, an important matter in operating pumping-plants for fire service. Another favorable condition is the small amount of attention necessary in their operation.

The availability of this class of motors in entirely a matter of condition, which may be adverse or favorable in any locality or for any

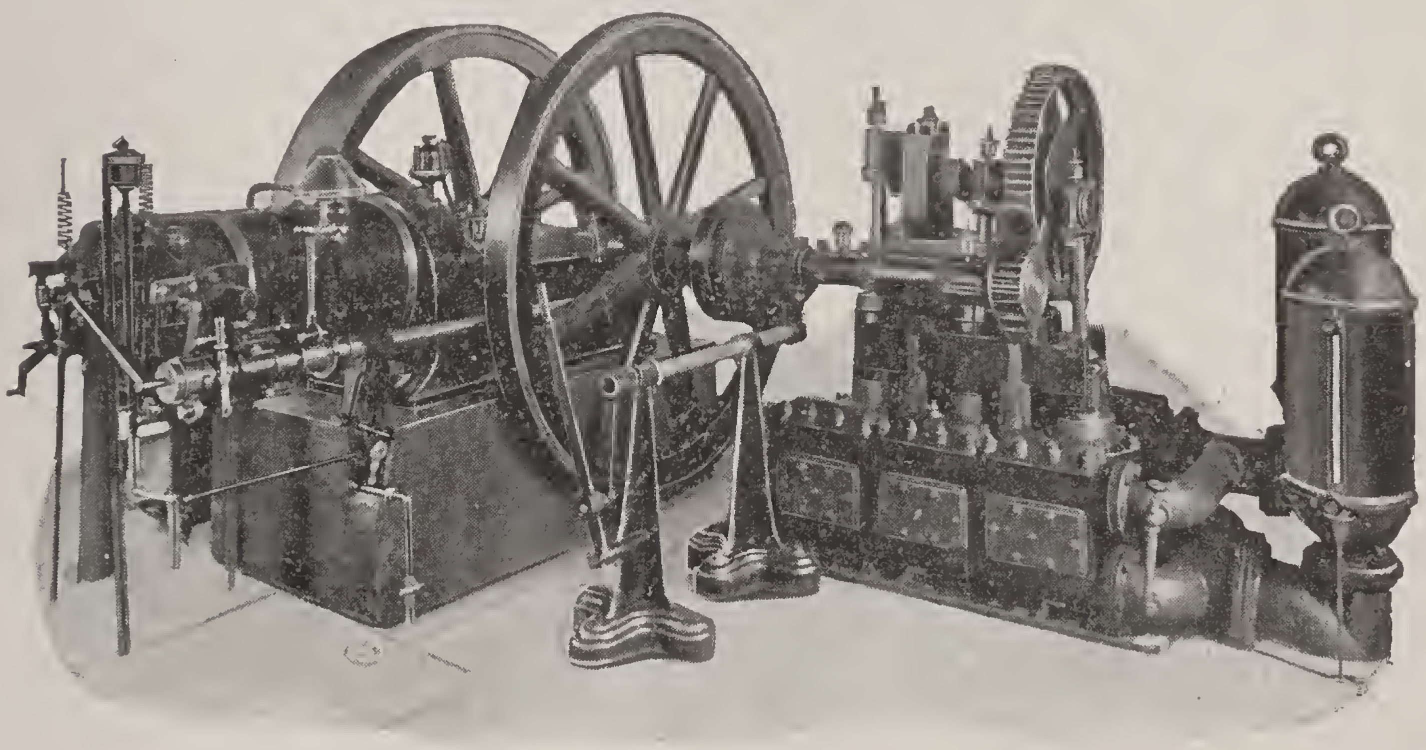


Fig. 179.—Gasoline Pumping-plant, Dundee, Illinois.

purpose. Fig. 179 shows one of two gasoline-engines with direct-connected pumps installed in the water-works at Dundee, Ill.

While these engines require but comparatively little attention, they must have such attention, and the expense of this must not be underestimated. Where other work can be done by the engineer, only a portion of his time need be charged to the operation of the plant, but where no such work is possible his entire time must be considered.

The average amount of fuel required by these engines at full load is about as follows:

Gas-engines: 12 to 15 cu.ft. of natural gas per actual H.P. per hour.
18 to 22 cu. ft. of coal-gas per actual H.P. per hour.
Gasoline-engines: .11 to .14 gal. of gasoline per actual H.P. per hour.

The makers of high-grade gas-engines will guarantee to develop an actual horse-power hour on 12,600 B.T.U.

TRANSMISSION OF ENERGY.

Having developed the power at the shaft of the engine or water-wheel, it must next be transmitted to the pump. This operation also involves some waste of energy.

671. Methods of Transmission and Approximate Efficiencies.—*Direct Connection.*—When the machine to which transfer is made is connected directly to the motor without interposition of extra boxes or gearing and with shafting directly in line, no extra friction is involved and no loss is sustained. (See Figs. 179 and 180.)

Shafting.—Where a long shaft is directly connected to the source of power without gearing, the loss is in proportion to the number of bearings, their lubrication, arrangement, and alignment. In shop systems the losses, including belt and shaft systems, are often from 15 to 50 per cent with full loads, and are much greater proportionally for light loads.

Gearing.—Bevel-gearing, used to turn a right angle with shafting, frequently uses from 15 to 25 per cent of power transmitted, even where cut gears are used.

In gear-trains or worm-gearing, 40 per cent of the power, or more, may be consumed, according to the construction and complication.

Belts.—The loss in simple belts is usually from 5 to 15 per cent; tight belts cause excessive friction in bearings and consequently large losses.

Rope Gearing.—With the American system of rope transmission the loss is less than with belts, and in single transmission will vary from 3 to 10 per cent. (See Fig. 62, page 316.)

Wire-rope Gearing—Unwin gives the efficiency of wire-rope gearing at full load from Zeigler's experiments as follows:

$$\text{Efficiency} = .967^{\left(\frac{M+2}{2}\right)},$$

for M intermediate and two terminal stations.

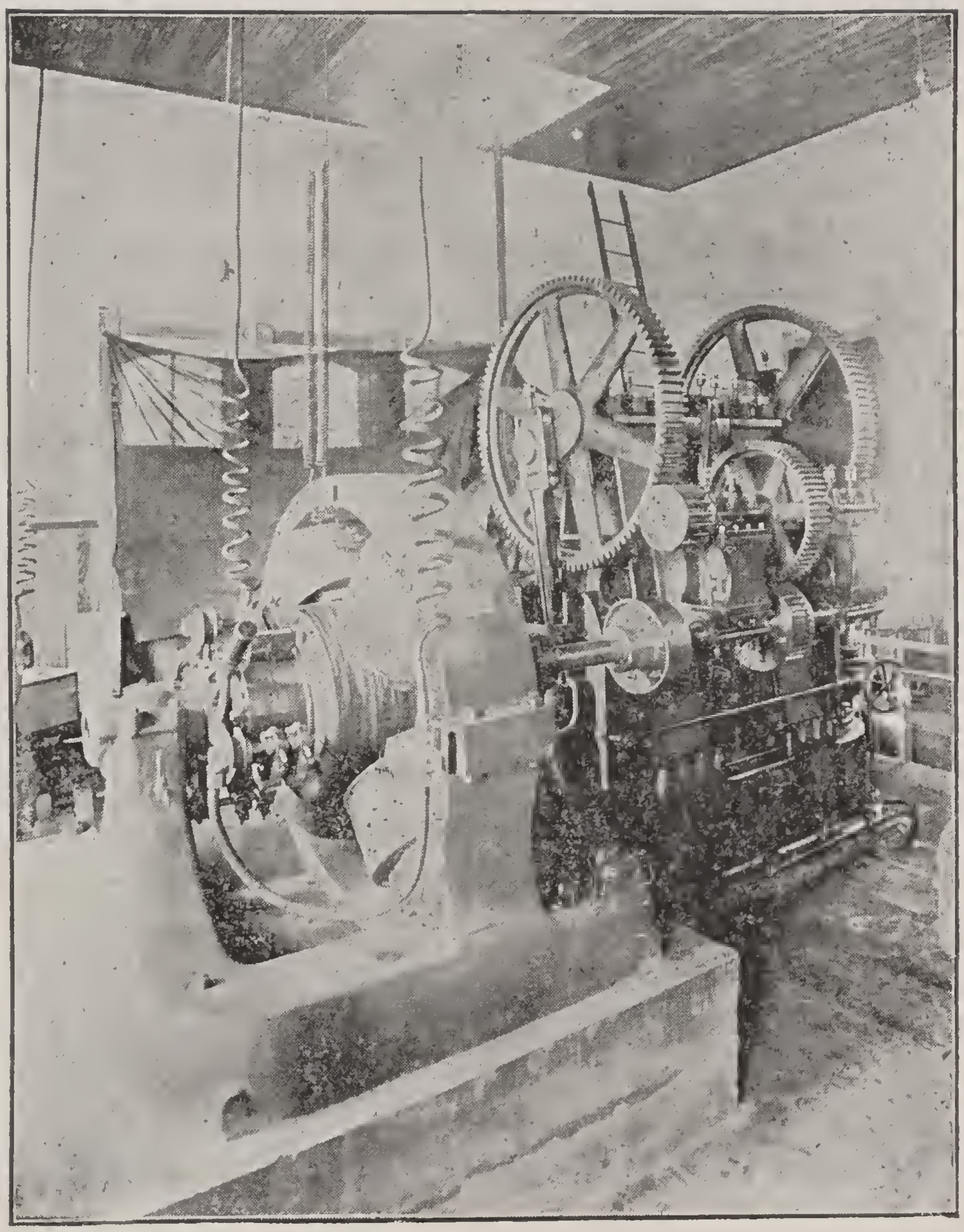


FIG. 180.—ELECTRIC PUMPING-PLANT, DEKALB, ILLINOIS.
Front View.

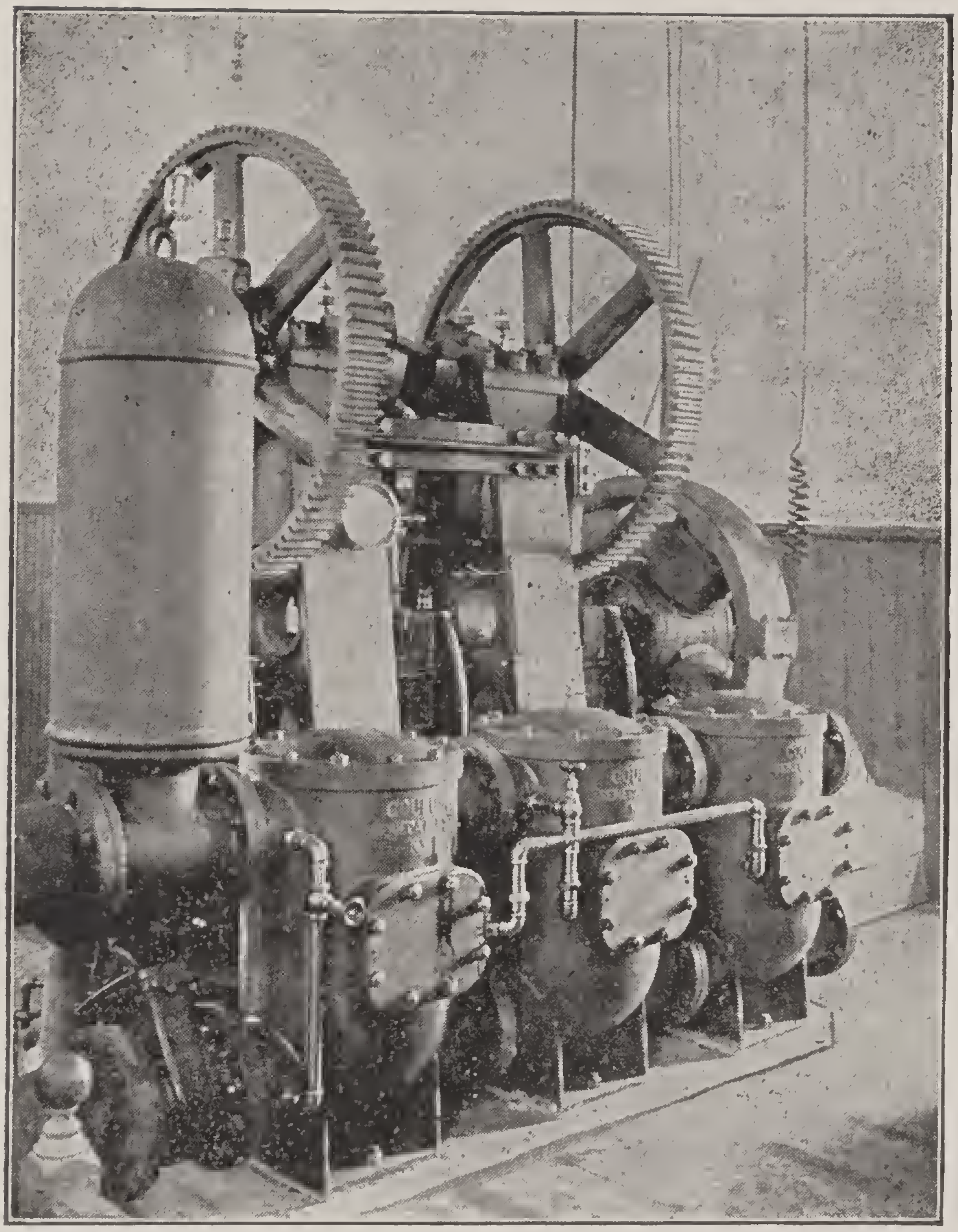


FIG. 180.—ELECTRIC PUMPING-PLANT, DEKALB, ILLINOIS.
Rear View.

Pneumatic Transmission.—Friction losses are from 3 to 8 per cent per mile. The efficiency of motors varies from 40 per cent, when air is used cold, to as high as 70 per cent, when the air is reheated before use.

Hydraulic Transmission.—This method of transmission can be calculated from friction tables, and the efficiencies of the class of pumps used as given herein.

Electrical Transmission.—The efficiencies of electrical transmission can be calculated from the efficiencies of the electrical machinery, together with the line and transformer losses, which in good practice is not more than from 5 to 10 per cent.

THE PUMP IN GENERAL.

672. Classification of Pumps.—The function of all pumping-machinery for water-works purposes is to take water from some given source and move it to a new position.

TABLE NO. 90.

CLASSIFICATION OF PUMPS.

- PUMPS.
  - DISPLACEMENT. Absolute volume displaced.
    - Reciprocating.
      - Action........
        - Double.
        - Single.
      - Class.........
        - Piston.
        - Plunger.
          - Inside-packed.
          - Outside-packed.
          - Center-packed.
      - Type.........
        - Power...
          - Single.
          - Duplex.
          - Triplex.
        - Steam.
          - Application...
            - High-pressure.
            - Compound.
          - Arrangement.
            - Direct-acting.
            - Crank & fly-wheel.
            - Compensator.
        - Hydraulic.
          - Direct-acting.
          - Crank and fly-wheel.
      - Arrangement.
        - Vertical.
        - Horizontal.
      - Use..........
        - Surface (suction).
        - Submerged or deep well.
    - Rotary.
    - Air-displacement.
    - Steam-vacuum.
    - Continuous-flow...
      - Screw.
      - Chain.
      - U pump.
      - Double-acting.
  - IMPELLER. Continuous application through some mechanical agency or medium.
    - Centrifugal...
      - Impeller.......
        - Opened.
        - Closed.
      - Case..... .....
        - Side suction.
        - Double suction.
      - Arrrangement.
        - Horizontal.
        - Vertical.
    - Jet.. ....
      - Water.
      - Steam.
      - Air.
  - IMPULSE (as name implies)........................ Water-ram.
  - BUCKET (receptacle alternately filled and emptied)..
    - Wheel.
    - Band.

Pumps may be classified in various ways, but for the consideration of their mechanical action they may be best considered under the following heads:

1. Displacement-pumps.
2. Impeller-pumps.
3. Impulse-pumps.
4. Bucket-pumps.

The various subdivisions of the classification are shown in Table No. 90.

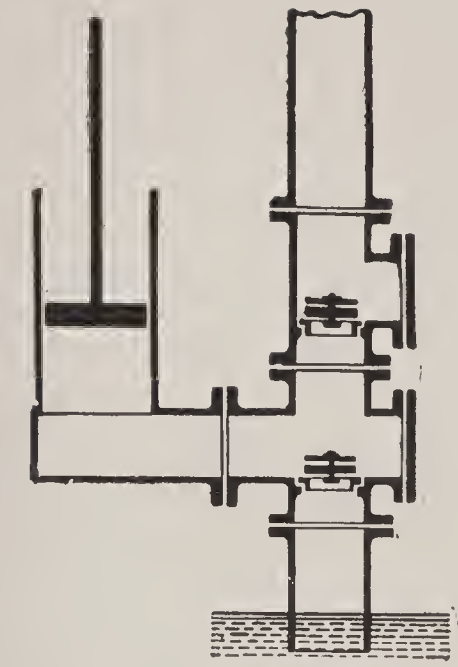
(1) *Displacement-pumps.*

673. Displacement-pumps are those in which the volume of water raised is forced from the pump-chamber by absolute displacement by some mechanical agency.

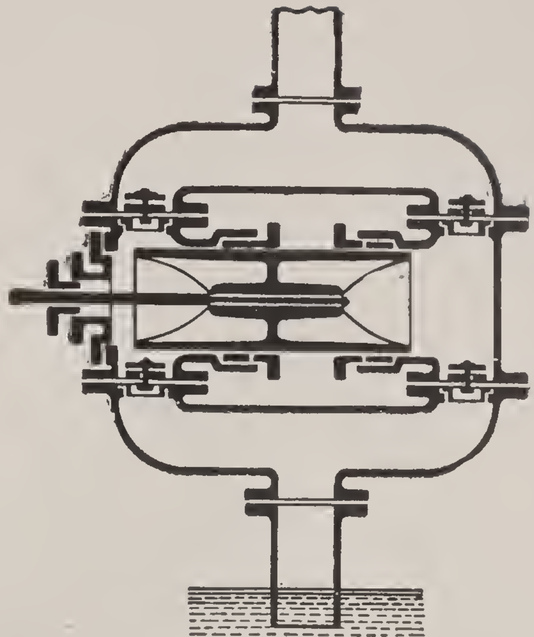
674. Reciprocating-pumps.—Of displacement-pumps the ordinary reciprocating-pump is the most common and well-known variety. In reciprocating-pumps a piston or plunger (which is the displacing agency) reciprocates in a closed cylinder provided with the necessary inlet- and outlet-valves, and alternately inspires and discharges the water from the cylinder. Such pumps are single-acting when one end of the plunger only acts on the fluid column (Figs. 181(*a*) and 187), and are double-acting when the cylinder is so constructed that the pump will act on both the forward and the return stroke (Figs. 181(*b*), 183, 184, and 186). Piston-pumps are those in which a finished cylinder is tightly fitted by a reciprocating-piston (Figs. 181(*a*) and 183).

Plunger-pumps are those in which the reciprocating part is a solid plunger which does not come in contact with the cylinder-walls. These plungers alternately enter and withdraw from the cylinder through packing-glands(Figs. 181(*b*), 184, 186, and 187). The methods of packing plunger-pumps divide them into the additional classes of inside (Figs. 184 and 186), outside (Fig. 187), and outside center-packed (Fig. 181(*b*)) plunger-pumps. The differential plunger-pump (Fig. 181(*c*)), while it inspires only on the upward stroke, is, on account of the design of the plunger, double-acting on the discharge.

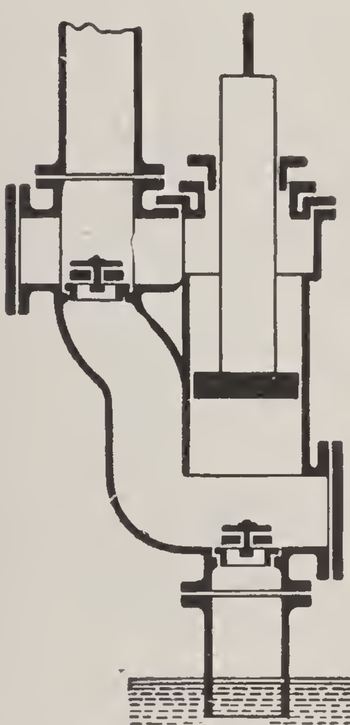
Fig. 181(*d*) shows a reciprocating piston-pump called a U pump, in which the valves are placed in the piston and the flow is in one direction, with no reversing of the current of water. The most serious defect in most reciprocating-pumps is the reversal of the current, which is here eliminated. These reversals may cause a considerable loss of energy and produce violent and injurious shocks; and on account of this defect the number of reversals of most reciprocating-pumps must



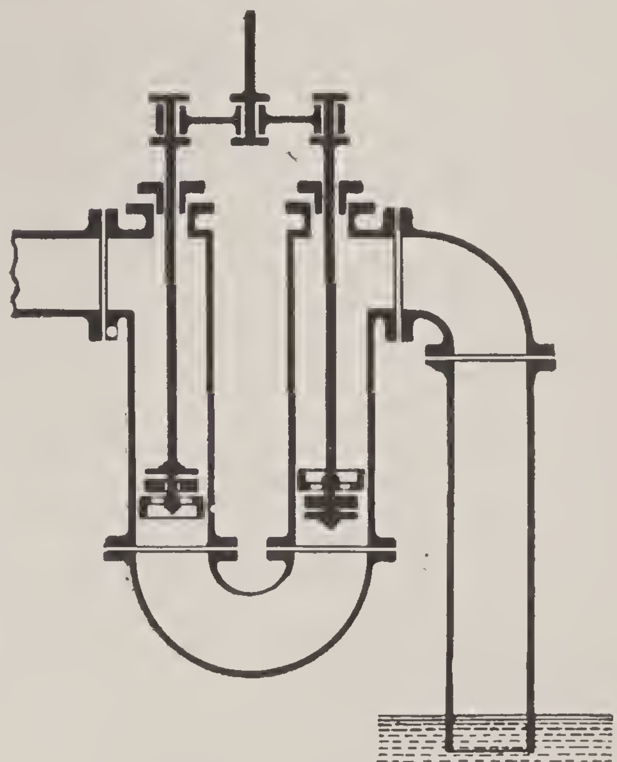
(*a*) Single-acting Piston-pump.



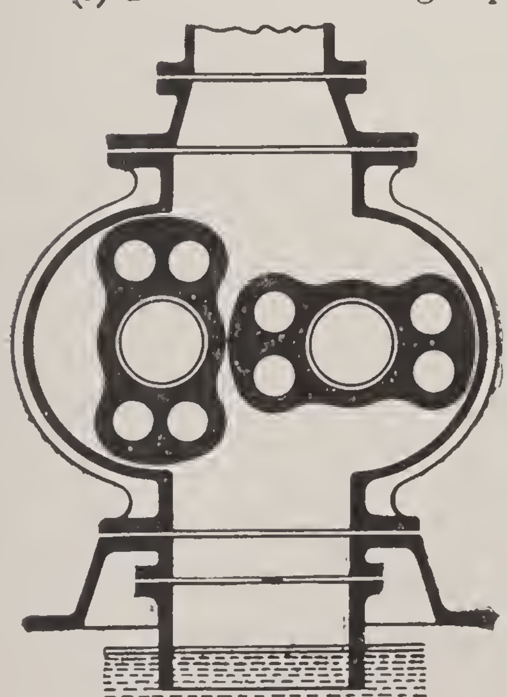
(*b*) Outside Center-packed Double-acting Plunger-pump.



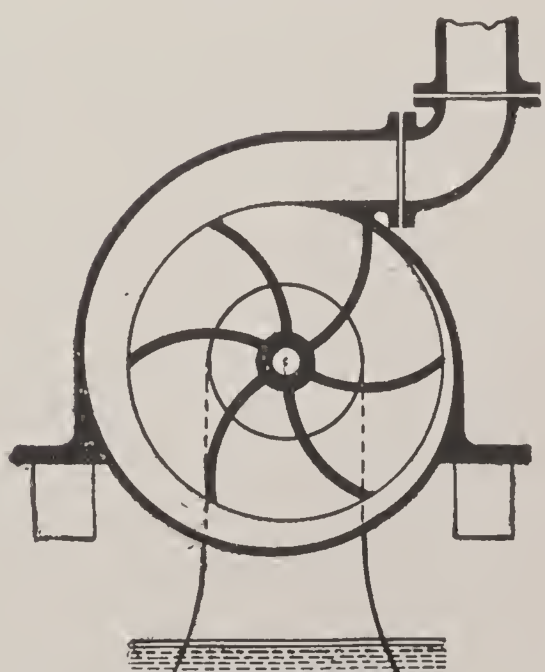
(*c*) Differential Plunger-pump.



(*d*) U Pump.

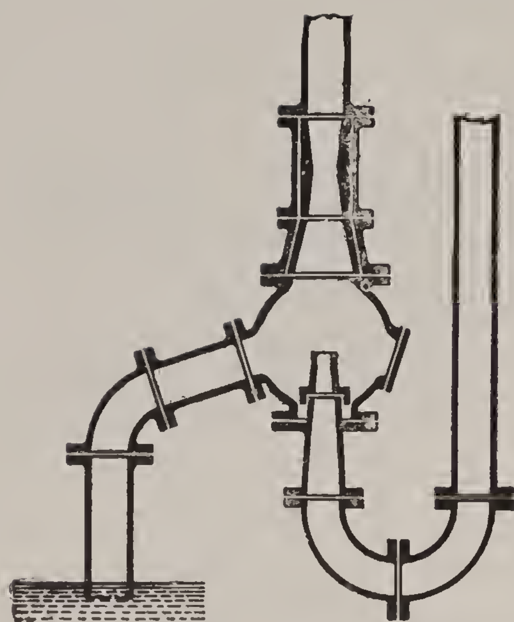


(*e*) Rotary Pump.

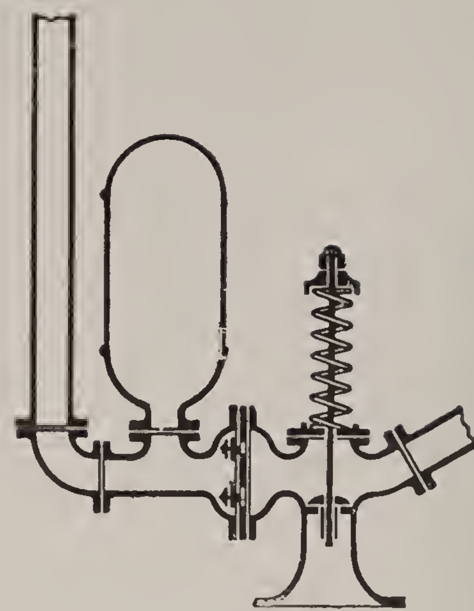


(*f*) Centrifugal Pump.

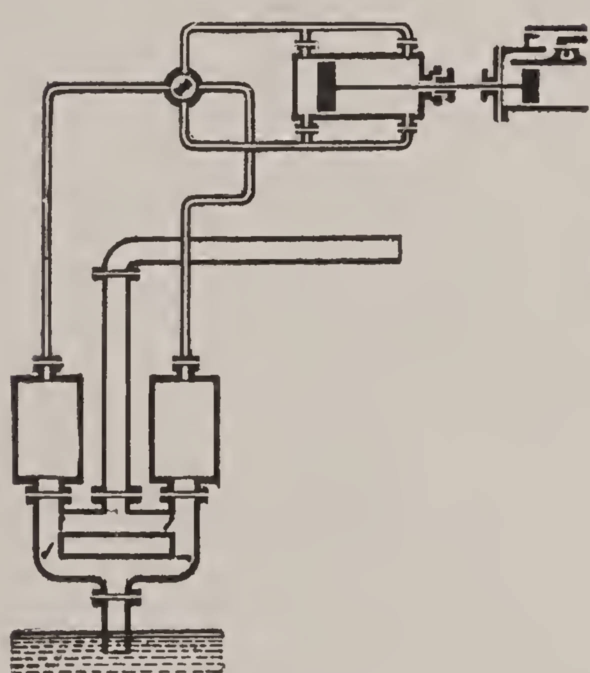
FIG. 181.—TYPES OF PUMPS.



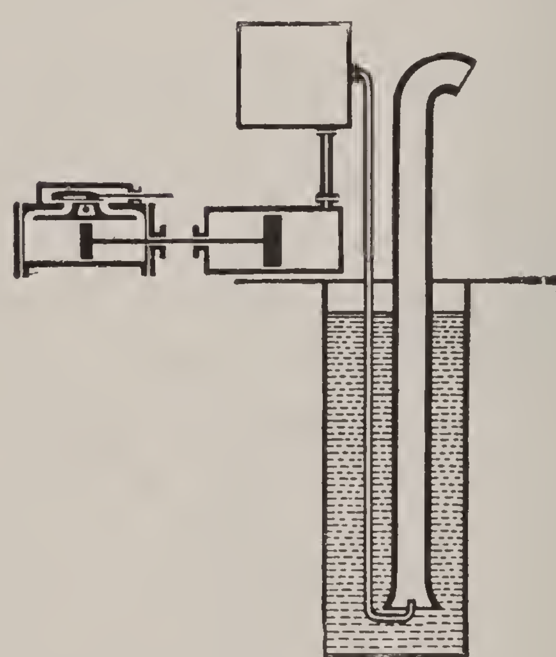
(*a*) Jet-pump.



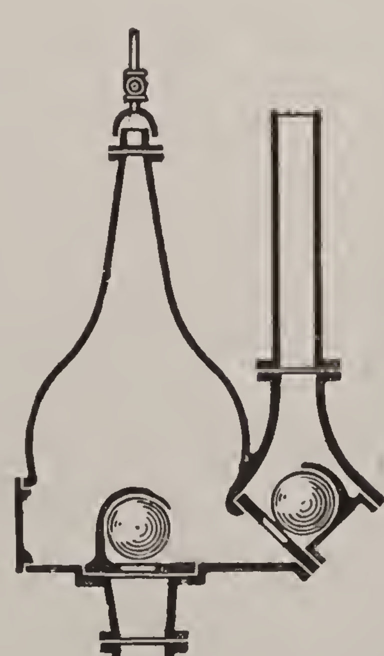
(*b*) Hydraulic Ram.



(*c*) Air Displacement-pump.



(*d*) Air Lift-pump.



(*e*) Vacuum-pump.

FIG. 182.—TYPES OF PUMPS.

be limited. It is therefore possible to run pumps of long stroke at a higher piston-speed than those of short stroke. The ordinary reciprocating-pump of 10- or 12-inch stroke and limited valve-area should seldom be operated at a greater rate of speed than 100 feet per minute, while pumps of long stroke and ample valve-area are sometimes operated at 300 to 400 feet of piston-travel per minute.

There is no such limit necessary with pumps of the type shown in Fig. 181 (*d*), and hence these pumps may be operated at much higher speeds. The speed in this case is limited by the necessity of limiting the velocity of the water in its passage through the valves.

In Figs. 181 (*a*), (*b*), (*c*), and (*d*) the water end only of the pump is shown. The piston- or plunger-rods may be operated by a connecting-rod and crank to which power is furnished by any form of motor, or a second cylinder may be attached to the other end of this rod and the water end operated by hydraulic or steam power directly. The arrangement of the pump for the application of power gives rise to the additional classification of reciprocating-pumps into power (Figs. 179 and 180), steam (Figs. 183–187), and hydraulic pumps. Power-pumps include all that class of pumps which require an independent motor for their operation. The term "power-pump," however, includes all other forms of pumps besides the reciprocating variety, which are operated by independent motors. Reciprocating power-pumps may be Single-cylinder, Duplex, or Triplex, in accordance with the number of cylinders of which the pump is composed.

675. *The Steam-pump.*—The steam-engine can be applied, in common with other motors, to the operation of power-pumps; and such an arrangement when properly made is highly efficient and worthy of careful investigation and consideration. Such an arrangement involves the use of two separate machines. In a large and important class of pumping-machinery the steam- and water-cylinders are placed in the same machine and in such cases are best considered together. Such pumps are called "Steam-pumps."

Numerous varieties of this class of pumps are in use. From the methods of application of steam this class of pumps is divided into "high-pressure" steam-pumps, which include all those in which the steam is used at its initial pressure for the full length of the stroke in the cylinder and not again used, and "compound" steam-pumps, in which the steam is used expansively in two or more cylinders. The arrangement and design of the pump give rise to other divisions. In the direct-acting pump the steam-piston is connected directly by means of a piston-rod with the pump-piston or plunger, the piston-rod being common to both steam-piston and water-plunger (Figs. 183 and 184). In steam-pumping machinery of this class having only one set of steam-cylinders the steam must be used at its initial pressure for the full length of the stroke, as in the simple form of this pump (Fig. 183) there are no parts with the function to receive, store, and finally give up the energy delivered by the steam at the beginning of the stroke as must be done when steam is used expansively. Consequently in this form of pump the only method of using steam is to exhaust it from the high-pressure cylinder directly into the low-pressure cylinder, and use it in both cylinders for the full length of the stroke and without the use of cut-

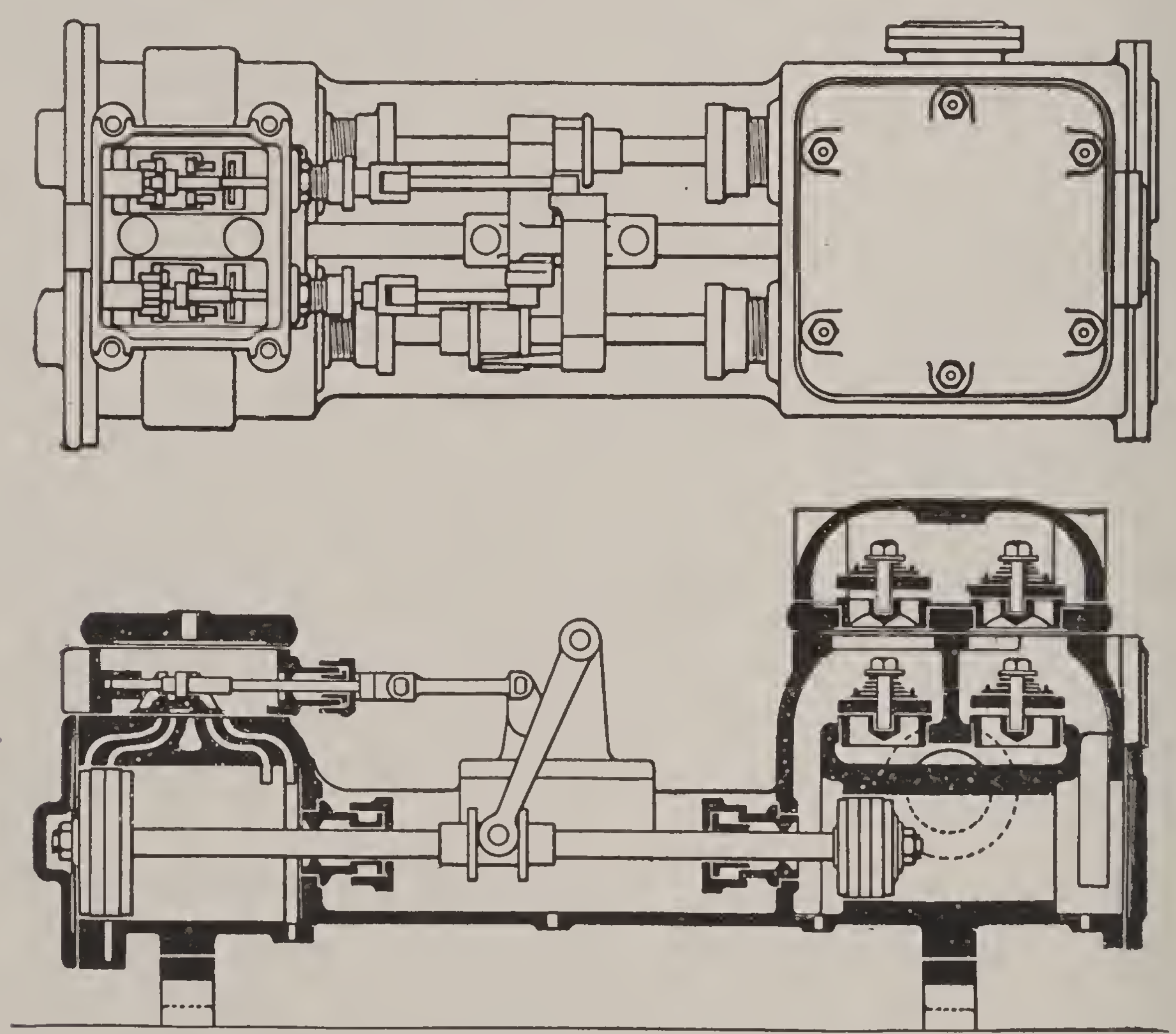


FIG. 183.—DIRECT-ACTING DUPLEX PISTON-PUMP.

offs. To overcome this disadvantage and to attain the economy possible with greater rates of expansion, and still obtain the compactness of this type of pumping-machinery, various types of compensators have been introduced. The Worthington device (Fig. 184) is perhaps the best-known type. In this machine the energy of the steam at the beginning of the stroke not only overcomes the resistance of the water pumped, but also forces the hydraulic pistons at the front end of the pump against a fixed pressure stored in an accumulator. When the center of the stroke is reached this stored energy is gradually returned to the pump after the steam is cut off and when it is expanding in the cylinder. A decided

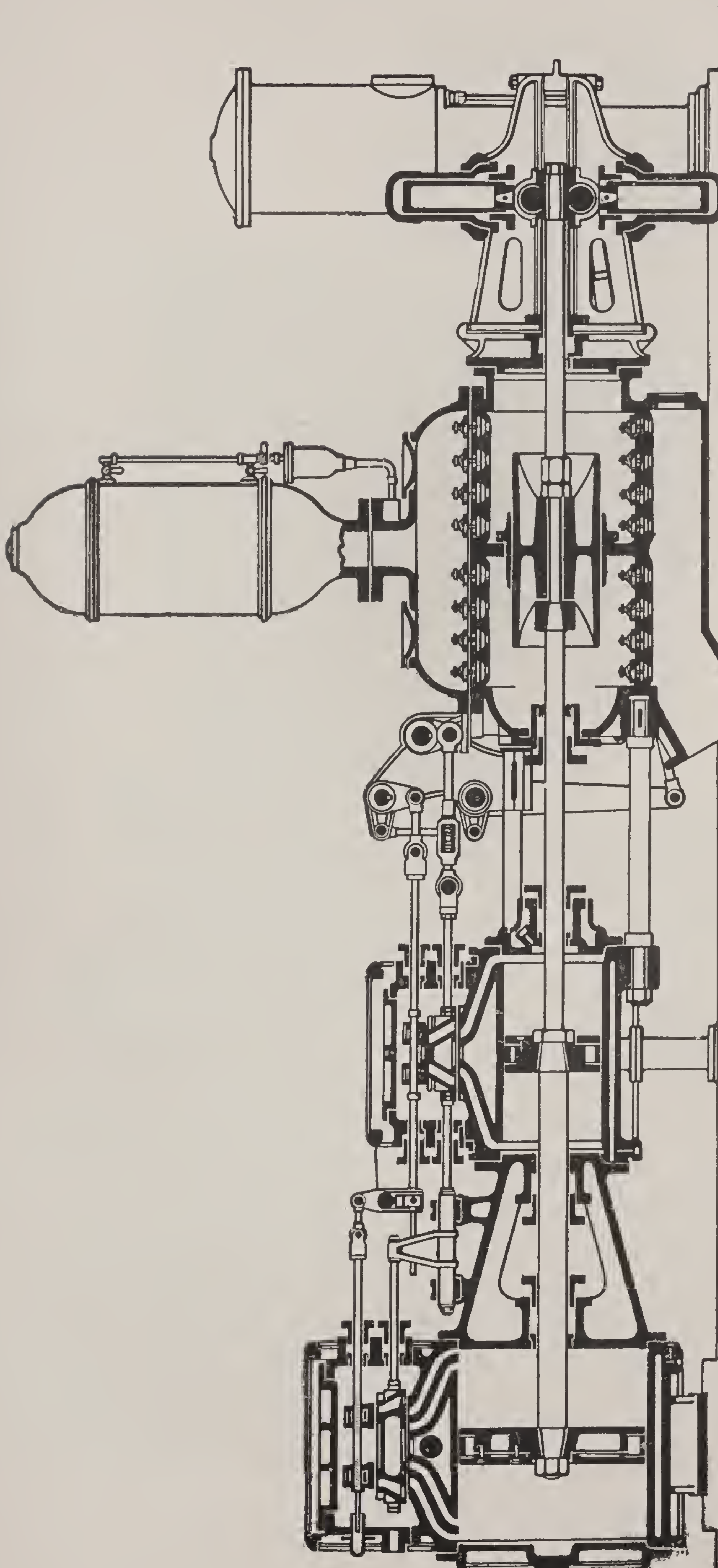


FIG. 184.—WORTHINGTON COMPOUND DIRECT-ACTING PUMP WITH COMPENSATOR.

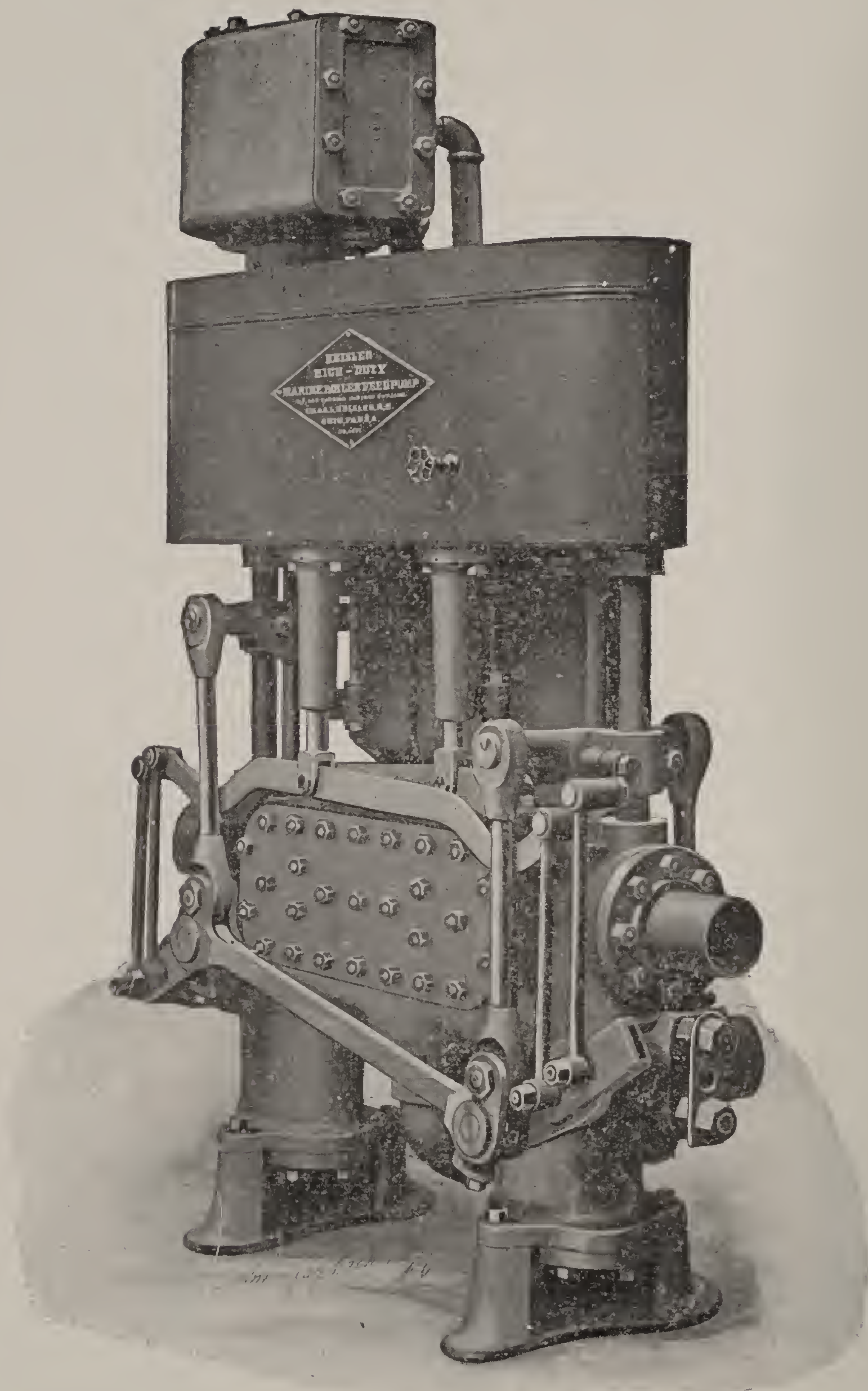


FIG. 185.—HEISLER PUMP WITH COMPENSATOR.

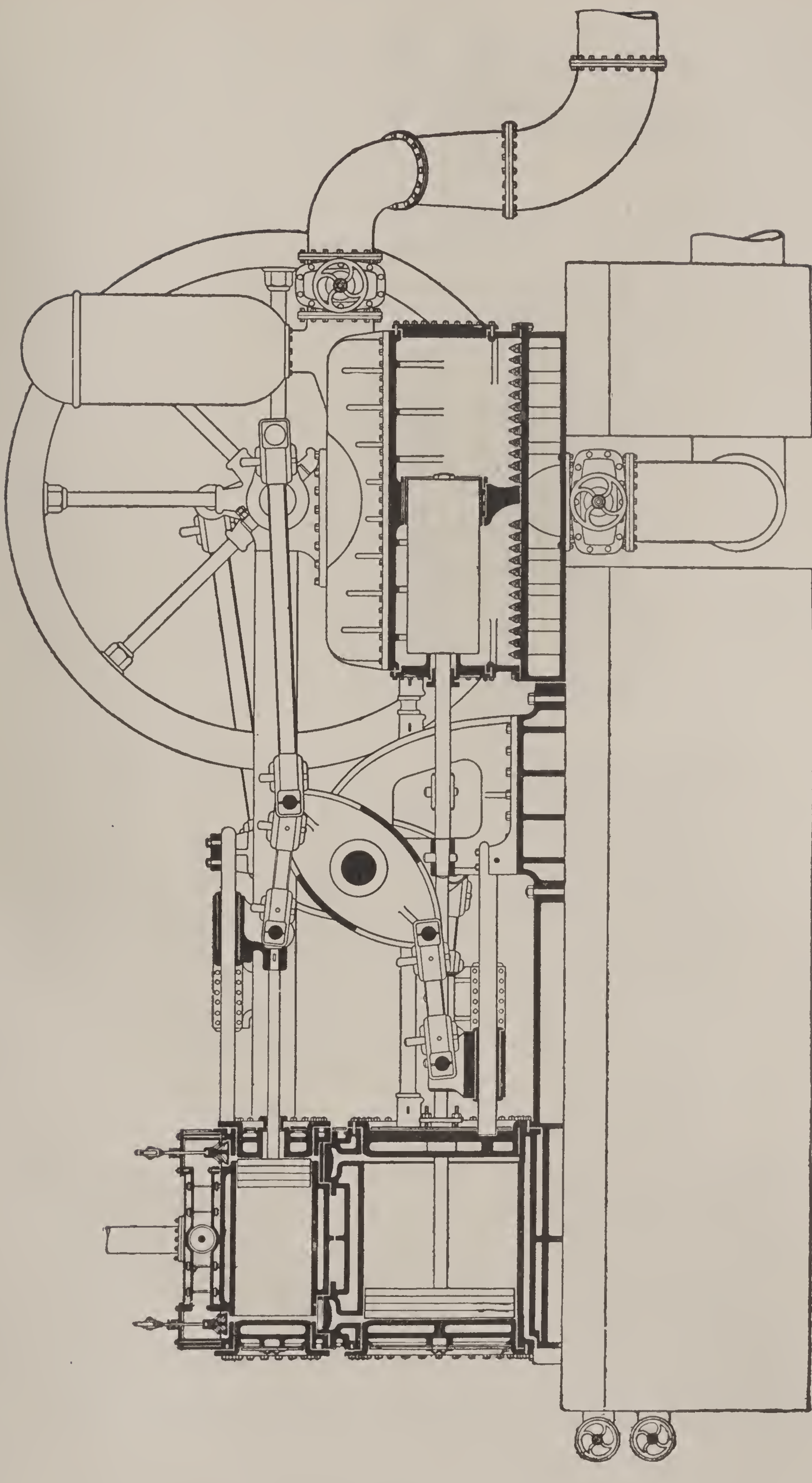


FIG. 186.—GASKILL CRANK AND FLY-WHEEL PUMPING-ENGINE.

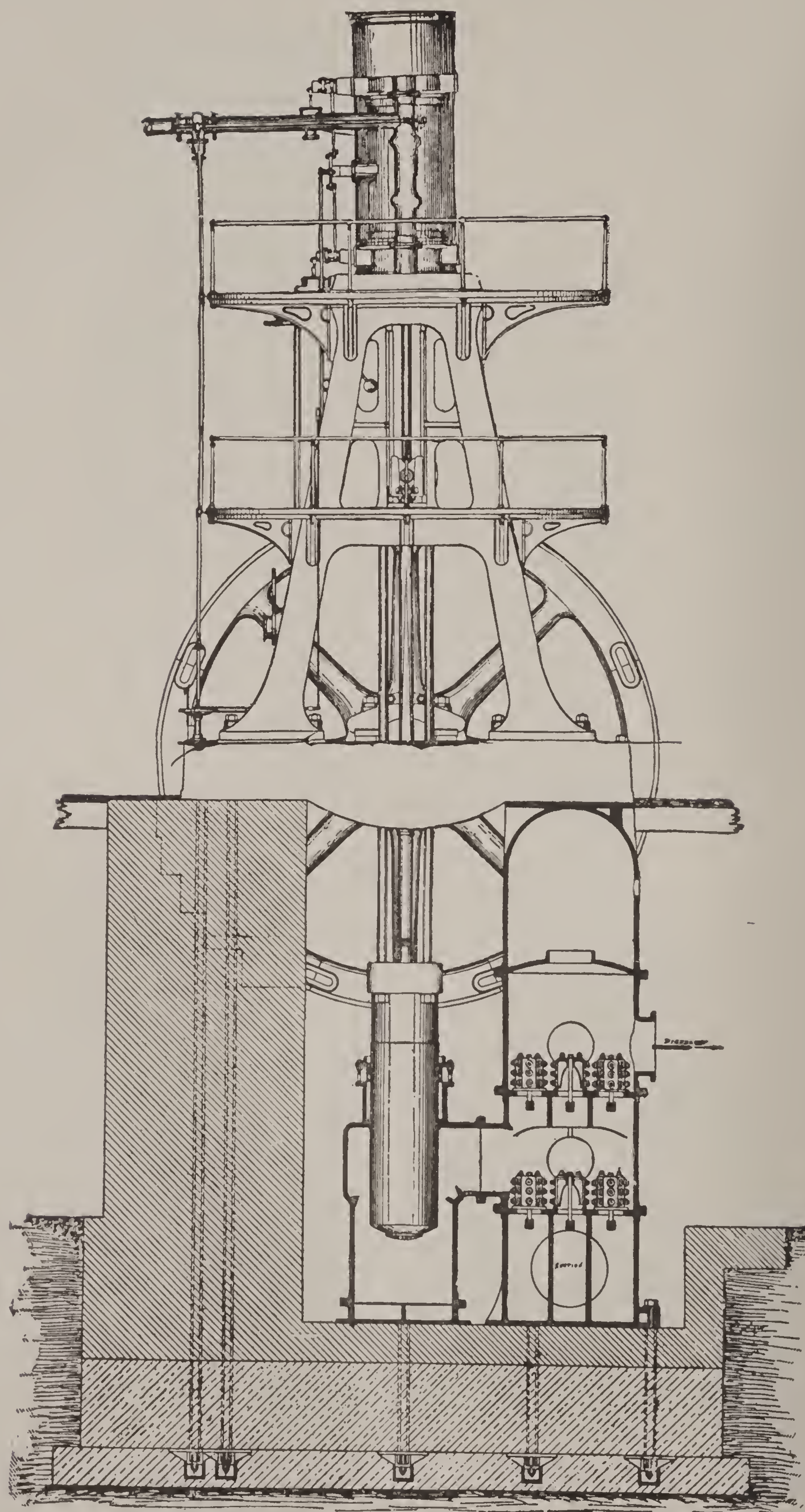


FIG. 187.—ALLIS VERTICAL TRIPLE-EXPANSION PUMPING-ENGINE.

increase in the economy of operation of this type of pumps results. Another type of compensator of considerable promise is the Heisler Compensator (Fig. 185). In this form of compensator the cylinder of one side at full steam-pressure transmits a portion of its force to aid the other side, which is working expansively.

The greatest economy in steam-pumping machinery has been developed by what is known as the crank and fly-wheel types of steam-pumps. Fig. 186 illustrates the Gaskill horizontal type of crank and fly-wheel pumps, and Fig. 187 illustrates the Allis vertical crank and fly-wheel type now manufactured by most of the leading pump-makers of the United States.

Hydraulic pumps are pumps arranged very much after the style of direct-acting high-pressure steam-pumps, but in which water under pressure is used in the power-cylinder. They are not in common use, but are occasionally applicable. The exhaust from the power end is often wasted into the discharge-pipe from the pump end.

676. Rotary Pumps.—In the rotary type of displacement-pumps two revolving pistons rotate in a pump-case, which they accurately and

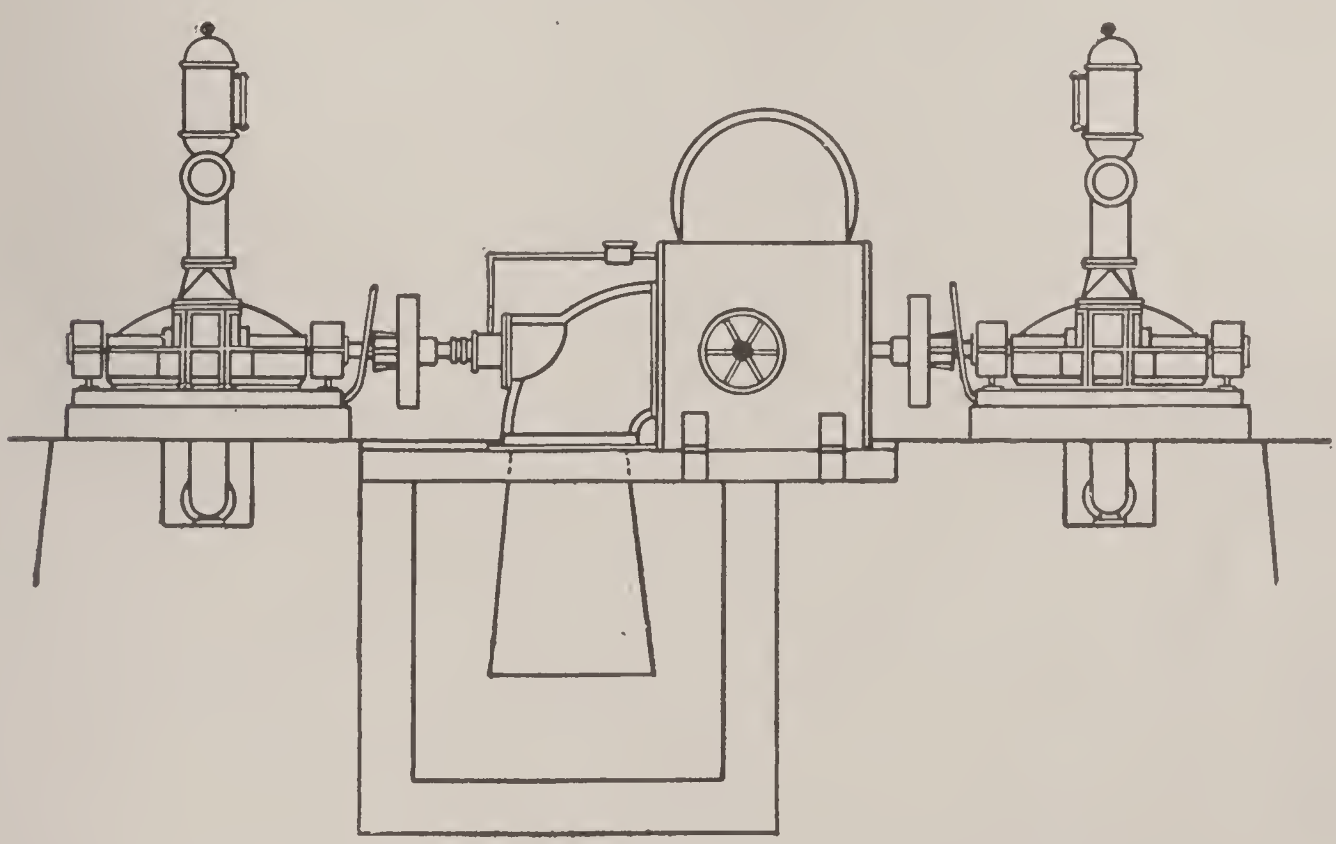


FIG. 188.—CONNERSVILLE ROTARY PUMPS WITH TURBINE.

completely fill (Fig. 181 (*e*)); the rotation of these pistons alternately inspires and discharges the water to be raised. These pumps act without the use of suction- or discharge-valves. A plant consisting of two such pumps operated by a turbine water-wheel, as installed at Connersville, Indiana, for city water-works purposes, is illustrated in Fig. 188.

677. Air and Steam Displacement-pumps.—Other forms of displacement-pumps are those which use air or steam as the displacing agency. A common type of air displacement-pump is the Merrill Pump, which consists of two cylinders, set below the water-surface; the water is admitted to each cylinder alternately by gravity and is forced from the cylinder by the direct pressure of compressed air, which is then exhausted into the atmosphere. In the Harris displacement-pump (Fig. 182 (*c*)) the air used to displace the water is returned directly to the inlet of the compressor, and is forced by the compressor into the second chamber, thus utilizing the work already done and effecting a considerable economy in the use of air. The steam vacuum-pump (Fig. 182 (*e*)) operates on somewhat similar lines; but in this case the condensing of the steam is an important factor. Steam is admitted to a chamber and condensed therein by a spray of water, thus creating a vacuum which inspires the water; the steam is again applied at full boiler-pressure and the water forced by the incoming steam through a discharge-valve and pipe, after which the steam is again condensed and the operation repeated. Such pumps commonly consist of two chambers, so that a comparatively continuous discharge of water results.

678. Continuous-flow Pumps.—An additional variety of displacement-pumps, which differ from those described, are the continuous-flow pumps. The most common type of these pumps is the ordinary chain-pump, where enlarged piston-links on the chain partially or completely fill a pipe or passage through which they pass in one direction. As the chain passes below the water and into the pipe, the spaces between the piston-links are filled successively and the water discharged at the outlet as the pistons pass it. The screw-pump also has a similar action, the displacement being produced by the screw-blade. The "U" type of pump (Fig. 181 (*d*)) is a less-known variety of the same class. A third type of continuous-flow pump is the Johnson Deep-well Pump. In this pump, by the use of a Whitworth quick-return motion, the double pistons make a quick down-stroke, during which time they are free from load, and a slow up-stroke under load. One piston of this pump is always on the up-stroke.

(2) *Impeller-pumps.*

679. Action of Impeller-pumps.—The second great class of pumps is that of the Impeller-pumps, in which a volume of water is moved by the continuous application of power through some mechanical agency or medium. These pumps consist of the centrifugal pumps and the various jet-pumps. In the displacement-pump, previously described, the motive energy is delivered to the water by a direct pressure which displaces the water and which must be equal in amount to the head against which the water is pumped and to various friction losses in the pumping-machinery and discharge-pipes. In the class of Impeller-pumps the energy is applied to the water by means of pressure due to the velocity of either a mechanical agency, as in the centrifugal pumps, or of a fluid agency, as in the jet-pumps.

680. Centrifugal Pumps.—For falling bodies we have the well-known formula $h = \frac{v^2}{2g}$; that is to say, the velocity v is generated by a fall from the height h; consequently when we reverse the action and generate pressure by the application of velocity we should, theoretically, have the same condition, and a velocity $v = \sqrt{2gh}$ should be capable of generating a head h. In the centrifugal type of pump (Fig. 181 (f)) the velocity of the periphery of the impeller must ordinarily exceed the velocity given by the above formula. A pump which acts strictly as a centrifugal pump must, however, have straight radial vanes or impellers. As soon as the vanes are curved, as is done in practically all centrifugal pumps now made (Fig. 189), an additional

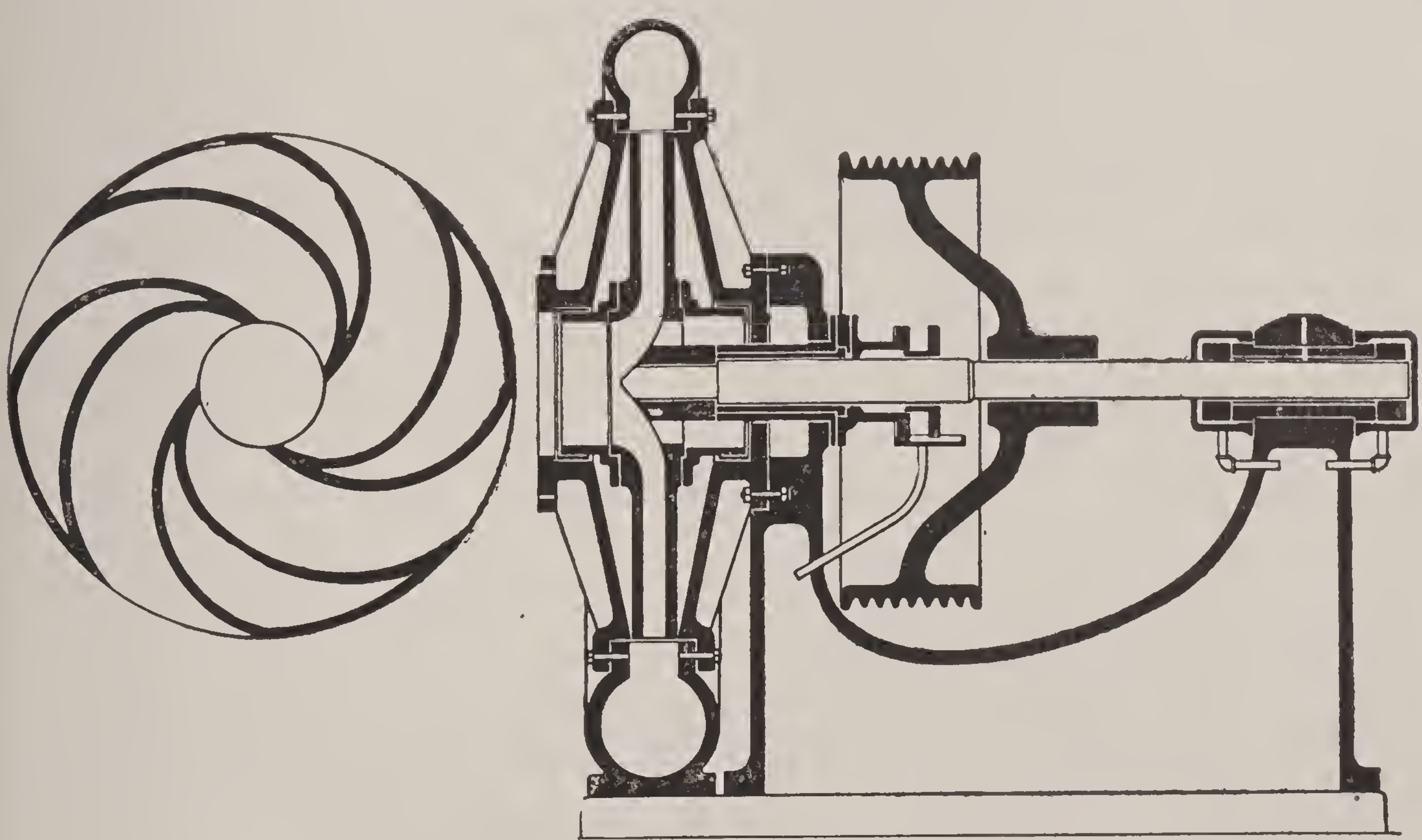


Fig. 189.—Section of Rockford Centrifugal Pump.

force results, and the pump ceases to depend solely on centrifugal force. The greater the curve of the vanes the more important becomes the action of the new force, which is the resultant of the pressure exerted by the inclined surface of the vane, and which acts more with a displacement than a centrifugal effect, as in the case of the screw-pump.

Centrifugal pumps may be classed in various ways according to their design and arrangement, as shown in the table of classification. Their selection depends on the various uses to which they are to be put, the method selected for the application of power, and various other factors incidental to the installation.

Fig. 189 illustrates a vertical side-suction centrifugal pump with impeller of the inclosed type, as used in the Rockford, Illinois, deep-well pumping-plant (see Fig. 62, page 316).

681. **Jet-pumps.**—Jet-pumps (Fig. 182 (*a*)) are arranged to utilize the velocity energy of water-, steam-, or air-jets. In all of this class of pumps a moving jet of the liquid used is delivered through a restricted throat, drawing with it the water to be raised, to which the velocity energy is delivered. Water, steam, and air have each particular attributes of their own in their application to this class of pumps. The air especially has an additional effect not due to the velocity, in that it reduces the specific gravity of the rising column of water, and may, under proper conditions, cause an overflow in the column so lightened (Fig. 182 (*d*)).

This class of air-pumps, called "air-lift" pumps, has recently been quite widely applied to raising large quantities of water from bore-holes. These pumps are not highly efficient, but are capable of raising a larger amount of water from a small hole than any other method. For reasonable efficiency the submergency of the discharge-pipe should be at least 60 per cent of its total height, or one and one-half times the height to which the water is raised above the surface.

The formula commonly used for determining the relation of the various factors in an air-lift problem is

$$q = \frac{125A}{h};$$

in which q = gallons of water per minute;
A = cubic feet of free air per minute;
h = height of lift in feet from water-surface to point of discharge.

(3) *Impulse-pumps.*

682. The third class of pumps comprises those of the impulse type, which raise water by the periodical application of force suddenly applied and suddenly discontinued. The hydraulic ram is the principal representative of this class (Fig. 182 (*b*)). In this pump the pulse-valve or

waste-valve is opened automatically either by gravity or some other agency (such as a spring, as in the figure) properly applied. The water in the drive-pipe wastes through this valve, acquiring a velocity which in turn generates sufficient friction to suddenly close the valve, thus causing an impact or impulse which, when properly applied, opens the check-valve and delivers a certain proportion of water into the air-chamber and delivery-pipe. As the impulse dies away the waste valve again opens and the cycle is repeated.

(4) *Bucket-pumps.*

683. The fourth class of pumps is composed of the bucket-pumps, which include all those in which definite receptacles are alternately filled, raised, and emptied. These pumps are often of the form of the continuous-conveyor type, in which a series of buckets attached to a belt and chain are dipped in the water, filled, elevated, and emptied.

PUMP DETAILS.

684. **General Rules.**—The class of any pump must modify largely the details used in its construction. A few general rules will apply, however, in all cases.

All pumps should be so designed and connected as to admit the free and unrestricted flow of water. They should be free from all air-traps, and when changes in the direction of flow are necessary, large easy bends should be used.

685. **Valves.**—Almost all displacement-pumps require the use of admission- and discharge-valves. These valves are a serious source of loss of efficiency in this class of machinery. Frequently as high as 12 to 15 feet of head is lost in the valve- and water-passages even of high-grade pumping-engines.

The primary requisites of pump-valves are as follows:

They should close tightly, to avoid loss from leakage.

They should close promptly, to avoid loss from slip.

They should have small lift, to permit of prompt closing.

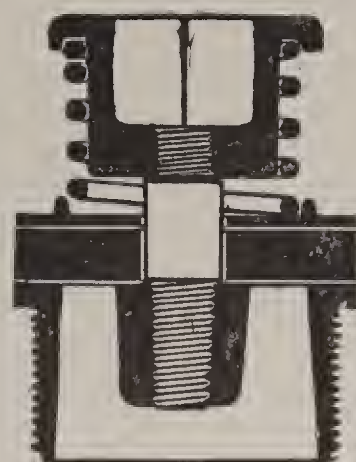
They should have large waterways.

They should open easily and without large extra pressure.

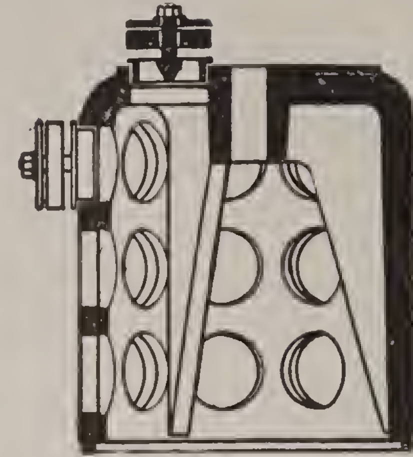
They should present small resistance to flow, in order to reduce the friction losses to a minimum.

They should be simple, readily accessible, and readily removable to facilitate repairs.

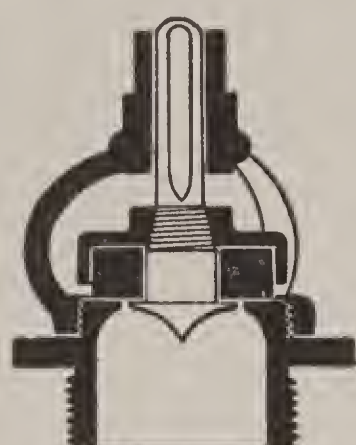
The type of valve most widely used is shown in Fig. 190 (*a*). These



(*a*) Ordinary Valve.

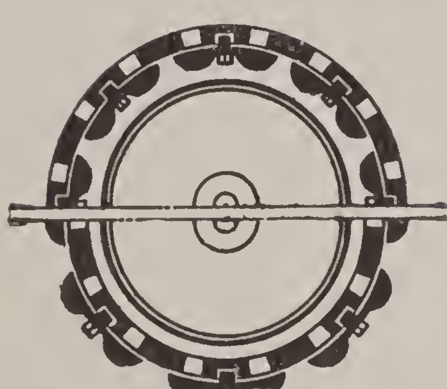


(*b*) Method of Grouping Valves.

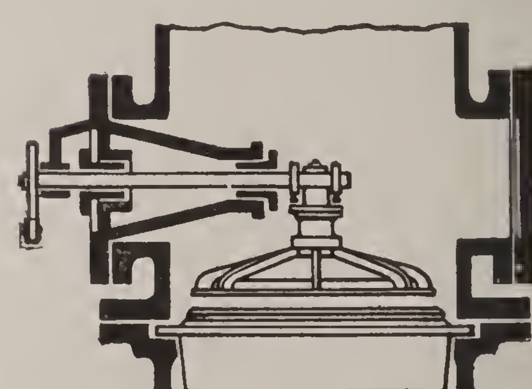


(*c*) Troy Valve.

(*d*) Battle Creek Valve.



(*e*) Walker Valve.



(*f*) Riedler Valve.

(*g*) Ball Valve.

(*h*) Cone Valve.

FIG. 190.—TYPES OF PUMP-VALVES.

valves are ordinarily from $3\frac{1}{2}$ to $4\frac{1}{2}$ inches in diameter. The valve-seat, stem, cover-plate, and spring should be of bronze.

True cylindrical springs are preferable to conical springs. The disks should be of medium hard India rubber, vulcanized sufficiently to give it firmness. For hot-water pumps this disk should be of hard rubber. The lift of the valve is limited by the stem-head, and the stem prevents its drifting sidewise. A sufficient number of these valves are grouped in the valve-chamber to give the desired waterway. In poor pumps this is commonly not over 25 per cent of the plunger area; while high-grade pumps will have a free waterway of from 50 to 100 per cent of the plunger area, according to the speed of operation and number of reversals of the plunger.

Fig. 190 (*b*) shows the arrangement of these valves as commonly used in large pumping-engines. Several of these groups may be used in a single valve-chamber.

Fig. 190 (*c*) shows the Troy valve used in the Gaskill pumping-engine. It is a small valve having a diameter of about $1\frac{3}{4}$ inches. No spring is used with this valve.

Fig. 190 (*d*) shows a metal valve used in the Battle Creek pumps. In this valve the lift is limited by the walls of the valve-chamber. The curved center guides the liquid through the opening with minimum friction and small eddy losses, as the liquid leaves the curve on a tangent when the valve is fully open.

Fig. 190 (*e*) shows the form of valves used in the Walker pump. The valve disk or cover is of rubber of a rectangular shape thickened at the sides, the center forming a hinge and the sides forming the valve-covers. The upper portion of the figure shows the arrangement of the suction-valves; the lower portion shows the discharge-valves.

Fig. 190 (*f*) shows the Riedler valve. Only one valve of this type is used on each inlet or outlet of the Riedler pumps. This valve is mechanically closed just as the direction of motion of the pump-piston is reversed. By the use of this valve, large waterway is provided, and by its mechanical closing, slip and pound is prevented when the pump reverses.

Fig. 190 (*g*) shows a ball valve which is commonly used in deep-well pump-cylinders. The valve is usually a bronze sphere and seats in a bronze ring simply by its own weight. Its lift is limited as shown in the cut. Groups of such valves are sometimes arranged in valve-chambers for reciprocating-pumps.

Fig. 190 (*h*) shows the Downie cone valve, which is also used for deep-well cylinders. This valve consists of two cones, the outside one being movable. When this outside cone is seated the valve is closed, the solid metal of each cone closing the apertures in the other. When the valve is raised the apertures in the cones are opposite and the water passes readily through the openings.

Rotary and centrifugal pumps can be operated without valves of any description, but it is desirable with these pumps, as with all others which are used for water-works purposes, to use a check-valve on the discharge, so that in case any accident should happen to the machinery, the reservoir, stand-pipe, or other device for storing water will not be emptied through the broken pump into the pumping-station. In the case of rotary and centrifugal pumps the check-valve is particularly necessary, as when the power ceases to be applied the pump will discharge the stored water back through the pump and suction-pipe into the source of supply.

686. Air and Vacuum Chambers.—All displacement-pumps except the continuous-flow variety, and even those unless the continuity of flow is perfect, should be provided with vacuum- and air-chambers on the inlet- and discharge-pipes in order to take up the irregularities of flow due to intermittent or irregular action and prevent injurious and sometimes destructive shocks.

The size of air-chamber depends on the condition of working. Since the function of the air-chamber is to eliminate irregularities, the greater the irregularities the larger the chamber should be. Hence, with a high-speed pump or a pump forcing water through great length of pipe or against a high head, the air-chamber should be enlarged over what would be needed with slow-running pumps and low lifts.

Triplex pumps under low lifts may be provided with air-chambers of a capacity equal to a single displacement of the piston, while for single-cylinder, double-acting pumps the air-chamber should be from six to eight times this size. Means for supplying air-chambers with air should also be provided. In suction-pumps this can be readily accomplished by connecting a small check-valve with a pump-chamber so that when the pump inspires it will draw in a small amount of air with the water. A globe valve outside of the check-valve will control the operation of the air-inlet as may be desired.

It is desirable to provide air-chambers with a gauge-glass, so that the amount of air in the chamber will be known (Fig. 179).

687. Inlet- or Suction-pipes.—Pumping-machinery may receive the water to be pumped in two ways.

First, the water may flow to the machine by gravity, the machine being below the water a depth at least equal to the sum of the velocity and friction heads.

Second, the water may be raised through a pipe into the machine by atmospheric pressure or suction.

Suction consists in creating a more or less perfect vacuum in the suction-chambers and suction-pipe and filling the same with water by atmospheric pressure. Before a suction-pump will work properly it must be able to create such a vacuum or to "prime" itself. The priming of a pump in which there is no water requires either the filling of the pump with water from some other source and the consequent expulsion of the air, or that the empty pump shall act as an air-pump and thus remove the air. A centrifugal pump cannot act as an air-pump. It must therefore be below the water to prime itself. In order to prime a pump which cannot act as an air-pump it must be provided with a foot-valve which will prevent the loss of water from some higher source, or a priming-pump (i.e., a small air-pump) must be attached to it.

Reciprocating-pumps can be primed by the piston action much more readily than rotary pumps, but, especially on high lifts, should be provided with priming-pipes.

For perfect suction and satisfactory operation care must be taken to secure the following conditions:

1. The openings between the moving and fixed parts of the pump must be as small as possible, that is, the pump must be well packed between the fixed and moving parts.

2. The suction-chambers and pipes must be air-tight.

3. All air-traps must be avoided in all suction members.

4. All unnecessary bends must be avoided, and the suction-pipe should be made as short and direct as possible. The pump should be placed as near the water as possible, and the suction-pipe should be of proper size. The possibility of suction is limited (see Table No. 37, page 224).

The amount of available suction-head must never be less than the sum of the following heads commonly lost in suction-pipes and the suction-passages of pumps:

1. Influx loss at end of suction-pipe (see eq. (36), page 248).

2. Velocity loss in suction-pipe (see Art. 657).

3. Friction loss in suction-pipe (see Fig. 34, page 243).

4. Friction loss in suction-valve and water-passage of pump (see Art. 685).

5. Acceleration head, or the pressure necessary to accelerate the water where the flow is not uniform.

6. Vapor tension of water (see Table No. 38, page 225).

If the sum of these losses together with the head against which the water is to be raised by suction is greater than the available atmospheric pressure, the pump will not work to its proper capacity and may not raise the water at all.

688. Location of Pumping-machinery with Respect to the Level of the Water Drawn from.—The conditions under which the water-supply must be obtained will to a large extent control the type of machinery which must be used. As before noted, it is always desirable to place the water end of the machinery as near the water as possible, while the power end must usually be placed above high water, or, if placed below the water, it must usually be arranged in a water-tight shaft or compartment. The ordinary horizontal type of reciprocating pumping-machinery should seldom be placed more than 18 feet above the lowest water it will be called upon to handle. A suction lift of 24 to 26 feet is, however, sometimes possible. Where the distance from pump to water-surface is greater than the maximum, and especially where large volumes of water are to be handled, such lifts become hazardous or impossible and other types of pumping-machinery must be used or other methods of locating the machinery employed.

The most obvious method of reaching water when it is below suction distance is to sink the pump to such a depth below the surface as to bring it within easy suction distance of the water. When the distance is not too great and where conditions are favorable, this can readily be done, and this method has frequently been employed.

The water ends of pumps demand but comparatively little attention. The flow of water lubricates most of the parts to a considerable extent, and such parts as are not in this way sufficiently lubricated can be easily cared for by attention at occasional intervals.

The motor end of the pump, however, is usually much more complicated and necessarily requires more particular and constant attention. When, therefore, the depth at which the water is obtained is considerable, it is often desirable to rearrange the design of the pumping-machinery, placing the water end within easy suction reach of the supply, and the motor end within ready access from the surface and near the boilers or other source of power. This has given rise to various types of vertical machinery which fulfill these conditions with more or less success.

The cost involved in the construction of large shafts, especially in unfavorable locations, may make it desirable to economize in shaft room. Special types of pumps have been designed to suit these requirements, and the same result may be obtained under favorable conditions by the enlargement of the base of the shaft and the use of ordinary types of horizontal pumps located near the shaft base where such arrangement is permissible. The arrangement of the machinery with this end in view is shown in the plant installed at Rockford, Illinois (page 318). The water end of this pumping-plant consists of high-grade centrifugal pumps, while vertical compound condensing engines furnish the motive power, and rope transmission is the means of connecting pumps and engines. The above arrangement assumes the ability to concentrate the water at one central shaft in which the machinery is located. This may be done in general in the following ways:

1. By the construction of a large open well in an open or coarse water-bearing stratum. Such plants have been satisfactorily adopted for small and medium quantities of water.

2. The various wells may be connected by pipes laid as deeply as possible in trenches open from the surface.

3. The various wells may be connected by tunnels into which the water may empty direct, or the wells may be connected by pipes laid in the tunnels and connected to the suction side of the pumps.

Where shaft and tunnel work is expensive it will sometimes become desirable to install small isolated plants on each well, which may be operated singly or together as the water required demands. For this use one or more small shafts may be built and connected directly with the water-bearing stratum, or one or more bore-hole wells may be sunk, and in such shafts or wells the secondary machinery may be placed.

Separate steam-pumps may be applied to such wells, but usually such pumps are far from economical. Power may, however, be generated in various ways from a central and more economical generator and transmitted to the separate wells by electrical transmission, by pneumatic transmission, or by hydraulic transmission.

At De Kalb, Illinois, where the water is raised from deep wells from a depth of 150 feet below the surface, electrical transmission is used, the water being raised by separate pumps and forced into the reservoir near the surface, from which it is taken by the service pumps (see Fig. 180), and pumped into the mains and stand-pipe.

At Peoria, Illinois, Mr. D. H. Maury, Mem. Am. Soc. C. E., has developed a unique and efficient method of hydraulic transmission. The water which operates the secondary plants is taken from the mains supplied by the high-duty pumping-engine. The secondary installations are operated by impulse water-wheels attached to horizontal centrifugal pumps. The water used to operate the impulse-wheels together with the water pumped by the centrifugal pumps is returned to the main-supply well.

The centrifugal pump is widely used on the Pacific coast for such purposes, and for lifts of 150 feet or less can often be operated to an advantage. Its use throughout the eastern part of the United States is not so general; the plant at Rockford, Illinois, (see Figs. 62 and 189,) being perhaps the highest lift of any attempted in the East, namely, 106 feet, when pumping to the full capacity of the plant.

DUTY AND EFFICIENCY OF PUMPING-MACHINERY.

689. Measures of Duty.—While the efficiency of pumping-machinery may be measured by the units included in the tables previously given, there is also another measure of efficiency which is largely used in considering pumping-plants and pumping-machinery. This measure of efficiency is termed "duty" and represents the ratio of work done to the energy expended in doing it. Duty may be expressed in almost any units, but in pumping it usually represents the ratio of foot-pounds of work done to a fixed weight of coal, or steam, or to a fixed number of heat-units used.

The terms most generally used to express duty of pumping-engines are foot-pounds duty per 100 pounds of coal, per 1000 pounds of steam, or per 1,000,000 heat-units.

It must be understood that the duties expressed in the above units are not necessarily equivalent, but vary largely in actual value. For example, the Indianapolis Water Company's engine, built by the Snow Steam-pump Company, is said to have developed a duty of 167.8 million foot-pounds for each 1000 pounds of dry steam, but only 150.1 million foot-pounds for each 1,000,000 heat-units.

Duty based on coal is very indefinite, for coal varies largely in its potential energy or calorific value (see Table No. 85).

When coal is considered the plant efficiency must also be included. This may include boilers, steam-pipe, feed-pump, heater, etc., which have not necessarily any relation to the individual efficiency of the pump itself. Duty based on coal should therefore only be used where the entire plant is considered and when the class of coal is also specified.

Duty based on steam is more specific, but hardly sufficiently so.

TABLE NO. 91.

DUTY, CORRESPONDING AMOUNT OF COAL PER H.P. PER HOUR, AND CORRESPONDING AMOUNT OF COAL REQUIRED TO RAISE 1,000,000 GALLONS 100 FT. HIGH.

Duty in Million Ft.-lbs.	Pounds of Coal per H.-P per Hour.	Lbs. per Million Gals. 100 Feet High.	Duty in Million Ft.-lbs.	Pounds of Coal per H.P. per Hour.	Lbs. per Million Gals. 100 Feet High.	Duty in Million Ft.-lbs.	Pounds of Coal per H.P. per Hour.	Lbs. per Million Gals. 100 Feet High.	Duty in Million Ft.-lbs.	Pounds of Coal per H.P. per Hour.	Lbs. per Million Gals. 100 Feet High.
1	198.00	83398	43	4.60	1939	84	2.36	992	125	1.58	667
2	99.00	41699	44	4.50	1895	85	2.33	981	126	1.57	662
3	66.00	27799	45	4.40	1853	86	2.30	969	127	1.56	656
4	49.50	20849	46	4.30	1813	87	2.28	958	128	1.55	651
5	39.60	16679	47	4.21	1774	88	2.25	947	129	1.53	646
6	33 00	13899	48	4.12	1737	89	2.22	937	130	1.52	641
7	28.29	11914	49	4.04	1702	90	2.20	926	131	1.51	636
8	24.75	10424	50	3.96	1668	91	2.18	916	132	1.50	632
9	22.00	9266	51	3.88	1635	92	2.15	906	133	1.49	627
10	19.80	8340	52	3.80	1604	93	2.13	890	134	1.48	622
11	18.00	7561	53	3.73	1573	94	2.11	887	135	1.47	618
12	16.50	6930	54	3.66	1544	95	2 08	878	136	1.46	613
13	15.23	6413	55	3.60	1516	96	2.06	868	137	1.45	609
14	14.14	5937	56	3.53	1489	97	2.04	859	138	1.43	604
15	13.20	5560	57	3.47	1463	98	2.02	851	139	1.42	600
16	12.37	5212	58	3.41	1437	99	2.00	842	140	1.41	595
17	11.64	4906	59	3.35	1414	100	1.98	834	141	1.40	591
18	11.00	4633	60	3.30	1389	101	1.96	825	142	1.39	587
19	10.42	4384	61	3.24	1367	102	1.94	817	143	1.38	583
20	9.90	4170	62	3.19	1345	103	1.92	809	144	1.37	579
21	9.43	3971	63	3.14	1323	104	1.90	802	145	1.37	575
22	9.00	3791	64	3.09	1303	105	1.89	704	146	1.36	571
23	8.60	3626	65	3.04	1283	106	1.87	786	147	1.35	567
24	8.25	3475	66	3.00	1263	107	1.85	779	148	1.34	563
25	7.92	3336	67	2.95	1244	108	1.83	772	149	1.33	560
26	7.61	3208	68	2.91	1226	109	1.82	765	150	1.32	556
27	7.33	3089	69	2.87	1208	110	1.80	758	151	1.31	542
28	7.07	2978	70	2.83	1191	111	1.78	751	152	1.30	539
29	6.83	2876	71	2.79	1174	112	1.77	744	153	1.29	534
30	6.60	2780	72	2.75	1158	113	1.75	738	154	1.28	531
31	6.38	2690	73	2.71	1142	114	1.74	731	155	1.27	528
32	6.18	2606	74	2.67	1127	115	1.72	725	156	1.27	525
33	6.00	2527	75	2.64	1112	116	1.71	719	157	1.26	522
34	5.82	2433	76	2.60	1097	117	1.69	713	158	1.25	518
35	5.65	2383	77	2.57	1083	118	1.68	707	159	1.24	516
36	5.50	2316	78	2 54	1069	119	1.66	701	160	1.24	511
37	5.35	2254	79	2.50	1055	120	1.65	695	161	1.23	508
38	5.21	2194	80	2.47	1042	121	1.64	689	162	1.22	504
39	5.07	2138	81	2.44	1029	122	1.62	683	163	1 21	502
40	4.95	2085	82	2.41	1017	123	1.61	678	164	1.21	499
41	4.83	2034	83	2.38	1004	124	1.60	672	165	1.20	496
42	4.71	1985									

Both theory and practice show that, with suitable conditions, steam at 150 pounds pressure has a greater value than steam at 90 pounds pressure. The entrained water from the boiler and the condensation in the steam-pipe also modify the results. When duty is based on the weight of steam used, the terms dry steam and a specified pressure should also be included.

In Table No. 91 the relation of duty to coal-consumption and steam-consumption per horse-power per hour is shown.

A table of the relations of duty to steam-consumption per horse-power per hour, and of steam required to raise 1,000,000 gallons 100 feet high, may be obtained by multiplying the figures for coal in the respective columns by 10. A similar table for heat may be obtained by multiplying by 10,000. The corresponding duty values in such tables are not, however, necessarily equivalent.

The duty of any pumping-engine or pumping-plant may be calculated from the formula:

$$\text{Duty} = \frac{\text{weight of water pumped} \times \text{head} \times \text{duty unit}}{\text{amount of energy used}}.$$

The "duty unit" is 100 for coal, 1000 for steam, and 1,000,000 for heat-units. The "amount of energy used" is the total weight of coal or steam, or the number of heat-units used.

It may be noted from Table No. 82 that with perfect efficiency the following duties should be developed:

	Foot-pounds.
100 pounds average anthracite coal..........	1,140,548,000
100 pounds average bituminous coal........	991,172,000
1000 pounds steam (approximate)..........	778,000,000
1,000,000 pounds British thermal units......	778,000,000

From the above it will be noted that 100 pounds of average coal has a greater theoretical value than 1000 pounds of steam. Under good average conditions, however, not more than 10,000 British thermal units per pound of coal can be transformed into the actual potential energy of steam, and in ordinary practice the amount transformed is usually much less.

The above theoretical equivalents should be compared with the results usually obtained in practice as shown in Table No. 92.

690. Ordinary Duty and Efficiency of Pumping-machinery.—Table No. 92 shows, first, the ordinary duty and efficiency of steam pumping-machinery; second, the duty and efficiency of pumping-plants

TABLE NO. 92.

DUTY OF PUMPING-PLANTS.

Class.		Duty per 1000 Pounds Steam.	Pounds Steam per A.H.P. per Hour.	Theoretical Efficiency. Per cent.
High-duty engines	Max.	160	12.3	20.6
	Min.	100	19.8	12.9
Pumping-engines	Max.	100	19.8	12.9
	Min.	75	26.4	9.8
Large well-designed steam-pumps	Max.	40	49.5	5.2
	Min.	20	99.0	2.6
Ordinary well-designed steam-pumps	Max.	20	99.0	2.6
	Min.	10	198.0	1.3
Direct-acting deep-well pumps	Max.	6	330.0	.77
	Min.	2	990.0	.26
Vacuum-pumps	Max.	8	247.5	1.04
	Min.	2	990.0	.26
Jet-pumps	Max.	4	495.0	.52
	Min.	1	1980.0	.13
Average Duty of Power-pumps with Direct-connected Engine. Pump Efficiency 75 per cent.				
Simple high-speed engine, non-condensing	Max.	47	42.0	6.1
	Min.	37	53.5	4.8
Simple Corliss non-condensing	Max.	58	34.0	7.5
	Min.	47	42.0	6.1
Simple Corliss condensing	Max.	75	26.4	9.8
	Min.	67	29.5	8.6
Compound high-speed condensing	Max.	75	26.4	9.8
	Min.	58	34.0	7.5
Compound Corliss condensing	Max.	92	21.5	10.2
	Min.	75	26.4	9.8
Triple-expansion condensing	Max.	114	17.4	14.7
	Min.	99	20.0	12.7
Average Duty of Air-lift Pump with Various Types of Air-compressors. Pump Efficiency 25 per cent.				
Compound Corliss condensing	Max.	31	63.8	4.0
	Min.	25	79.0	3.2
Simple Corliss condensing	Max.	22.5	88.0	2.9
	Min.	18	110.0	2.3
Simple Corliss non-condensing	Max.	14	141.0	1.8
	Min.	12	165.0	1.5
Well-designed high-pressure compressor	Max.	12	165.0	1.5
	Min.	8	247.0	1.03
Small straight-line	Max.	10	198.0	1.29
	Min.	6	330.0	.77

composed of power-pumps of 75 per cent efficiency direct-connected to various types of steam-engines, with steam-consumption as given in Table No. 89; and third, the duty and efficiency of pumping-plants composed of various types of air-compressors and the air-lift pump of 25 per cent efficiency.

These results may be considered as fair average results of well-designed plants. Results much higher may be obtained under ideal conditions, and much poorer results are only too common in actual practice.

The efficiency of various power-pumps, and of some other pumps before mentioned, is about as follows:

ORDINARY EFFICIENCY OF PUMPS.

	Minimum.	Maximum.
Reciprocating-pumps	60	85
Centrifugal pumps	50	80
Rotary pumps	50	80
Displacement air-pump exhausting into atmosphere	20	23
Harris displacement air-pump	60	70
Air-lift pump	15	40

With other values from between the limits named above substituted in Table No. 92 there will, of course, be corresponding changes in the ultimate duty and efficiency of the respective plants.

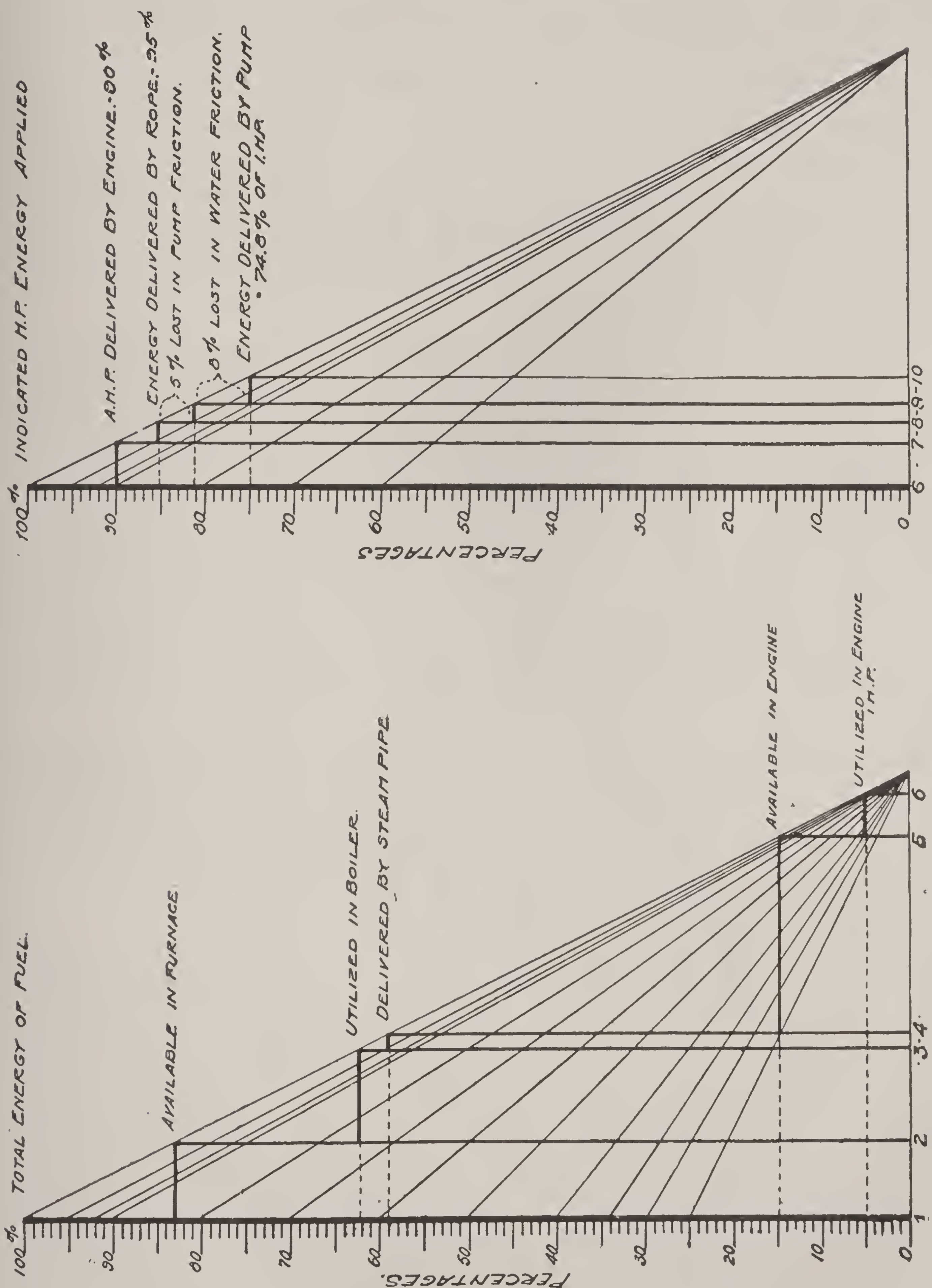
691. Methods of Analyzing Losses of Energy.—From statements already made it will be seen that a great variation in duty and in efficiency exists between the various types of pumping-plants, and consequently in the cost of their operation. Careful analysis of various combinations which can be utilized for any place should be made in order to obtain a basis for intelligent comparison. This analysis can be made either analytically or graphically, but the graphical methods possess the advantage of showing at once to the eye the points at which all losses occur, and where attempts to economy can best be made.

A graphical analysis of the power losses in the centrifugal pumping-plant at Rockford, Ill., is illustrated in Fig. 191. The diagram to the left illustrates the losses from the fuel used to the indicated horse-power developed in the engine.

The line in this diagram numbered 1 represents, by its length, the total energy of the fuel used. It is subdivided into one hundred parts. Diagonal lines drawn from any point on this line to the focal point at the right will subdivide every vertical line in the diagram proportional to the percentage line of the total energy of the fuel.

It is found that, on account of natural limitations, not more than 83 per cent of the actual fuel-value is theoretically available in the furnace. The length of the line No. 2 therefore represents the proportion of the total energy of the fuel which theoretically should be utilized by the boiler.

In the plant in question only about 75 per cent of the energy



EFFICIENCY DIAGRAM OF ROPE-DRIVEN CENTRIFUGAL PUMP.

EFFICIENCY DIAGRAM OF STEAM-PLANT.

FIG. 191.—EFFICIENCY DIAGRAMS, ROCKFORD PUMPING-PLANT.

theoretically available is utilized, hence the energy utilized in the boiler is represented by line No. 3, which is 75 per cent of the line No. 2, or about 62¼ per cent of the total energy of the fuel burned.

The loss in the steam-pipe is assumed at 5 per cent of the boiler energy, hence there is delivered to the engine about 59 per cent of the total energy of the fuel. Of the total energy delivered to the engine as steam only about 25 per cent is theoretically available in the engine. This proportion is represented by line No. 5 of the diagram. In the plant in question, the amount actually utilized in the indicated horse-power of the engine is about 8½ per cent of the energy delivered to it. This amount is represented by line No. 6 of the diagram. From this diagram it is also seen that the amount of energy utilized in the indicated horse-power of the engine is about 34 per cent of the energy theoretically available in the engine and about 5 per cent of the total energy of the fuel consumed.

The percentage line in the right-hand diagram is an enlargement of the line representing the energy utilized in the indicated horse-power of the engines as shown by line No. 6 of the left-hand diagram. From this diagram it will be noted that there is a loss of about 10 per cent in engine friction, that is, the actual horse-power delivered by the engine is 90 per cent of the indicated horse-power of the engine. About 5 per cent is lost in the transmission rope, an additional 5 per cent in the pump friction, and about 8 per cent in the friction of the water in passing through the pump, the energy actually delivered by the pump being about 74.8 per cent of the indicated horse-power of the engine.

From this diagram it will be noted that the actual efficiency of the centrifugal pump is about 88 per cent of the power delivered to the pumps by the rope-drive. This is an exceedingly high record for a centrifugal pump, and it is believed to be the highest results recorded for this type of machinery.

It will be noted from the left-hand diagram that 5 per cent of the calorific value of the fuel is utilized in the indicated horse-power of the engine, while by the right-hand diagram about 75 per cent of the indicated horse-power is utilized in the actual water raised. Thus in this plant only about 3¾ per cent of the calorific value of the fuel is realized in water pumped. To those who have not before analyzed the losses in power transmission, the amount utilized in this plant may seem absurdly small. It is, however, exceptionally large for the type of plant used. In pumping-engines of medium capacity it is seldom that more than 7 or 8 per cent of the fuel is utilized, and in the poorer type of plants the utilization of less than 1 per cent is more often the result.

A graphical analysis of the probable power losses in the De Kalb, Ill., Electric Pumping-plant (see Fig. 180) under domestic service, and from the I.H.P. of the engine, is shown in Fig. 192. In this plant the power is generated by the De Kalb Electrical Company at their central station and transmitted as a 220-volt direct current for a distance of about two-thirds of a mile to the city pumping-station, where the power

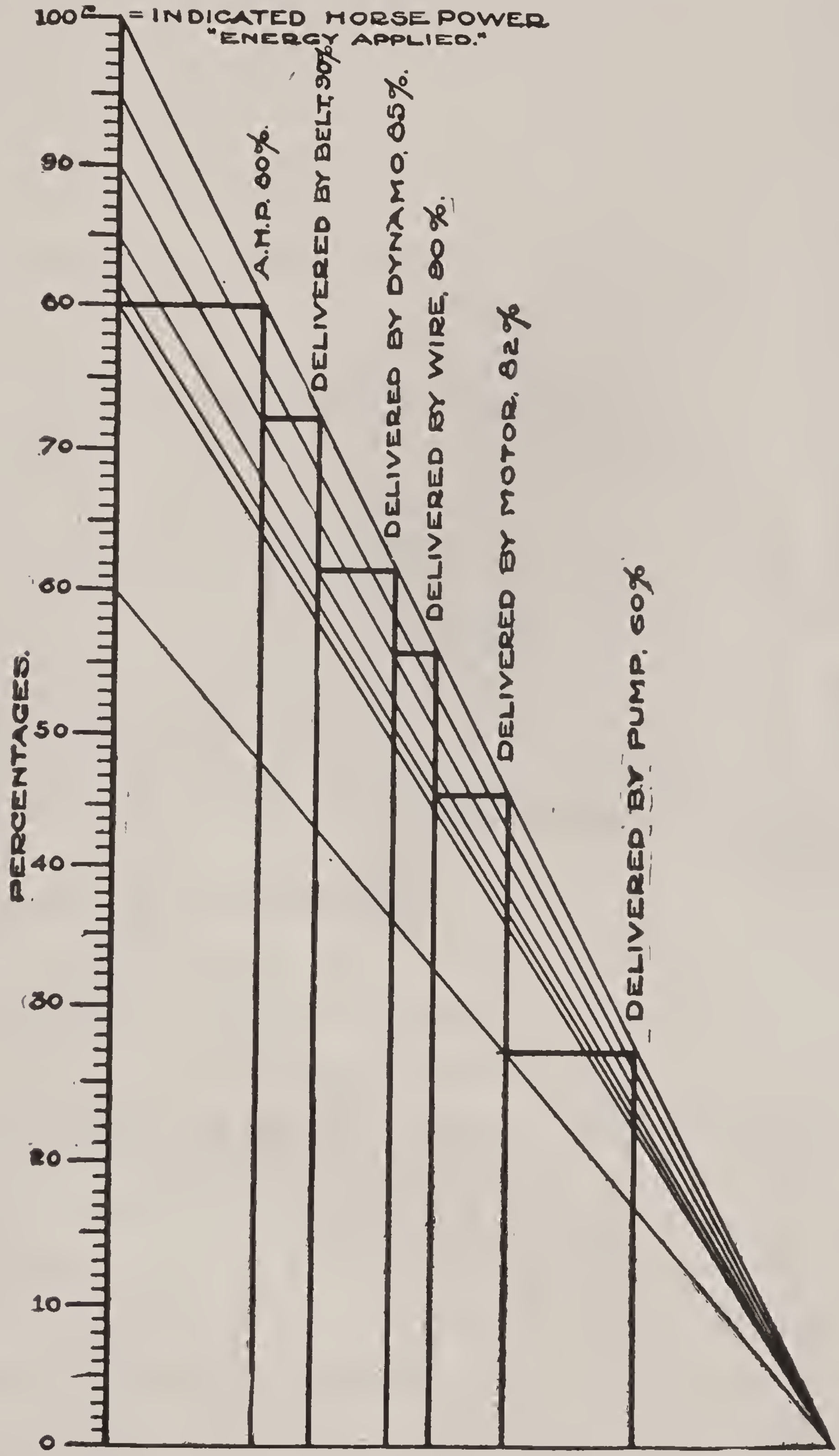


FIG. 192.—PROBABLE EFFICIENCY DIAGRAM OF THE DE KALB, ILL., ELECTRIC PUMPING-PLANT, DOMESTIC SERVICE.

is used for pumping in the motors and pumps shown in Fig. 180. From the graphical diagram it will be noted that the work delivered by the pump is about 27 per cent of the I.H.P. If the I.H.P. is but 5 per cent of the fuel-value, the total plant efficiency will be 1.35 per cent.

692. Considerations Influencing the Selection and Arrangement of Pumping-plants.—The preceding diagrams, showing the losses in energy due to the transformation of energy from coal burned to water pumped, emphasize the fact that energy cannot be transformed or transmitted without loss, and, other things being equal, the more directly energy is utilized, the more economical the results obtained. Other factors, however, must be considered in this connection. A steam vacuum-pump is a more direct application of steam than a pumping-engine. Steam is, however, here applied in a very extravagant manner, and the vacuum-pump can only be used for emergency or occasional purposes where simplicity is of more importance than economy.

A direct-acting high-pressure steam-pump is a more direct application of steam than the combination of a steam-engine and power-pump. Steam, however, is applied in this steam-pump without taking advantage of expansion, and the expansive use of steam in the steam-engine will usually more than offset the greater complication in the application of power.

Other conditions also have an important influence. When small amounts of water are to be pumped the cost of attendance may more than offset the large energy losses. Thus at De Kalb, Ill., (Fig. 180,) it was found that the De Kalb Electrical Company, having an electrical plant in constant operation, could furnish electric power at less cost, in spite of the large transformation and transmission losses (Fig. 192), than the cost at which the city could generate the power for their own plant.

The engineer should, however, aim at simplicity in arrangement and avoid all unnecessary complications. All losses should be traced and reduced to the lowest practicable amount.

The plant when selected should be arranged to facilitate its care and operation, and due regard should be taken to foresee and provide for future repairs and renewals with the least possible expense.

Fig. 193 shows the arrangement of a small pumping-plant. The plant is arranged for future duplication. The boiler is of the internally fired Scotch marine type. No brick is used for setting, but the boiler is covered with magnesia sectional covering to prevent condensation. The coal is brought from the coal-room on a car, being first weighed so that a systematic account of fuel used may be kept. The boiler is fed either by a direct-acting duplex steam-pump or by an injector. The feed-water is pumped from the main suction-pipe through a closed heater through which the exhaust steam from the engine is passed. All feed-water passes through a meter so that a daily record of evaporation may be kept. A high-speed engine furnishes power for pumping by direct connection to a triplex power-pump. The steam-

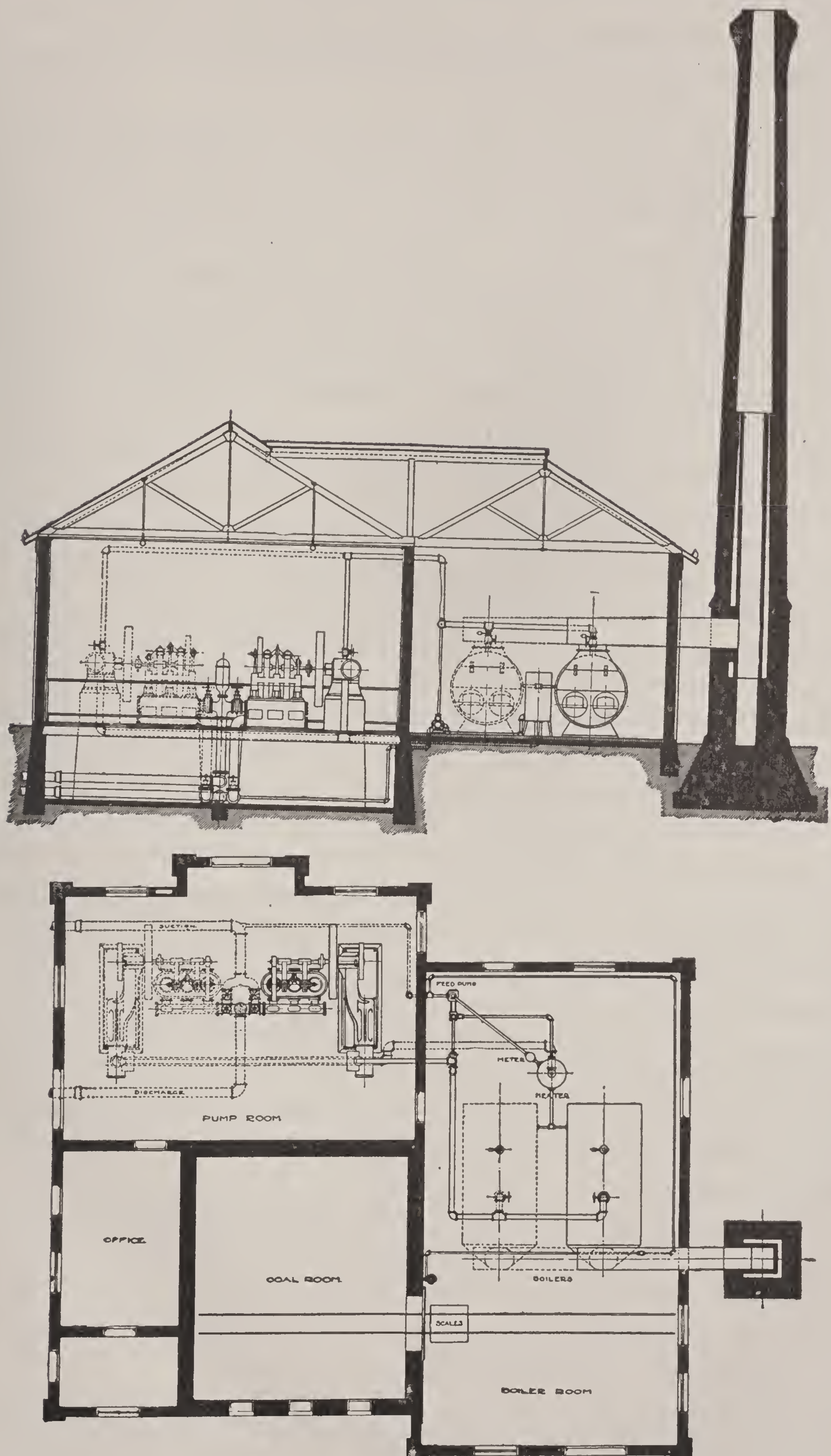


FIG. 193.—PLAN AND ELEVATION OF PUMPING-STATION, SHOWING BOILER, POWER-PUMP, AND DIRECT-CONNECTED ENGINE.

pipe is as direct as practicable—enough angles being used to allow for expansion. To prevent radiation and condensation the pipe is covered in a manner similar to the boiler.

The engine- and boiler-rooms should be large enough to allow plenty of room to work around the machinery. They should be well lighted and reasonably well finished. A good building and plenty of light are great inducements to the proper care of machinery.

693. **Capacity of Pumping-machinery.**—Two methods of pumping are possible for water-works purposes. One is that of pumping into some form of storage-reservoir. The other is that of continuous and direct pumping, in which the pump is operated at a speed just sufficient to supply the demand for water.

From what has already been stated it will be seen that for water-supply purposes there are great variations in the demands for water between the maximum and minimum consumption, and especially between minimum consumption and fire service. A pumping-plant for water-supply must be equal to the maximum demands unless the storage capacity is sufficient so that a uniform rate of pumping can be maintained and any unusual demand can be cared for by the stored supply. If the pumps are of sufficient capacity for maximum demands, their average rate of work will usually be very low, and low efficiency will often result (see Art. 669). In pumping into a storage-reservoir, the pumps can be operated at their most efficient rate regardless of consumption, which renders their operation much more economical. The direct-pressure system involves attendance night and day, while when pumping to a reservoir, night work, and consequently perhaps half the labor, can be saved in small plants. In large plants the pumps must be run night and day in any event. The variation of consumption in large plants is comparatively small, and it does not therefore greatly affect the economy of operation. The pumps can be run at or near their most efficient rate, and for great changes in consumption the variation in quantity can be cared for by starting or stopping some of the reserve machinery.

The quantity of water used in different cities for domestic consumption varies in the United States from 30 to 300 gallons per capita, according to conditions previously discussed. The quantity of water which will be needed at the maximum rate of consumption has already been considered. For fire service the number of fire-streams which should be estimated for any community depends largely on the character and nature of the community to be protected. The formula of Mr. E. Kuichling, Mem. Am. Soc. C. E., for the number of fire-streams which should be provided for any community is as follows:

$$\text{Number of streams} = 2.8\sqrt{x};$$

in which x = the population of the community in thousands (see Chapter II).

693a. **The Selection of Suitable Boiler Capacity.**—The use of the term "Boiler horse-power" is somewhat misleading, for the unit so designated is not necessarily and is in fact very seldom the equivalent of the engine or pump horse-power. "Boiler horse-power" as most commonly understood means the evaporation of 30 pounds of water from a feed-water temperature of 100° Fahr. to steam at 70 pounds pressure. Ordinarily it is sufficiently close to leave the steam pressure out of consideration, although higher pressure would mean in fact reduced evaporation.

The boiler horse-power required to do a given amount of useful work in lifting water will depend on the type of pumping machinery adopted. For steam pumping machinery and for power pumps operated by steam-engines Table No. 92 gives the steam consumption per A.H.P. per hour. Knowing the total amount of power required, the boiler horse-power may be found by dividing the total steam consumption by 30. For example a 2,000,000-gallon (1400 gallons per minute) compound direct-acting steam-pump pumping against a 200-foot head, with a duty of 25,000,000 foot-pounds per 1000 pounds of dry steam, will use 79.2 pounds of steam per A.H.P. (see Table 91). The total horse-power is closely equal to $1400 \times 200/4000 = 70$ (see Table 83). The boiler horse-power required is therefore equal to $79.2 \times 70/30 = 185$ horse-power.

Where a transmission system is in use the various losses must be estimated and the I.H.P. of the engine determined for the A.H.P. of the water pumped. From Table No. 89 the amount of steam used by various classes of steam-engines can be determined and the amount of steam needed to meet the various losses and perform the work of pumping can be determined as before. For example, with a compound Corliss engine direct connected to a generator, and operating a motor and pump two miles away, with the quantity of water pumped and the pressure as before, the losses may be assumed as follows:

Engine Friction 8 %,	efficiency	92	per	cent.
Generator,	"	94	"	"
Wire loss 5%,	"	95	"	"
Motor,	"	85	"	"
Pump,	"	80	"	"

The combined efficiency will be $.92 \times .94 \times .95 \times .85 \times .80 = .5585$ and to perform 70 A.H.P. of work will require an engine of $70/.5585 = 125.1$ horse-power.

The steam consumption of a simple Corliss may be taken at 20 pounds per I.H.P. The boiler horse-power required will therefore equal $125 \times 20/30 = 83.5$ horse-power.

694. Comparison of the Economy of Different Designs.—The first cost of a pumping-plant, and the fuel cost, are not the only considerations in its selection. There must also be considered all other costs in connection with the plant, including all expenses involved in the original installation and all expenses entailed in its operation and maintenance. For example, one pumping-plant may require a more expensive foundation, or a larger building, and the interest and sinking fund on such additional cost may more than offset any saving due to higher duty or greater efficiency.

Simplicity of arrangement is also very desirable. Complication in construction is objectionable because it necessarily entails greater expense in repairs and maintenance, and greater probability of accidents to the installation.

Pumping-machinery for fire service and for public water-supplies must be so installed as to be practically free from the danger of a failure in the service. This is accomplished either by the duplication of the plant as a whole or in part, or the storage of water under pressure, or often by both. In important cases simplicity in design and the consequent positive assurance of successful operation at all times may outweigh economy in operation.

These points cannot be given a definite value or basis of comparison, except when all facts regarding the demands on an installation are known.

The comparative financial relations of various plants can be made on the following basis:

Interest on cost of installation.		$ . . . .
Annual cost of operation:		
Labor	$	
Fuel	. . . .	
Oil.	. . . .	
Waste.	. . . .	
Light.	. . . .	
Miscellaneous.	. . . .	
Total.		. . . .
Annual cost of maintenance and repairs. . . .		. . . .
Annual debit to sinking fund.		. . . .
Total relative cost of plant		. . . .

695. *Example.*—The following is an example of the application of the above principles to a small pumping-plant which was to be installed to replace a plant already in use. The plant in use consisted of two double-acting direct steam-pumps raising water from deep bore-holes from a depth of 160 feet below the surface into a reservoir at the surface. From this reservoir the water was pumped into the water-mains against about 45 pounds direct pressure. It became necessary to increase the water-supply, which was to be secured from deep wells as before. To accomplish this various plans were investigated.

The cost of operation of the old plant was excessive. This was due to the fact that the pumps used were all extravagant of steam, and the pump which was pumping into the mains had to operate constantly and at only about one-fifth its capacity. To reduce the cost of operation a stand-tower was proposed, and its cost was included in all estimates made. A new deep well was also included in the cost of each plant.

The plans investigated were:

1. The enlargement of the old plant, including, besides the stand-tower and deep well, a deep-well pump and a direct-acting duplex steam-pump.

2. The old system enlarged as above, but substituting steam-engines and power deep-well pumps for the direct-acting deep-well pumps.

3. Using deep-well power-pumps and steam-engines, and pumping directly into mains and stand-tower without using duplex steam-pump.

4. Substituting the air-lift system for the deep-well pumps, the estimate to include compressor air-lift apparatus and new duplex steam-pump.

5. Sinking shaft 150 feet deep and installing suction-pump within reach of the water to force the water directly into mains and stand-pipes. Estimate to include shaft and pump.

Table No. 93 gives the estimate of probable results made on the various plants outlined above.

From the showing in the table under *a*, *b*, and *c*, as well as for many other reasons, it was decided that the shaft system was the best system to adopt.

Various classes of pumps could be used with the shaft system. Table No. 93(*d*) gives an estimate of the relative cost and economy of various types.

This plant was intended to have a capacity for fire service of 1,000,000 gallons per day, but an average of about 200,000 gallons per day would be used for domestic purposes. From the above table it will be observed that at the rate of pumping of 200,000 gallons per

TABLE NO. 93.

EXAMPLE OF A FINANCIAL COMPARISON FOR A PUMPING-PLANT.

(*a*) *Estimated Cost of Operation and Relative Expense.*

System.	Engineers	Repairs.	Oil.	Fuel.	Total.	Saving over Present Cost.
Present system	$1440	$362	$33	$2041	$3876	
Present system enlarged	1440	400	35	1400	3475	$401
Present system with power deep-well pumps	1440	300	35	1250	3025	851
Direct deep-well pumps	1440	350	30	800	2620	1256
Air-lift system	1440	100	25	1400	2965	911
Shaft system	1440	100	25	500	2065	1811

(*b*) *Amount of Investment Warranted by Saving Effected.*

System.	Estimated Cost.	Cost of Operation.	Saving per Year.	Saving Capitalized at 6%	Saving Capitalized at 5%
Present system	....	$3876			
Present system enlarged	$17000	3475	$401	$6683	$8020
Present system with power deep-well pumps	20000	3025	851	14185	17020
Direct deep-well pumps	21500	2620	1256	20966	25120
Air-lift system	22000	2965	911	15185	18220
Shaft system	22000	2065	1811	30183	36110

(*c*) *Financial Comparison.*

System.	Interest on Cost of Installation at 5%.	Annual Cost of Operation.	Annual Cost of Repairs, etc.	Sinking Fund.	Total Cost per Annum
Present system enlarged	$850	$3075	$400	$1015	$5340
Present system with power deep-well pumps	1000	2725	300	1235	5260
Direct deep-well pumps	1075	2270	350	1275	4970
Air-lift system	1100	2865	100	1325	5390
Shaft system	1250	1965	100	1085	4400

(*d*) *Cost of Operating Various Types of Pumps for Shaft Systems.*

Class of Pump.	Cost.	Cost. Dif.	Int. on Dif. at 6%.	Duty, 1000 lbs. Steam. Lbs.	Estimated Cost of Fuel per Year on Rates in Gallons. 200,000 Gals.	Dif.	500,000 Gals.	Dif.	1,000,000 Gals.	Dif.
Engine and power-pump	$4000			50	$350		$875		$1750	
		$2500	$150			$83		$208		$415
Corliss geared pump	6500			75	267		667		1335	
		5500	330			92		230		460
High-duty pump	11000			100	175		437		875	

day the difference in the cost of fuel between pumps of types 1 and 2 is estimated at $83 per year, or less than the interest on the difference in the cost between the two engines. The cheapest of the above pumps is therefore the most economical for these conditions. If the rate of pumping were 500,000 gallons per day, the second pump in the above table would be most economical, and if the rate were 1,000,000 gallons, the third and most expensive pump would be best.

The various other conditions which have heretofore been mentioned, and which are often as important as the financial conditions, were carefully considered. The object was and always should be to secure the best possible pumping-plant after having carefully examined the question of safety and security in construction, operation, and maintenance, and economy in the first cost, in operation and in maintenance.

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CHAPTER XXVII.

DISTRIBUTING AND EQUALIZING RESERVOIRS.

696. Office.—The forms of reservoirs to be here treated include all those which are interpolated at any point in a system between the original source and the consumer. The particular function of these reservoirs differs considerably according to circumstances, but in general they are inserted to furnish elasticity to the distributing system, that is, to enable the different portions to be more or less independent of each other in their operation.

Such independence of action is desirable from the standpoint of economy and safety, and in many cases is of importance with respect to the quality of the water. For example, where the water is brought from the source through a long conduit, a distributing or equalizing reservoir will enable the conduit to be operated at a comparatively uniform rate and hence to be made of minimum size. Likewise such a reservoir will make it possible to reduce the capacity of pumps, or filters, or other similar works, and to operate them more uniformly and economically; or in the case of small works to operate the pumps at full capacity for a portion of the day only. In the case of a ground-water supply a small reservoir will greatly increase the capacity of the source by making the demand more uniform. Again, in a large distributing system, several reservoirs placed at different points will effect considerable economy in the size of the pipe system.

As a measure of safety against the interruption of the supply from accidents to conduit or machinery, distributing-reservoirs are of great value; or, looked at in another way, additional safety against interruption may often be obtained much more cheaply by this means than by duplication.

With respect to the quality of the water a reservoir is often of great advantage, as pointed out in Art. 463, by affording opportunity for sedimentation and also by making it possible to avoid taking water from streams during periods of great turbidity.

Small reservoirs are required also for convenience in operation, such as receiving-reservoirs at the terminals of conduits, small reservoirs for regulating the pressure at intermediate points, and similar reservoirs or air-chambers at pumping-stations for equalizing the action of the pumps.

In all cases the purpose of the reservoirs here considered is to afford elasticity of operation.

697. Kinds of Reservoirs.—In discussing forms of construction, reservoirs may be classified, according to the material employed, into (1) earthen reservoirs, (2) masonry reservoirs, (3) iron or steel reservoirs, and (4) wooden reservoirs. The first two kinds can conveniently be considered together, as the two materials are very often combined in the same structure. The last two will also be treated under the general title of stand-pipes and tanks.

When the reservoir does not need to be elevated above the natural surface, the most economical form, and the usual one for large capacities, is the open reservoir with earthen embankments. The storage of surface-waters in such reservoirs does not usually affect their quality, especially if they have previously been stored in large impounding-reservoirs; but in the case of ground-waters, or filtered surface-waters, it is usually desirable that they be stored in closed reservoirs. Such reservoirs are usually built with masonry walls and covers, partly in excavation and partly above the surface. If a reservoir requires to be considerably elevated, a steel stand-pipe or a tank of wood or reinforced concrete is usually employed. A few large reservoirs have also been constructed of masonry that have extended to a considerable height above the ground.

698. Capacity.—The purpose of a reservoir of the kind here considered being chiefly a matter of economy and safety, the capacity for which it should be designed is not subject to a rigid set of rules, but depends entirely upon local circumstances. It may be wise, and good economy, for one city to have a reservoir capacity equal to 8 or 10 days' supply, while for a town located on a level plain it may be best to dispense with a reservoir and rely entirely upon reserve machinery. In determining the proper capacity, the cost of the reservoir must therefore be balanced against the benefit derived therefrom in safety and in the reduced expense for other structures and reduced cost of operation.

This question is conveniently considered in three parts: (1) the capacity necessary only to equalize the demand for a single day; (2) a capacity greater than this to provide additional safety or economy; (3) a capacity less than this where reservoirs become very expensive.

(1) In Chapter II, page 30, are given several curves showing the hourly variations in consumption throughout the day. Assuming the supply uniform, the accumulated deficiency during the hours when the rate of consumption is greater than an average would be, for New York City, about 1.4 hours' average supply; for Rochester, 2.5 hours; for Binghamton, 1.8 hours; for Des Moines, 3.7 hours; for Rockford, 1.7 hours; and for Rock Island, about 1.1 hours. The higher the consumption the less the variation and hence the less the required storage as measured by the number of hours' average supply. To equalize the demand on any particular day will then ordinarily require a storage capacity of from 1.5 to 3 hours' average consumption for the day in question, varying much according to local conditions. Assuming the same variation on the day of maximum consumption, and taking the maximum daily consumption at 150 per cent of the average, the required storage to equalize the demand on any day will be equal to 1.5 times the above figures, or 2.2 to 4.5 hours' consumption taken as the yearly average.

The quantity thus determined is sufficient only to equalize the demand during any single day, but does not provide for the variations in daily consumption. These must be met by varying the supply from pumps or conduits or other works.

In addition to the above capacity, the fire consumption for a single fire must be provided for. The maximum rate of fire consumption is given on page 31. According to Freeman, a supply of 6 hours for the full number of streams is a sufficient provision for fire. For small towns and villages 3 or 4 hours' supply at the maximum rate would in many cases be ample. The amount required for fires will, for example, be equivalent to about one day's consumption for a population of 5000, and about 6 hours' consumption for a population of 100,000, assuming an average consumption of 100 gallons per capita.

The capacity as here determined is the minimum desirable, where uniformity of operation is important for at least a day at a time; as, for example, for the clear-water reservoir of a filter system, or the storage-reservoir of a ground-water supply. It is also the minimum desirable size for the distributing-reservoir of a gravity or a large pumping system, and is less than would be used except where the cost of construction is very high. In the case of small towns where it becomes a consideration to operate the pumps but a portion of the day, the capacity must be made sufficient to furnish water during the hours when the pumps are idle, and in addition a reserve for fire extinguishment.

(2) With a capacity equal to that determined under (1), provision

for interruption of the supply for repairs would have to be made by the duplication of conduits, by reserve pumps, etc. The expense of such duplication may, however, be largely or wholly avoided by increasing the capacity of the reservoir. The best size of a reservoir depends then upon the time required for repairs, and upon its cost as compared with the expense of duplication. Where it is possible to construct an inexpensive open reservoir at a suitable elevation and in a good location it should be given a capacity of several days' supply. In practice the capacity of such reservoirs varies from 2 or 3 days' supply up to 8 or 10 days, and occasionally more. Where water is conveyed in a long conduit the larger capacity is desirable in order to avoid all danger of interruption from accidents. In a purely pumping system a very large reservoir is not so necessary, but having it, the amount of reserve power may be reduced to a minimum.

A reservoir of the kind here considered may be an elevated distributing-reservoir, or a low receiving-reservoir from which the water may be pumped, according to the local conditions.

(3) Where, owing to the topography, it becomes necessary to artificially elevate a reservoir in the form of a stand-pipe or elevated tank, the expense of construction becomes so great that the economical capacity is usually less than that mentioned under (1). The best capacity in this case depends much upon the size of the city. For large cities it is hardly practicable to provide much storage by means of artificially elevated reservoirs, the small stand-pipes which are often used in such cases serving merely to equalize the action of the pumps. In large cities the variations in demand occur more gradually than in small cities; the fire consumption is also of less relative amount, and with the large number of pumps in use their operation can be more easily varied to suit the consumption. The percentage of necessary reserve power is also much less than in small cities where the number of pumps is small.

In small cities (up to a population of 50,000 or more) it is desirable to provide a small storage even at considerable cost, as a measure of safety and economy. The fire rate is here the principal consideration, and the minimum capacity should be such as to provide water at the maximum fire rate for a sufficient length of time to enable the pumping-station to respond with ease and certainty. This is ordinarily taken as about one hour. Beyond this it will usually be desirable to add to the capacity enough to equalize the ordinary flow over several hours of the day, or, in the case of small works, to enable the pumping to be done by operating a part of the day only. The capacity beyond this minimum one-hour's fire consumption depends largely upon the cost of the tank and cost of pumping. If the tank can be placed on a natural elevation so as to reduce the height of construction, the capacity may approach that mentioned in (1) and thus reduce the amount of reserve power for fire purposes to a low figure. If the ground is level, the cost will be high and the capacity correspondingly low.

699. Location.—The location of an elevated reservoir is governed in the first place by the topography, and the choice of location is therefore often very limited. In general a distributing-reservoir should be located as centrally as possible with respect to the district to be served, as this will insure the most uniform and the highest pressures and will give the smallest size of main and branches. The best arrangement is to have several reservoirs serving as many districts, but this is seldom practicable except in very large cities, the number being usually limited to one or two.

(*a*). *The Single Reservoir.*—In a gravity system the conduit is terminated at a reservoir, and if this reservoir is centrally located a longer conduit will be required than if it be placed near one side of the system. A proper balance must be struck between the two extremes. In a pumping system the pumps are usually located near one side of the city, and the reservoir is placed either in the vicinity of the pumps or at a more remote point in the system. In the first case all the water is usually passed through the reservoir, and the action of the pumps is very steady and uniform. In the second case a main usually leads to the reservoir from some point of the distributing system. The pumps force water directly into the system, and the reservoir takes only the surplus at times of low consumption and distributes it at times of high consumption. Certain portions of the area are thus served direct, and others are served from the reservoir. With this arrangement a more uniform pressure will be maintained in the mains, but the operation of the pumps will not be as uniform. The conditions are illustrated diagrammatically in Fig. 194. Here P is the pumping-station, R is the reservoir, and AB the town to be served. During the night, water will flow into the reservoir, and the hydraulic gradient will be the line CR, say. During the day when the consumption is greater than the pumpage the reservoir will supply the deficiency, and water will flow to some point E from both directions, giving a pressure-line CDR. With the reser-

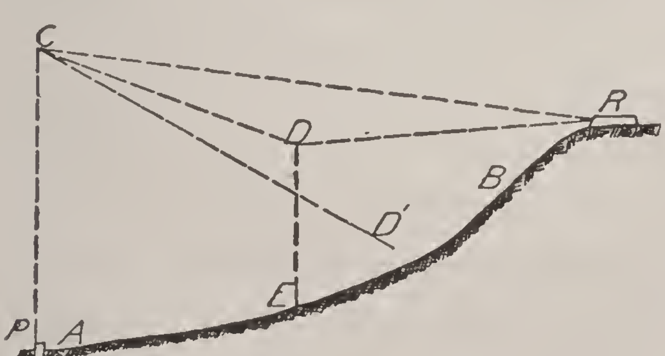

FIG. 194.

voir located at C the gradient will be a line CD', steeper than CD if the size of pipes remains the same. To give as great average pressures in this case as in the other arrangement will require larger pipes except in the immediate vicinity of R.

(*b*). *Two or More Reservoirs.*—Where two reservoirs are constructed, the best arrangement would be to locate one near the pumps, or on the side of the town where the conduit enters, and the second near the opposite side of the system; and where several can be built this scheme can be duplicated if the topography admits of it. An instructive example of such an arrangement is that of the reservoir system for the supply from the Canal de l'Ourcq, Paris, illustrated in Fig. 195.* This arrangement is especially applicable to a city located in a river valley.

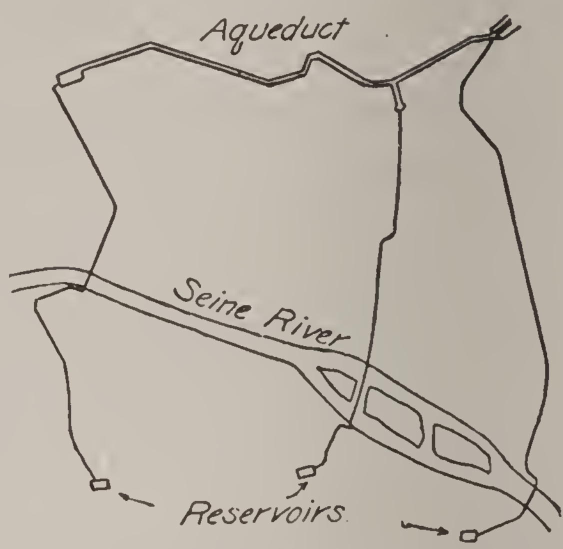

FIG. 195.—RESERVOIR SYSTEM, PARIS.

700. Elevation.—The proper elevation of a reservoir depends on the required pressure in the mains, a subject fully discussed in Chapter XXVIII. Where more than one zone of pressure is employed it will usually be possible to find sites for reservoirs to serve all but the highest zone. The latter may then be operated without a reservoir, or with a tank or stand-pipe.

EARTHEN AND MASONRY RESERVOIRS.

701. Form and Arrangement.—Earthen reservoirs are usually constructed partly by excavation and partly by the building up of embankments. If masonry walls are used in place of embankments, or as interior linings, the reservoir may be called a masonry reservoir.

When not limited by other considerations, the location and elevation of the bottom is so chosen as to secure the most economical relation between excavation and filling, which relation depends much upon the ease with which material suitable for embankments can be obtained. For single reservoirs the form most economical of material is the circular, but for large reservoirs the rectangular form is more convenient to construct and requires less land area, and except when the topography favors an irregular outline, or where the reservoir is small, it is the form usually adopted.

* Bechmann, p. 367.

Where a town is served by a single reservoir it is desirable to divide this reservoir into two basins in order that one basin may be in use at all times. This is quite necessary where cleaning must be done at frequent intervals, and it may be advisable in such a case to subdivide the reservoir still farther, as in the construction of settling-basins.

For a single rectangular basin the square is evidently the most economical form. Where a reservoir is divided into two or more parts by interior embankments or walls, the economical proportions will be somewhat different from those suitable for a single basin. The best proportions may readily be determined by trial estimates, but where the embankments are of uniform height the general formula derived in the discussion relating to settling-basins is applicable. (See Art. 479.)

702. Depth.—The most economical depth is again a matter that is in any case easily determined by trial. It will, however, be useful to determine by analysis approximately the effect of various elements on the depth. Assuming a reservoir square in plan, let x = length of one side; h = depth; Q = given capacity; and c = cost per unit area of all that portion whose cost is proportional to the area, such as land, reservoir lining, cover, etc. The cost of wall or embankment will vary approximately as h^2, or will be equal to $c'h^2$, where c' is a constant. The total cost will then be

$$C = 4xc'h^2 + cx^2. \qquad (1)$$

But $Q = hx^2$, or $x = \sqrt{\frac{Q}{h}}$, whence, substituting in (1), we have

$$C = 4c'h^2\sqrt{\frac{Q}{h}} + c\frac{Q}{h}. \qquad (2)$$

Differentiating with respect to h, equating to zero, etc., we find that for a minimum C

$$h = \sqrt[5]{\frac{Qc^2}{36c'^2}}. \qquad (3)$$

The economical depth is therefore proportional to the fifth root of Q, and hence it should vary but little for considerable variations in capacity. Since $Q = hx^2$, we have, from eq. (3), $h = \sqrt{\frac{xc}{6c'}}$, that is, h is proportional to $\sqrt{x}$. From eq. (3) we also see that as the cost per unit area increases from any cause, h should also increase, but only in the proportion of $c^{\frac{2}{5}}$. In practice the depths vary from 12 to

18 feet, for small covered reservoirs holding one million gallons or less, to 25, 30, or 35 feet, for open reservoirs holding 50 or 100 millions, depending upon local circumstances. With a fixed bottom elevation it is to be noted that the lift of the pumps increases with increased depth, which fact would tend to reduce the economical depth; also that shallow reservoirs give a less variable pressure in the distributing system. On the other hand too shallow reservoirs favor higher temperatures and increased vegetable growth, and are thus disadvantageous.

703. Embankment Construction.—The construction of the embankment is based on the same principles as discussed in Chapter XVI, but the conditions are somewhat different from those obtaining with impounding-reservoirs. Distributing-reservoirs are relatively expensive structures and are usually located in populous districts and so need to be particularly impervious. No porous form of embankment is permissible. In this case also the foundation is frequently pervious and the embankment cannot be connected with an impervious stratum below. Under such conditions it is necessary to construct a water-tight lining over the entire area, and to carefully connect it with the water-tight portion of the embankment. Where a lining is not necessary to secure imperviousness, one is usually put in to facilitate the cleaning of the reservoir.

According to circumstances the entire embankment may be impervious, or imperviousness may be secured by a puddle or concrete core, or by a layer of puddle placed near the face. The same objections are made to puddle cores as in the case of high embankments, but with perhaps less force. A puddle wall near the face, Fig. 196,* is readily

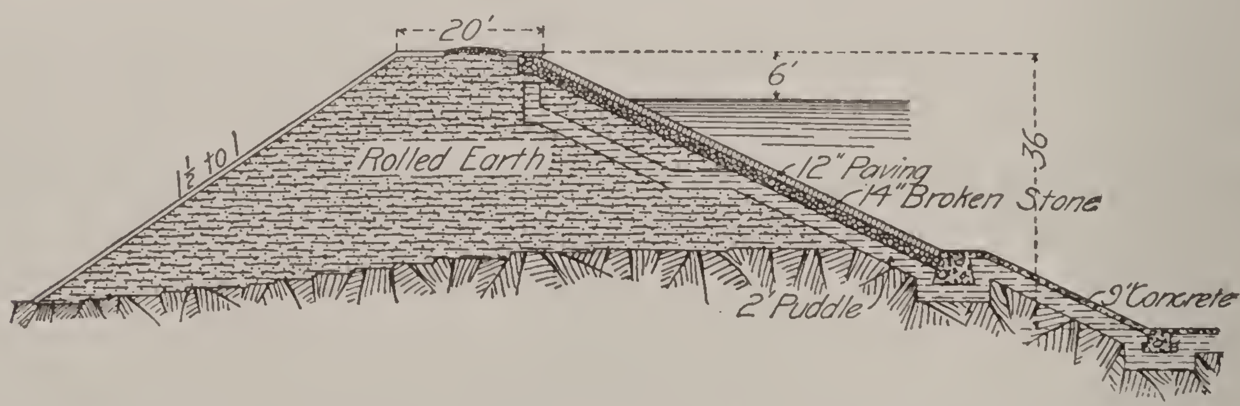

FIG. 196.—SECTION OF RESERVOIR EMBANKMENT, PITTSBURG.

connected with the bottom lining, and in this case requires less material than when placed as a core as in Fig. 197.† It gives a less firm base for the pavement, however, than coarser earth, and when the water is drawn down there is more danger of slips, such as have

* See *Eng. Record*, 1897, XXXVI. p. 54.
† See *Eng. News*, 1891, XXXVI. p. 78.

occurred in several instances. To avoid this, the paving should be placed on a layer of broken stone and have a good support at the base, somewhat as shown in the two sections here illustrated. To protect the puddle from frost action it is well to place it at some depth below the surface as in Fig. 196.

The corners of all embankments should be rounded in order to admit of convenient working with rollers. If it becomes necessary to

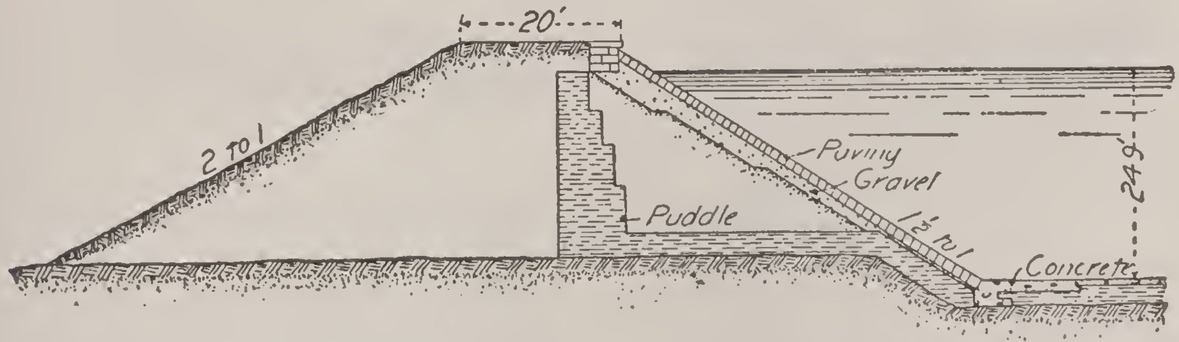

FIG. 197.—SECTION OF RESERVOIR EMBANKMENT, BROOKYLN.

support the embankment on the outside at any point by a retaining wall, such wall should be made of a strength equivalent to the portion of the embankment removed and should not be made impervious.

704. Linings of Earthen Reservoirs.—The most common form of lining consists of about 1½ to 2 feet of puddle protected by a layer of concrete, brick, or stone paving, or sometimes only by gravel. On the slopes the concrete is usually covered with paving or replaced entirely by it, experience showing that unprotected concrete is apt to be injured by ice. Various methods of construction are illustrated in Chapter XVI. Fig. 72, Art. 388, illustrates a case where the natural material was impervious and a concrete floor was all that was needed. A layer of paving-brick laid in cement makes a good finish for a concrete lining which is to be frequently exposed.

Concrete alone can be made impervious by using a rich mixture and exercising great care in placing, or it can be made impervious by a coat of cement plaster. Practically, however, such imperviousness is difficult to secure, chiefly because of the shrinkage cracks which are almost certain to develop where the exposed areas are large. To minimize this difficulty, concrete is often laid in blocks, with asphalt joints between. At Pittsburg the concrete was made in the proportions 1, 2, and 4, and laid in blocks 9 inches thick and about 7 feet square, with V-shaped joints of asphalt between, ⅜ inch wide at the bottom and ¾ inch at the top. The whole was laid on a puddle lining. The concrete was plastered with ½ inch of Portland-cement mortar, 1 to 1. A similar process was used at Minneapolis, the concrete being laid in

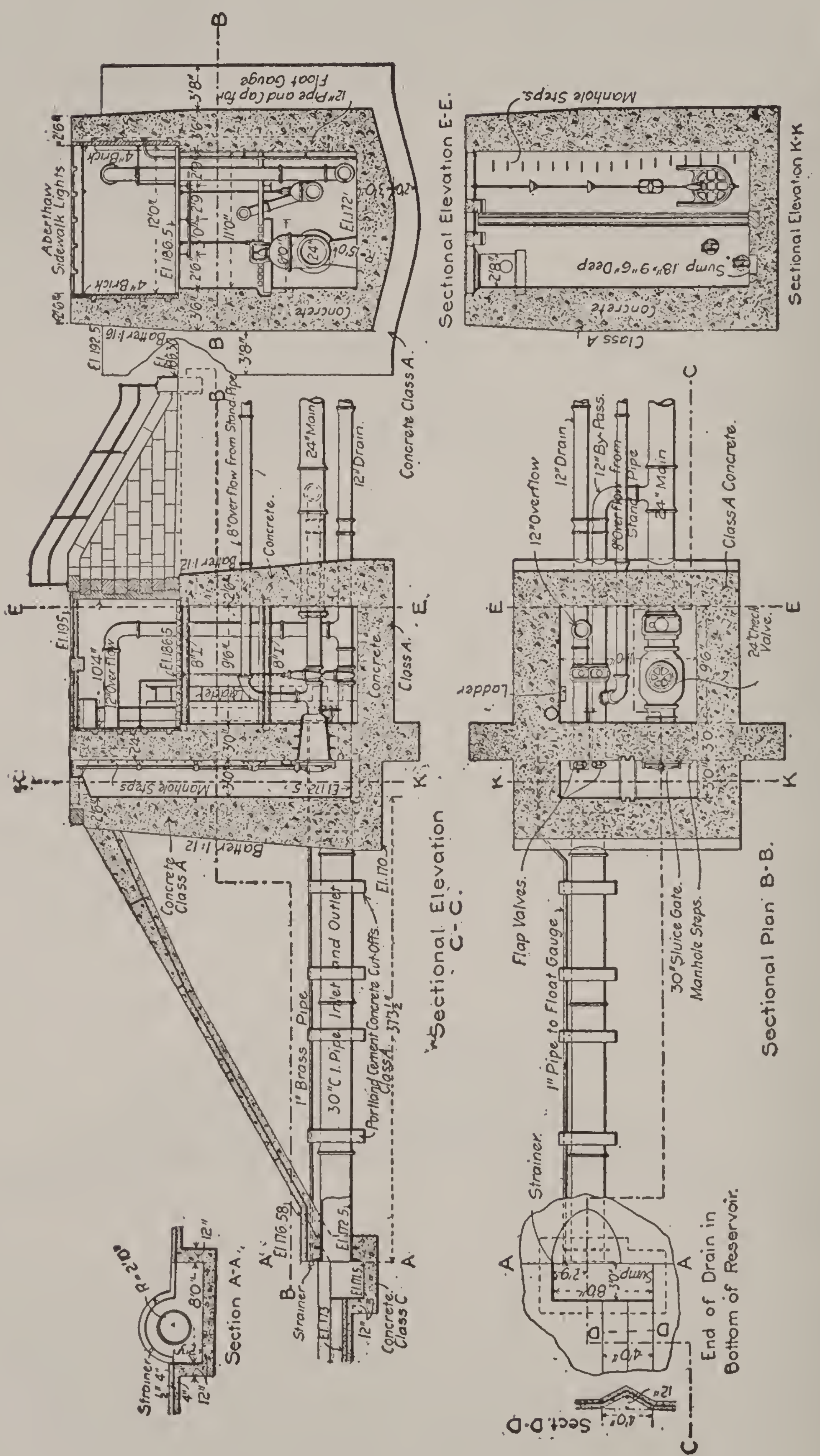

FIG. 198. FORBES HILL RESERVOIR.
(From *Engineering Record*, vol. XLV.)

20-foot squares. At the Albany filter-beds the same plan was used, it being specified that the asphalt was to remain soft at freezing temperatures. In the Forbes Hill reservoir, Fig. 198, the lining consists of three layers: first a layer of 4 inches of concrete, then $\frac{1}{2}$ inch of cement plaster for imperviousness, then 4 inches of concrete for a paving, laid in large blocks. The lining increases in thickness near the top, as shown in the figure. In some later works the lining has

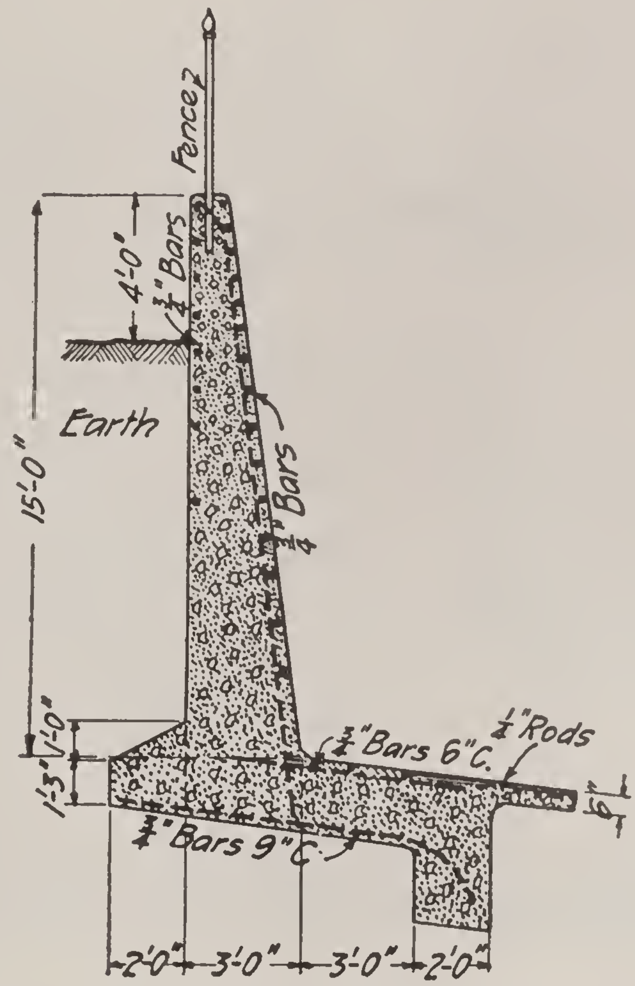

FIG. 198a.—DETAIL OF RESERVOIR, BLOOMINGTON, ILL.

been made of two layers of rich concrete, each about 3 inches thick. Each layer is constructed in rectangular blocks, the blocks of the upper layer breaking joints with those below.

By the use of reinforced concrete a much more nearly impervious floor can be made without depending upon a puddle substratum. A considerable amount of reinforcement well distributed will limit the cracks to very minute dimensions and will give a practically impervious layer. The Bloomington reservoir, Fig. 198a, is an example of such an arrangement. The bottom consists of a 6-inch layer of concrete reinforced each way with $\frac{1}{4}$-inch rods spaced 6-inch centers. In the Cobb's Hill reservoir, Fig. 198b, reinforcement is used only at the joints of the lower layer of concrete.

If ground-water is met with, which is under considerable pressure, it will be necessary, in order to avoid rupture of the floor, to drain the soil beneath the lining. In some cases the ground-water has been permitted to enter the reservoir, when its head exceeds that in the reservoir, through flap-valves which will close when the difference of head is in the reverse direction. Drainage of the soil beneath the lining should be done with great caution, and especial care taken to surround all drains with gravel and sand so graded in fineness as

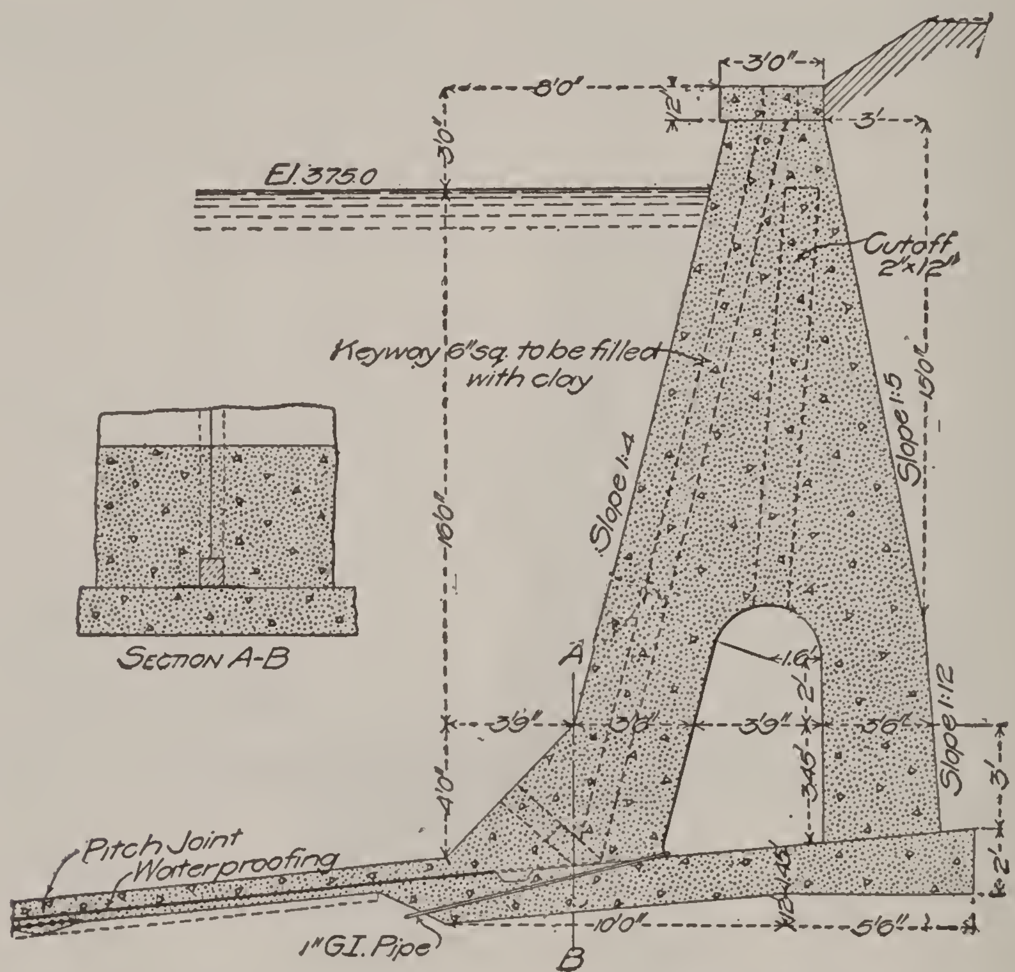

FIG. 198b. COBB'S HILL RESERVOIR.
(From *Engineering Record*, vol. LV.)

effectually to prevent the washing out of any of the material. Seepage-water is also sometimes taken care of by means of drains.

705. *Asphalt Linings.* — Asphalt is frequently used for reservoir linings with good results. It may be used for the entire lining or as an intermediate layer between layers of concrete. Used alone it has the advantages of greater elasticity and imperviousness as compared to concrete. Another advantage in many cases is its cheapness. Its chief disadvantage is the effect of the sun in rendering it more or less plastic and liable to creep if used on steep slopes. Its durability in water is also not fully determined. Great care and expert knowledge are required in determining the proper proportions of the various ingredients necessary to give good results.

Asphalt is applied either alone, or in the form of asphaltic mortar or concrete, consisting of mixtures of asphalt with sand or broken stone. For rigidity and strength the broken stone mixture is to be preferred. Regarding the use of asphalt, the following is quoted from L. J. LeConte, M. Am. Soc. C. E., who has had much experience with this material: *

"For the bottom and side slopes flatter than $1\frac{1}{2}$ to 2 the best mixture is either asphalt mortar or asphalt concrete. It is the cheapest and best lining, and there is no danger of its crawling down the slopes. For steeper slopes, up to vertical faces, this kind of lining has been tried and found wanting in many respects. Under a hot summer sun it will creep down the faces in spite of all precautions. Steep slopes or vertical walls are now coated as follows: *First*, with a cold liquid asphalt paint, which has great penetrating and adhesive properties but is lacking in sun-proof qualities; *second*, with a heavy layer of ordinary burlap, which is tightly stretched and pressed into this liquid asphalt paint; *third*, with a heavy outside coat of hard asphalt paint, put on boiling-hot. This constitutes the weather coat, and is hard, tough, and resists the hot summer sun admirably. Wherever this lining has been used, no signs of creeping have developed even on smooth vertical faces. Hard asphalt paint is lacking in adhesive qualities and consequently cannot be placed directly on the slopes. The contract price of this lining has varied from 12 to 16 cents per square foot, depending upon local conditions." The second coating referred to is usually laid at a temperature of from 300 to 400 degrees.

When the earth is firm and compact, asphalt linings can be placed directly upon it, and have frequently been so placed. Considerable settlement has in some cases taken place without cracking the lining, but this cannot, of course, be relied upon.

In relining the Queen Lane reservoir at Philadelphia, with the old concrete lining left in place, asphalt concrete 2 inches thick was used for the floor and a double layer of asphalt on the slopes, in a way essentially similar to that recommended above, with the exception that a priming coat of asphalt dissolved in benzine was first applied to all concrete surfaces of the old lining to insure good adhesion. The price was $1.15 per square yard for the bottom and $1.40 for the slopes. The slopes were furthermore lined with brick, laid flat in an outside priming coat of asphalt, to give protection from sun and ice. In the new settling-basins for the Cincinnati water-works the lining consists of concrete 6 inches, asphalt $\frac{1}{4}$ inch, brick $2\frac{1}{2}$ inches. In the Upper Belmont reservoir, Philadelphia, the lining consists of concrete, finished with a $\frac{3}{4}$-inch layer of asphalt.

* Trans. Am. Soc. C. E., 1896, XXXV. p. 94.

A commendable design is that of the Cobb's Hill Reservoir, Fig. 198b. Here the floor consists of a 6-inch layer of concrete, then a layer of waterproofing material consisting of five layers of coal tar felt laid in hot coal tar pitch, and finally a 6-inch layer of concrete laid in blocks with joints filled with hot coal tar pitch. The lower layer of concrete is reinforced underneath the joints of the upper layer.

706. **Reservoirs with Masonry Walls.** — These occupy less space than earthen reservoirs, but are more expensive to construct. They are, however, often the best form for small reservoirs where space is limited, and are a suitable form in case covers are required.

When the reservoir is excavated in firm earth or is backed by a well-compacted embankment, the earth serves to support the walls against water-pressure. They must then be designed to sustain the earth-pressure with reservoir empty. By adopting the circular form the masonry will resist largely by compression as a ring, and the dimensions can be considerably reduced below those required for a wall resisting by gravity alone. The relative resistance as a ring decreases as the square of the radius increases, but for reservoirs up to 75 or 100 feet in diameter this element may be largely relied upon for support. Several small circular reservoirs have been built of diameters of 50 to 75 feet, with walls from 16 to 22 inches in thickness.

The masonry may be of rubble, concrete, or brick, according to circumstances. If exposed, a lining of paving-brick makes an excellent finish. It is needless to say that in all work of this character the greatest care should be taken to secure the best workmanship, particularly in the mixing and laying of concrete and the thorough filling of masonry joints with mortar, essentially as in dam construction.

Imperviousness is usually secured in large masonry reservoirs by a layer of puddle placed back of the wall and thoroughly rammed, and the bottom lining is treated in a similar way. In small reservoirs more reliance is placed upon impervious masonry, made so by an asphalt coating, or by a coat of Portland-cement mortar, or by the use of reinforced concrete. In covered reservoirs cracks are easier to prevent than in open reservoirs as the temperature changes are much more moderate.

Two modern examples of reservoir walls are illustrated in Figs. 198a and 198b. The former is a reinforced concrete wall forming part of a circular reservoir of 300 feet diameter. The wall is thoroughly connected to the floor, which is also reinforced so that the entire structure is a concrete monolith. No expansion joints are used, the circular form being favorable to such construction. The inner face of the wall was coated with a 1 : 1 mixture of waterproof cement and the floor finished with a surface coat of 1 : $1\frac{1}{2}$ mortar.*

In the Cobb's Hill reservoir plain concrete is used for the walls. These are constructed in sections 20 feet long and at the joints a key-way is provided which is filled with clay puddle. A passageway for inspection purposes is built in the wall, and to assist in detecting leaks a series of drain pipes are provided leading from beneath the floor into the passageway.†

While it is comparatively easy to secure imperviousness at the outset by the use of cement, it is difficult to prevent the formation of slight cracks. These permit the water to find its way into the surrounding soil, and when the reservoir is quickly emptied this water exerts a back pressure on the walls and an upward pressure on the floor. It is also likely to injuriously affect the backing and the foundation. This contingency may be provided against by draining the backing outside and near the base of the walls, and the ground beneath the floor. In some large masonry reservoirs constructed in France, a double bottom was put in, with a large interior space from which all seepage is removed by drains. Drains also lead into these galleries from behind the exterior walls. In this way water is prevented from soaking into and weakening the foundation (Fig. 204).

In the case of covered reservoirs, the floor may be designed to resist the upward pressure due to ground-water by the use of inverted groined arches, held down by the piers supporting the roof.

707. **Arrangement of Pipes, Valves, etc.**—Distributing-reservoirs are usually provided with separate inlet and outlet-pipes, located preferably on different sides of the reservoir in order to promote circulation of the water. In earthen reservoirs these are constructed in the same manner as described in Chapter XVI. A by-pass should be provided to enable the reservoir to be cut out at any time. The gate- or valve-chamber will vary in design from a single vault placed over a gate-valve, to an elaborate structure provided with screens and arrangements for drawing water from different levels, as in the Syracuse reservoir (page 363), according to the size of reservoir and the necessities of the case. To prevent flooding of the embankments from carelessness in operation, an overflow must be provided. This is merely an open-

* *Eng. Record*, 1906, LIII. p. 285. See description of Baden Reservoir, St. Louis, in *Eng. Record*, 1905, LII. p. 454, for example of another type of reinforced concrete wall.

† *Eng. Record*, 1907, LV. p. 254.

ended pipe or short weir, admitting water to the gate-chamber or to a manhole, whence it is conducted away by a drain-pipe laid through the embankment. To facilitate draining and cleaning, a waste-pipe should lead from a low point in the floor of the reservoir. These details are illustrated in Figs. 198 and 199 and in Figs. 77 to 81 of Chapter XVI.

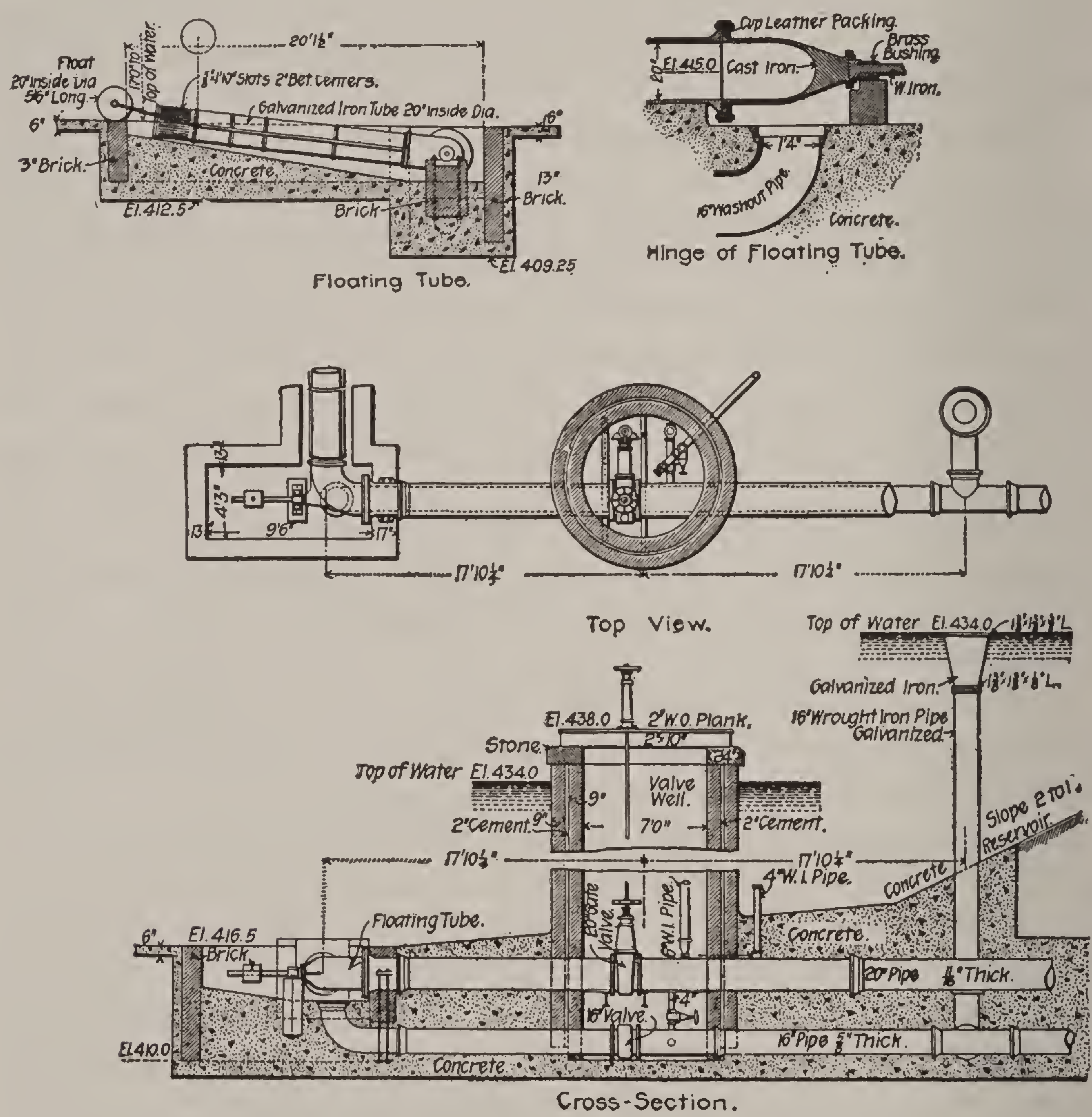

FIG. 199.—OUTLET-PIPE DETAILS, STEUBENVILLE RESERVOIR.
(From *Engineering Record*, vol. XXXVIII.)

Where the reservoir serves merely as an equalizing-reservoir, receiving only the surplus water from the distributing system, a single pipe will serve for both inlet and outlet. Circulation of the water can be secured by extending the pipe to the center, or beyond, and there placing a flap-valve through which water is admitted to the reservoir. A branch pipe opening near the side of the reservoir can then be made to act as an outlet-pipe only, by the use of a check-valve opening outwards.

For reservoirs serving partly or wholly as settling-reservoirs, the adjustable outlet-pipe shown in Fig. 199 is advantageous in enabling the water to be drawn off at all times from near the surface. The pivoted arm is provided with a float and screen, and, in some works, provision is made to draw it to any desired depth by means of a chain and windlass.

In open masonry reservoirs gate-chambers are conveniently built in connection with the reservoir wall. In covered reservoirs they are usually omitted, the valves being placed within the reservoir and operated from a suitable platform or from the outside.

708. **Covered Reservoirs.** — In Chapter IX, Art. 196, the effect of storage on various classes of waters was discussed. It was there shown that ground-waters should be stored in covered reservoirs, for the reason that such waters usually contain sufficient quantities of plant-food to promote a luxuriant growth of vegetable organisms unless the light be excluded. Many cases have arisen of bad tastes and odors due to this cause which have been entirely removed by covering the reservoir, but the conditions are often so favorable for the growth of plants that considerable care must be taken to exclude all light. Filtered surface-waters should also as a rule be stored in covered reservoirs, since by the process of filtration they are rendered somewhat similar in nature to ground-waters. Where reservoirs are located in the densely populated portions of cities, covers are also advisable, in order to exclude soot and dust. Distributing-reservoirs are almost universally covered in European works, and as the use of filtered supplies becomes more general in this country covered reservoirs will become more common.

Covers are usually made of masonry, but wood has been used in a number of cases. It is much cheaper than masonry, but is much less durable and does not keep the water as cool in summer or wholly prevent freezing in winter.

709. *Wooden Covers.* — A wooden cover for a large area may consist simply in a horizontal floor of boards, supported by a system of joists and girders resting on a series of wooden posts. No attempt need be made to exclude the rain. For small areas the covers can readily be made sloping, and this is a preferable arrangement. Covers for small circular reservoirs and large wells are conveniently made conical, with the rafters resting against the wall or supported on light trusses.

710. *Masonry Covers.*—Masonry covers are now generally made of concrete, either in the form of groined arches of plain concrete or flat slab and beam construction of reinforced concrete. Piers are spaced from 10 to 15 feet apart. They were formerly made of brick, but now are generally made of concrete proportioned at ordinary working stresses. Above the arches, about 2 feet of earth is placed to prevent extreme variations of temperature and to protect the masonry, and embankments are constructed against the side walls to meet the covering above. As the loading is all dead load, a low factor of safety may be employed and piers and arches made relatively light. Much of the older construction is heavier than necessary. Table No. 94, by Coffin,* gives the dimensions and pressures for brick piers of several covered reservoirs.

TABLE NO. 94.

DIMENSIONS OF AND PRESSURES ON PIERS OF COVERED RESERVOIRS (COFFIN).

Reservoir.	Height. Feet.	Cross section. Sq. Feet.	Area of Tributary Roof Surface. Sq. Feet.	Approx. Weight on Pier. Tons.	Pressure on Pier. Tons per Sq. Foot.
Newton	13.5	2.78	136	32	11.5
Brookline	17.5	4.00	144	26.5	6.63
Franklin	16.5	1.00	90.5	20	20
Ashland	5.0	4.00	248	54	13.5
Wellesley	12.25	4.00	196	51	12.75
Albany	7.50	2.78	187	41	14.75
Clinton	7.0	4.00	210	78	19.50
Proposed	7.0	2.78	196	46	16.55

As between the groined arch of plain concrete and the flat reinforced concrete cover the former is probably the cheaper for large reservoirs, as in such a case the construction and manipulation of forms can be reduced to an economical system. In modern designs groined arch covers have been worked out to very economical dimensions, so that little gain can be effected by the use of reinforcement. An example of such design is shown in Fig. 200, representing a clear-water basin at one of the Philadelphia filtration plants. Fig. 201 illustrates the common type of concrete cover with exterior walls built of the same material. This form, or the circular reservoir, is probably the most economical type for small capacities. Fig. 122, p. 468, shows the

* From a very complete paper on covered reservoirs in Jour. Assn. Eng. Soc., 1900, XXXIII. p. 1.

interior of a filter where the groined concrete arch is used. Another example of the use of the groined arch is in the Albany filter illustrated

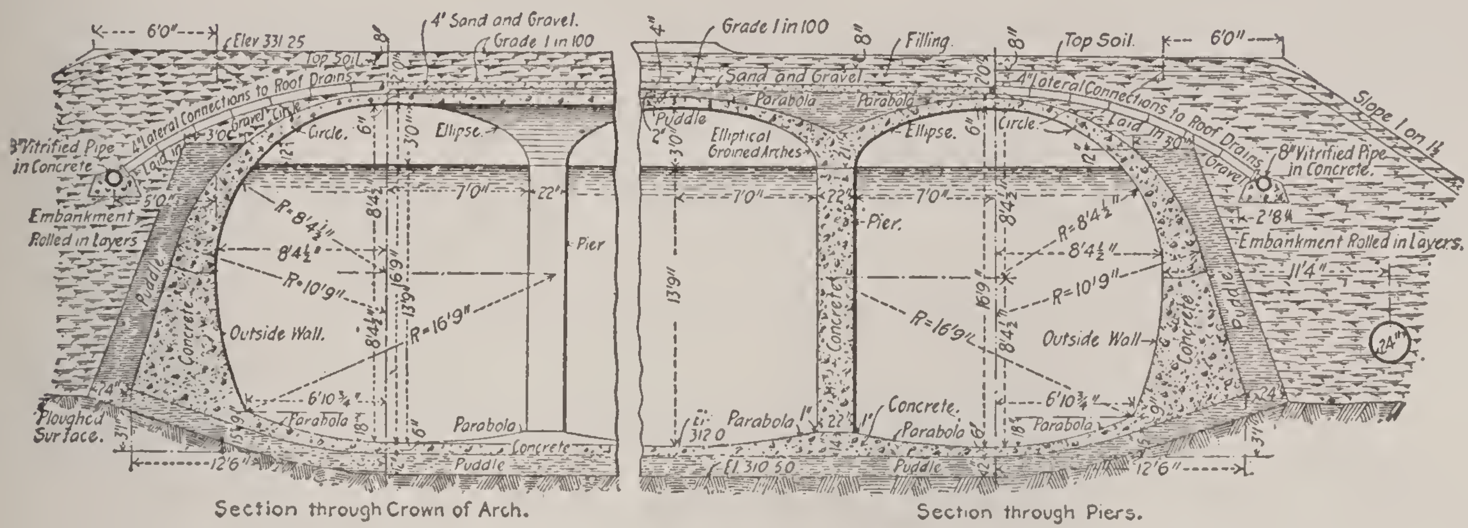

FIG. 200. —CLEAR-WATER RESERVOIR, PHILADELPHIA.
(From *Engineering Record*, vol. XLII.)

on pages 465 and 477. Here the span is 12 feet and rise $2\frac{1}{2}$ feet. Concrete arches were used with a thickness at the crown of 6 inches.

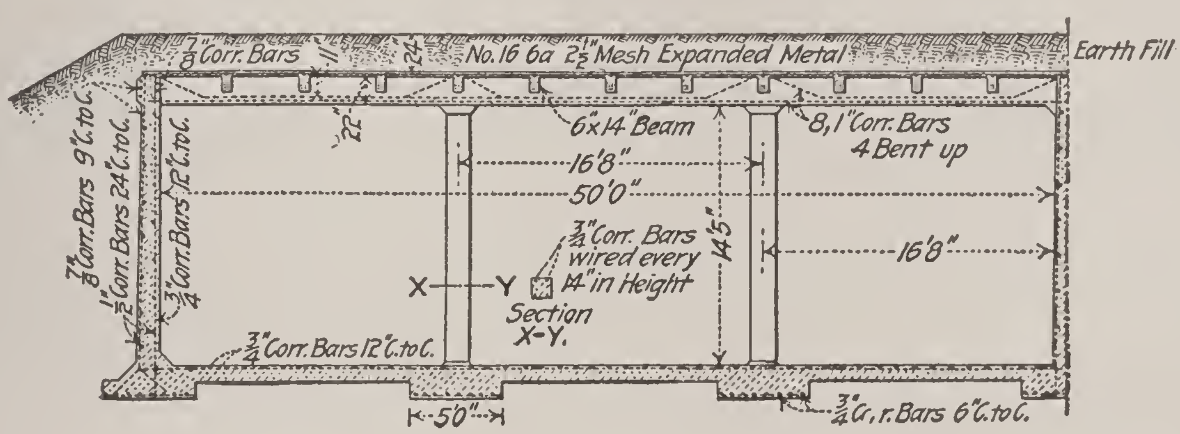

FIG. 201.—FORT MEAD RESERVOIR.
(From *Engineering News*, vol. LIV.)

Fig. 202 is an illustration of a modern design in which the plain concrete arch and invert is used in connection with reinforced walls. Pile foundations were there necessary. The concrete is depended upon for imperviousness, the wall joints being made by means of wide steel plates or keys.

The stresses in groined arches are so complicated and uncertain that an analysis based on the assumption of simple arch action is of very little value. As a matter of fact the concrete probably acts more as a cantilever than an arch. This was shown to be the case at the Albany filters, where the arches actually opened at the crown from the effects of temperature changes, the concrete being so constructed as to give a line of weakness at this point. The economical proportions can best be determined by tests or from actual experience. The dimensions shown in the illustrations have proven abundantly large.

In Paris, Belgrand has constructed covers of concrete groined arches with a thickness of but 2.8 inches at the crown for a span of 13.3 feet, and 4.4 inches for a 20-foot span. The piers were from 10 to 17 feet high and 13 to 18 inches square.*

Piers should be spread out at the base so as to distribute their load sufficiently to avoid practically all settlement, and the floor should be

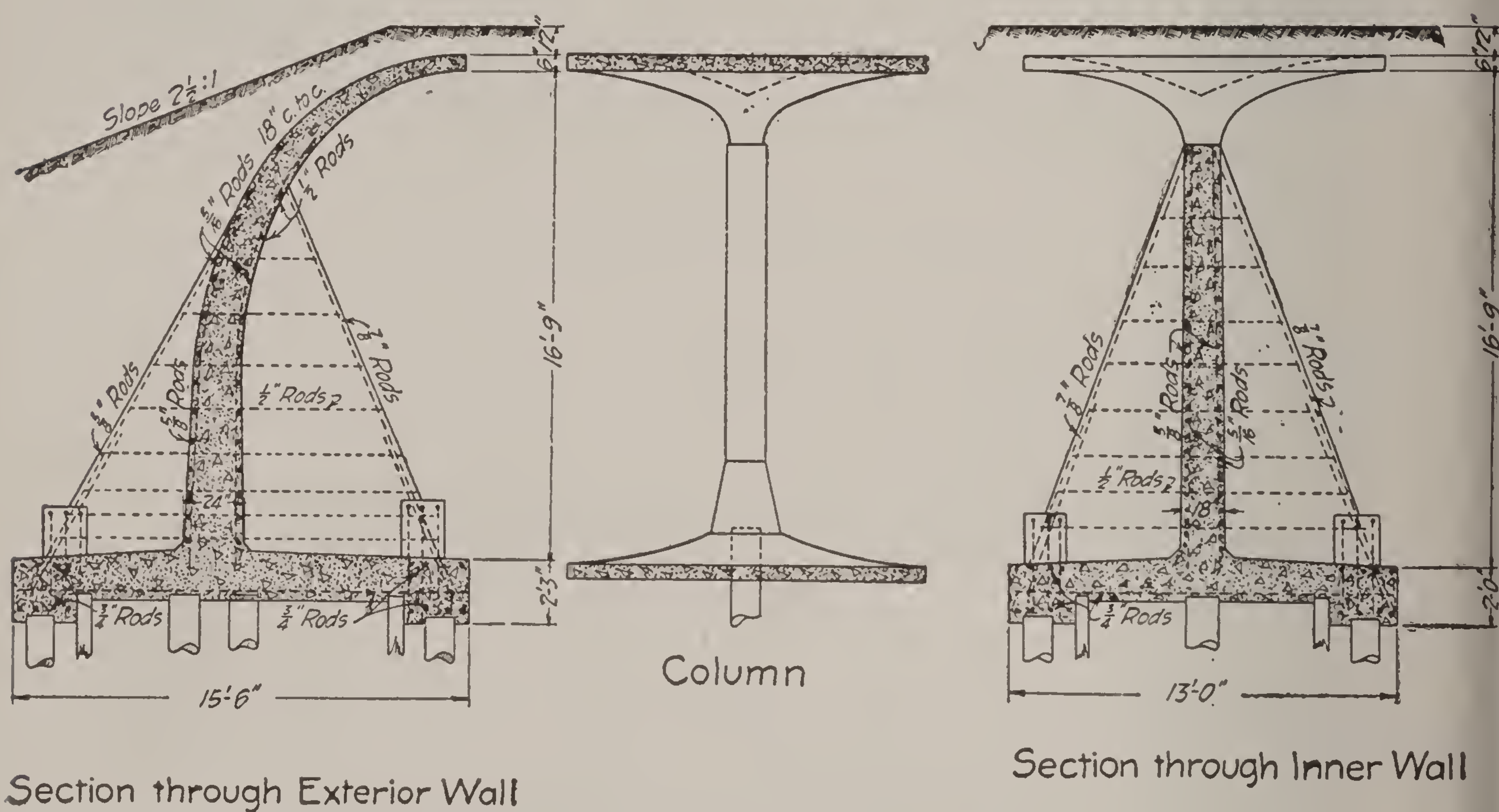

FIG. 202.—RESERVOIR DETAILS, NEW ORLEANS.

well bonded thereto. Where the foundation is soft, inverted groined arches may be used, thus distributing the weight over the entire area.

711. *Exterior Walls.*—Where vaulted covers are used the exterior walls must of course be designed with reference to the arch thrust. In the earlier designs of brick and stone masonry these walls were of relatively heavy section having vertical inside and battered outside face, but in the more modern designs of concrete they have been proportioned on more economical lines. This point is well shown in Fig. 200. A somewhat extreme design of this character is shown in Fig. 203.†

* See analysis of stresses in Coffin's paper, also in paper by Metcalf in Trans. Am. Soc. C. E , 1900, XLIII. p. 37.

† *Zeit. Ver. dt. Ing.*, 1898, XLII. p. 1059.

Fig. 202 illustrates the use of reinforced exterior and division walls economically proportioned. In case the cover is of reinforced concrete, as in Fig. 201, the exterior and division walls are of simple reinforced construction and are very economical.

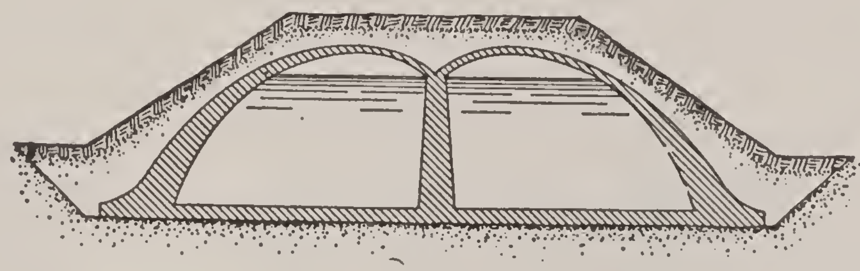

FIG. 203. — SMALL RESERVOIR AT VIENNA.

712. Masonry Reservoirs Above Ground. — Where no suitable elevation can be found for a reservoir of the kind already considered it will be necessary to provide artificially the required elevation. Ordinarily a small elevated tank is made to suffice, but in the case of large cities served by long conduits it is desirable to have a larger storage capacity, and in a few instances this has been provided for by large masonry reservoirs. In these reservoirs the thickness of the walls is determined in the same way as for masonry dams. Sometimes earth embankments are thrown up around the walls to maintain the water at a lower temperature. Very interesting examples of high masonry reservoirs are furnished by those of Paris. The most remarkable perhaps is the Montmartre reservoir, which is four stories high, each of the upper three stories being used for different services. The lowest story is for the piping and for the drainage of water which leaks through the floor. Fig. 204 shows a section of this reservoir. Reservoirs of this class should receive careful architectural treatment and may be made fine monumental works.

In repairing the inevitable cracks which appear in these large reservoirs, the cracks are first cleaned out and then vulcanized rubber strips are cemented in with rubber cement. and the whole is covered with mortar.*

713. Cost. — The cost of reservoirs varies of course greatly according to local conditions, kind of reservoir, and capacity. According to capacity the cost per unit will be less the larger the reservoir. If in eq. (2), Art. 702, we substitute the value of h from eq. (3), we have $C =$ constant $\times Q^{\frac{4}{5}}$, or the cost per unit capacity $= \dfrac{C}{Q} = \dfrac{\text{constant}}{Q^{\frac{1}{5}}}\cdot$ that

* *Eng. News*, 1895, XXIV. p. 419.

is, the cost of reservoirs per million gallons will vary inversely approximately as $Q^{\frac{1}{5}}$. Thus if a reservoir with a capacity of 100 million gallons

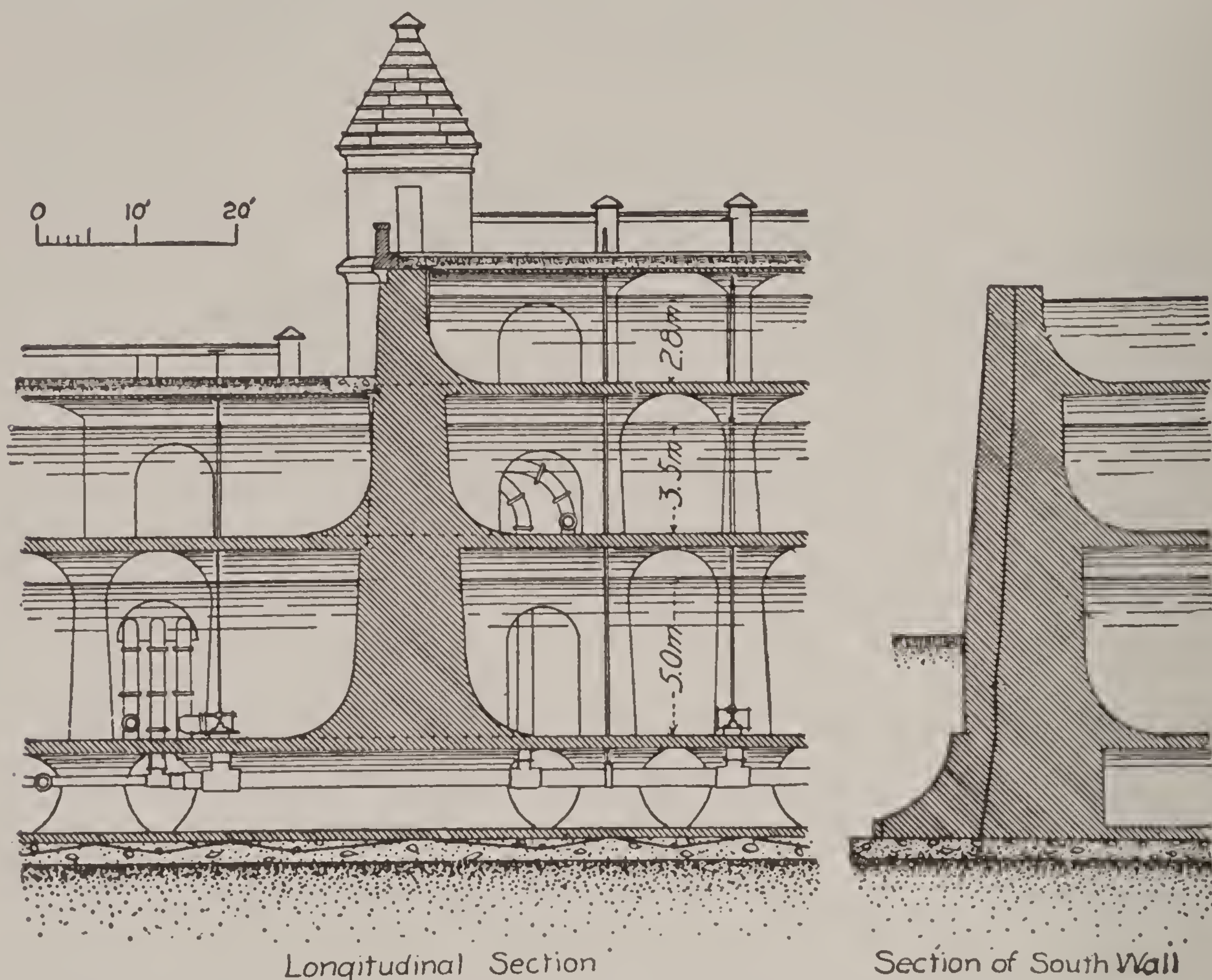

FIG. 204.—MONTMARTRE RESERVOIR, PARIS.

costs \$3.00 per thousand gallons capacity, one of 10 millions, similarly constructed, will cost about $3 \times 10^{\frac{1}{5}} =$ about \$4.75 per thousand gallons; one of 1 million capacity will cost \$7.50, etc.

The actual cost of several reservoirs is as follows:

Place.	Capacity in Gallons.	Cost per 1000 Gals. Capacity.	Remarks.
Earthen reservoirs:			
Pittsburg, Highland Park	125,000,000	\$3.35	
Trenton, N. J.	104,000,000	3.37	
Minneapolis, Minn., two, each	46,000,000	4 80	
Cincinnati, O., subsiding reservoirs	50,000,000	3.50	Estimated.
Covered masonry reservoirs:			
Albany, Ga	280,000	6.30	Wooden cover.
Coshocton, O.	320,000	13.47	Brick dome.
Franklin, N. H.	504,300	18.00	Brick arches.
Wellesley, Mass.	600,000	17.35	Concrete cover.
Rockford, Ill.	1,000,000	20.00	Monier construction.
Brookline, Mass.	1,200,000	22.00	Brick arches.
Montmartre, Paris	3,000,000	74.00	Elevated reservoir.

Mr. Coffin estimates the cost of circular reservoirs with concrete covers as follows :*

Capacity. Gallons.	Diameter. Feet.	Depth. Feet.	Cost per 1000 Gallons Capacity.
500,000	75	16	15.60
1,000,000	98	18	12.85
1,500,000	115.5	19	11.70
2,000,000	125	22	11.00
3,000,000	144	25	10.07
4,000,000	166	25	9.47
5,000,000	186	25	9.12

His estimates for square reservoirs are about 4 per cent higher than the above figures.

STAND-PIPES AND ELEVATED TANKS.

714. Where a reservoir requires to be artificially elevated it is usually built as a stand-pipe — a tall slim tank resting on the ground — or as an elevated tank of steel, wood, or reinforced concrete, supported by a suitable tower. Such an elevated reservoir may or may not be enclosed in a covering of masonry or wood, according to the necessities of the case and the notions of the designer.

Reservoirs of this type are relatively so expensive that a minimum amount of storage capacity is usually provided. As shown in Art. 698, they may be used in small towns to enable the pumps to be more economically operated, or in larger towns to provide for fire consumption for an hour or so, or in large cities to act merely as equalizers for the pumps. The capacities of stand-pipes and tanks range ordinarily from 50,000 gallons up to a maximum of about 1,500,000 gallons.

715. Location. — For storage purposes only, the location would be the same as that for any other reservoir, as discussed in Art. 699. To reduce the cost, it is, however, desirable to place the tank on the highest ground available if it be within a reasonable distance. Too great distances will be undesirable on account of the cost of mains and the loss of head caused by a long line of pipe. If the stand-pipe acts simply as a pressure-regulator, it should be located near the pumping-station, or at least at some point on the force-main before any considerable number of branches occur.

Steel Stand-pipes.

716. General Dimensions. — The useful capacity of a stand-pipe is only that part of the volume which is at a sufficient elevation to give

* *Jour. New Eng. W. W. Assn.*, 1900, XIV. p. 283.

the required pressure. All water below this level acts merely as a support for the portion above. There should therefore first be determined the lowest useful level of the water, and the pipe should then be made of the desired capacity above this plane. The ratio of height to diameter should be chosen with respect to the following considerations: Cost of pipe and foundation, variation in water-level, cost of pumping, and practicable thickness of plates.

If Q = useful capacity in gallons, H = height of pipe in feet up to the lowest useful level, x = additional height necessary to give the desired capacity Q, and d = diameter of pipe, then $Q = 5.9xd^2$ and $xd^2 = \frac{Q}{5.9}$, = a constant, = K. The weight and cost of the pipe-shell are nearly proportional to $(H + x)^2$ and to d^2, or, Cost $= K'd^2(H + x)^2$; but $d^2 = \frac{K}{x}$, whence, Cost $= K'K\frac{(H + x)^2}{x}$. Differentiating, etc., we find that for a minimum cost, $x = H$. That is, the total height should be $2H$, and $d = \sqrt{\frac{Q}{5.9H}}$. This result will of course be modified by the other considerations mentioned above, but the relation brought out will aid in selecting the best dimensions. If H is large, this rule would be likely to give such a height as to make the variations in pressure too great and also give too heavy plates at the bottom, plates thicker than $1\frac{1}{8}$ inches being undesirable. A high tank will also increase the cost of pumping. On the other hand a large diameter will increase the cost of foundation. It is on the whole desirable to use rather large diameters. With ordinary values of capacity, and with H equal to 50 to 100 feet, the best value of x will probably be from $\frac{3}{4}$ to $\frac{1}{2}H$. The best proportions can readily be determined by trial estimates of cost of pipe and additional cost of pumping per foot in height, having regard to the limiting conditions mentioned above.

If the entire volume can be counted on as useful, then $H = 0$, and the best proportions will depend very largely upon cost of foundation and cost of pumping. Neglecting the last item, and assuming as before that the cost of shell varies as x^2d^2 and that the cost of foundation and bottom plate is proportional to the area, or to d^2, it will be found that the economical height is the same for all capacities, and is in the neighborhood of 25 to 30 feet. The cost of pumping additional height will tend to reduce this slightly, while the cost of the upper plates, whose thickness must be much greater than required for water-pressure, will tend to increase it, so that for very large pipes, 40 feet will be more nearly the economical height.

If a stand-pipe is used only as a relief to the pumps, its diameter may be made from 3 to 6 feet. A diameter of twice that of the force-main would reduce the rate of variation of pressure on the pumps to one-fourth that in the mains, which would be sufficient in most cases.

717. **Forces and Stresses.**—The forces to be considered in the design of a stand-pipe are the water-pressure, the wind-pressure, the weight of the pipe, and the action of ice. In what follows let h = distance in feet of any point below the top, d = diameter of pipe in feet, r = radius in feet, and t = thickness of shell in inches at any given point.

The water-pressure causes a stress per vertical lineal inch of pipe equal to

$$S = \frac{62.5hd}{2 \times 12} = 2.6hd. \qquad (1)$$

The stress per square inch is

$$s = \frac{2.6hd}{t}. \qquad (2)$$

The wind-pressure is usually taken at 40 to 50 pounds per square foot on one-half the vertical projection of the tank. At the higher figure the bending moment in foot-pounds at any distance h below the top, caused by the wind, is

$$M = 50 \times \frac{dh}{2} \times \frac{h}{2} = 12.5dh^2. \qquad (3)$$

This moment causes a maximum stress in the shell of the pipe (the extreme fibre) equal to

$$s' = \frac{My}{I}.$$

In this case $y = r$, and $I = \frac{1}{4}\pi(r_1^4 - r_2^4)$ = approximately $\pi\frac{t}{12}r^3$, (t in inches,) whence, in pounds per square inch,

$$s' = \frac{1}{144} \cdot \frac{Mr}{\frac{\pi t r^3}{12}} = 1.33\frac{h^2}{td}, \qquad (4)$$

and the stress per lineal inch along a circumferential line will be equal to

$$S' = 1.33\frac{h^2}{d}. \qquad (5)$$

If W = weight of pipe in pounds, the stress per lineal inch due to its weight will be

$$S'' = \frac{1}{12} \cdot \frac{W}{\pi d} = .026\frac{W}{d}, \qquad (6)$$

and per square inch will be

$$s'' = .026 \frac{W}{dt}. \qquad (7)$$

Assuming the average thickness above the point in question to be $\frac{t}{2}$ and adding 15 per cent for laps, etc., the weight W will be approximately equal to $75dth$, and hence $s'' = 1.9h$, which value will never exceed a few hundred pounds.

Besides the overturning effect of the wind there is to be considered the collapsing effect on the empty pipe, especially near the top where the plates are thin. This cannot readily be computed, but must be provided for by an ample margin of strength at the top of the stand-pipe.

The effect of ice action is a very serious matter in unprotected stand-pipes, but is very difficult to calculate or provide for. It may occur in various ways. During severe weather a heavy cylinder of ice will form next to the shell. A warm spell may cause this to melt somewhat around the outside, and then the water in the annular space thus formed may again freeze, causing a heavy bursting pressure. Or the water may be drawn down a considerable distance after heavy ice is formed so that a thaw will allow the mass to drop, thus causing heavy water-hammer; or, after the water is drawn down, the pipe may be so rapidly refilled as to blow out the ice cover, causing sudden shocks and stresses. The importance of this matter is attested by the many accidents traceable to the action of ice.*

The stresses caused by ice action can only be provided for by the use of a good quality of soft steel which will allow of deformation without injury, and by the use of a large factor of safety. It may well be questioned, in view of the uncertainties of the case, if all metal tanks built in cold climates should not be encased in masonry or wood. The construction of exposed metal tanks in cold climates would scarcely be considered possible in the more conservative European practice.

718. Material Employed.—The material used for stand-pipes should be soft, open-hearth steel, of a tensile strength of about 54,000 to 62,000 pounds per square inch. The best practice now calls for a grade corresponding to flange steel, with phosphorus limit of about .06 per cent, an elongation of 22 to 25 per cent, reduction of area of 50 per cent, and flat bending tests, both cold and after heating and quenching. Many stand-pipes were formerly constructed of cheap tank-steel, which is doubtless one of the principal causes of the many failures.

* See reference 14, p. 740.

Rivets, being hand-driven, are preferably made of wrought iron. Plates thicker than $\frac{3}{4}$ inch should be drilled.

719. Thickness of Plates.—The safe tensile stress on net section, where but little ice is likely to form, may be taken at about 15,000 pounds per square inch. Where thick ice is to be expected the working stress should be reduced to 12,000 or even 10,000 pounds, to provide for the unknown ice stresses. The vertical joints will usually be so designed as to have an efficiency of 60 to 70 per cent. If a = safe stress on net section and e = efficiency, then by eq. (2), page 713, the required thickness to resist the water-pressure will be

$$t = \frac{2.6hd}{ae}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

or, if $a = 12,000$ and $e = \frac{2}{3}$, then, approximately,

$$t = \frac{2.6hd}{8000} = .000325hd. \quad . \quad . \quad . \quad . \quad . \quad (9)$$

The thickness near the top should not be less than $\frac{1}{4}$ inch, or for very large pipes, $\frac{5}{16}$ inch. Plates thicker than 1 inch or $1\frac{1}{8}$ inches should be avoided.

The stresses due to wind and weight need not be considered here, as they act at right angles to the stresses due to water-pressure and are also much less in amount.

720. Riveting.—The plates forming a stand-pipe are usually of such a width as to build 5 feet of pipe, and are from 8 to 10 feet long. Each course is preferably made cylindrical, and alternately an "inside" and an "outside" course.

The riveting of the vertical seams is the most important part of the construction, as this determines the strength and economy of the stand-pipe. Lap-joints are most commonly used, but for thicknesses exceeding $\frac{1}{2}$ inch, double-butt strap-joints are much preferable and are stronger. The butt-joint is arranged as shown in Fig. 205, thus avoiding the forging at corners which is necessary with lap-joints.

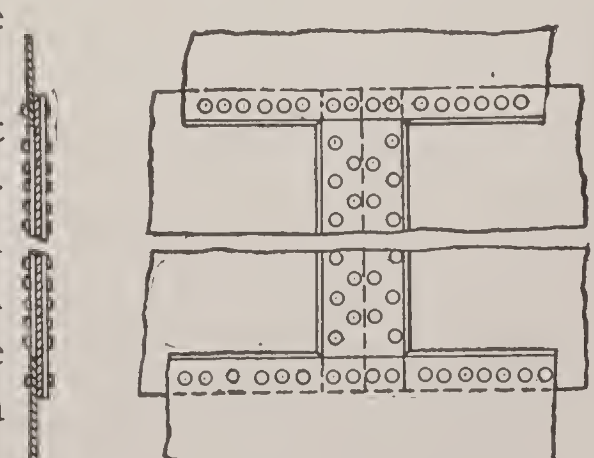

FIG. 205.

The maximum economy of riveting would be secured by selecting a diameter of rivet such that its shearing strength would equal its crushing strength, but in practice the diameter selected is usually somewhat less than this, in order to avoid too great a pitch and too large a rivet. For lap-joints the diameter is made equal to about twice the plate thickness, but not less than $\frac{5}{8}$ inch nor more than $1\frac{1}{8}$ or $1\frac{1}{4}$ inches. For double-butt joints the diameter need not be made so great. With the selected diameter, the pitch is determined by making the tensile strength on net section equal to the shearing value of the rivets, using a safe shearing strength of about three-fifths of that used for the tensile strength. The efficiency is then the ratio of safe stress on net section to safe stress on gross section.

Joints are single-, double-, or triple-riveted, depending upon the thickness of the plates and the economy desired. The efficiency of a joint increases with the number of rows of rivets used, but for any particular style of riveting the efficiency decreases somewhat as the thickness of the plates increases, on account of the limitations to the size of rivets. It is therefore of greater relative importance to use multiple riveting on thick plates than on thin ones. In stand-pipe construction it is usual to employ single riveting for the upper sections where the plates are not fully stressed; then double riveting up to a thickness of $\frac{7}{8}$ or 1 inch, and triple riveting for 1 inch and above. With a high cost of material it would be economical to employ triple riveting for thinner plates.

Table No. 95, from Johnson's "Framed Structures," gives suitable

TABLE NO. 95.

PROPORTIONS FOR RIVETED JOINTS FOR STAND-PIPES.

Kind of Joint on Vertical Seams.	Thickness of Plate.	Diameter of Rivet.	Pitch of Rivets. Centre to Centre.	Distance between Pitch-lines.	Distance of Pitch-line from Edge of Plate.	Percentage of Total Strength of Plate Developed.
	Inch.	Inches.	Inches.	Inches.	Inches	
Single-riveted lap	$\frac{1}{4}$	$\frac{5}{8}$	$1\frac{5}{8}$	......	1	50
" " "	$\frac{5}{16}$	$\frac{5}{8}$	$1\frac{3}{4}$	......	$1\frac{1}{8}$	
Double- " "	$\frac{5}{16}$	$\frac{5}{8}$	$2\frac{5}{8}$	$2\frac{1}{8}$	$1\frac{1}{8}$	
" " "	$\frac{3}{8}$	$\frac{3}{4}$	$2\frac{5}{8}$	$2\frac{1}{8}$	$1\frac{1}{4}$	60
" " "	$\frac{7}{16}$	$\frac{3}{4}$	$2\frac{3}{4}$	$2\frac{1}{4}$	$1\frac{3}{8}$	
" " "	$\frac{1}{2}$	$\frac{7}{8}$	$2\frac{7}{8}$	$2\frac{3}{8}$	$1\frac{1}{2}$	
" " butt	$\frac{9}{16}$	$\frac{7}{8}$	$2\frac{7}{8}$	$2\frac{3}{8}$	$1\frac{5}{8}$	
" " "	$\frac{5}{8}$	$\frac{7}{8}$	$2\frac{7}{8}$	$2\frac{3}{8}$	$1\frac{5}{8}$	
" " "	$\frac{11}{16}$	1	$2\frac{7}{8}$	$2\frac{3}{8}$	$1\frac{3}{4}$	
" " "	$\frac{3}{4}$	1	3	$2\frac{1}{2}$	$1\frac{3}{4}$	70
" " "	$\frac{13}{16}$	1	3	$2\frac{1}{2}$	$1\frac{7}{8}$	
" " "	$\frac{7}{8}$	$1\frac{1}{8}$	3	$2\frac{1}{2}$	2	
" " "	$\frac{15}{16}$	$1\frac{1}{8}$	3	$2\frac{1}{2}$	2	
" " "	1	$1\frac{1}{4}$	$3\frac{1}{8}$	$2\frac{5}{8}$	$2\frac{1}{8}$	
Triple- " "	1	$1\frac{1}{8}$	4	3	$2\frac{1}{8}$	75

proportions for riveted joints, together with their efficiencies, as compiled from the Watertown Arsenal reports.

Horizontal joints are made single-riveted lap-joints, with rivet spacing of about three diameters. The wind-stresses will not require consideration unless the pipe is extraordinarily tall and slim. They can in any case easily be considered by the use of eq. (4). All seams should be thoroughly calked with a round-nosed calking-tool, and any leaky seams which may exist when the pipe is filled should be recalked.

721. Bottom Details.—The bottom is made of plates riveted up with circular and radial joints, the former being made lap-joints and the latter butt-joints. The thickness need be only enough to permit of good calking and to be durable,—about $\frac{1}{2}$ inch. This bottom plate is preferably connected to the side plates by means of a heavy angle on the outside, or one on both outside and inside the tank. The riveting of the side plates to the bottom angle is referred to in the next article.

In erection, the bottom is riveted up and attached to the lower course of the side plates while supported a short distance above the foundation. The foundation is then prepared and the bottom carefully lowered thereon. To furnish an even bearing and to level up the foundation, a dry mixture of cement and sand is often used, in order to avoid any trouble from setting before the work is in place. Grout has also been used by forcing it through holes in the bottom while the latter is supported about an inch above the foundation. The holes are afterwards plugged up.

722. Foundation and Anchorage.—The foundation should be made monolithic and sufficiently broad to give such low pressures on the soil that there will be practically no settlement. Failures have occurred due to poor work in this respect. Wind-pressures should be carefully considered. Concrete is a very suitable material for foundation purposes.

Stand-pipes must be anchored to the foundation to prevent being overturned by the wind. Eq. (5), page 713, gives the tensile stress per lineal inch, circumferentially, at any point in the pipe due to wind. It is $S' = 1.33\frac{h^2}{d}$. The effect of the weight of the pipe in reducing this need not be considered. The stress on any anchor-bolt will then be $S'p$, where $p =$ distance in inches between bolts. If numerous bolts are used, their size will not be great, and they may be put through the exterior bottom angle and the latter double-riveted to the pipe. If arranged in this way, they should be numerous enough so that the stress in one bolt is not greater than can be transmitted to the lower plates by four or five rivets, which will limit the size of bolts to about $1\frac{3}{4}$ times the diameter of the lower rivets. By spacing the bolts sufficiently close this arrangement may be followed in almost any case. If this method gives a large number of bolts, it will be simpler to use fewer and larger bolts, in which case they should be fastened to the stand-pipe by long vertical pieces of angles, and the bolts placed close to the pipe as shown in Fig. 206. The number of bolts should not be less than six in any case. Anchor-bolts should extend well into the masonry and be fastened to anchor-plates embedded therein.

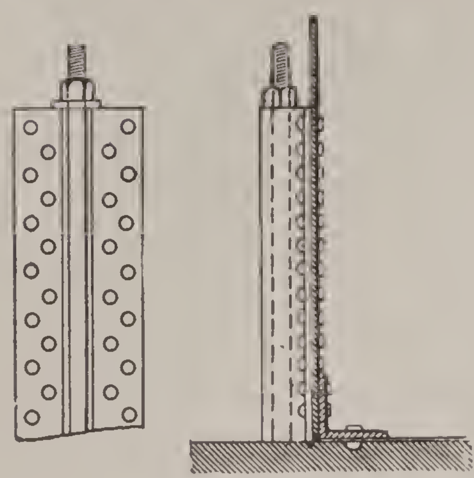

FIG. 206.

The method here given for determining the stress in anchor-bolts is not equivalent to the usual method of equating moments about the edge of the pipe, but gives larger values than that method. It is the same as would be used at any other horizontal joint of the pipe, or at any section of a beam, and it assumes that a tension will exist on the windward side before the resultant pressure reaches the outer edge of the joint—in fact as soon as it passes the edge of the "middle third," as is the usual assumption in all masonry designs.

With very high pipes, and on soft soils requiring broad foundations, it may be desirable to distribute the pressure by the use of large brackets of a triangular shape, riveted to the pipe, at the outer ends of which the anchor-bolts may be placed.* These bolts may be figured in the same way as explained above. For very slender tanks, stays or guys of wire are sometimes used. These should be very taut so as to prevent injurious deflections.

723. **Pipes and Valves.**—Usually a single pipe serves both as inlet and outlet. This passes through an arched opening in the foundation, turns upwards and enters the stand-pipe at the bottom, and extends into it a foot or two. A lead joint is usually made in a bell casting riveted to the bottom of the pipe as shown in Fig. 207. Another arrangement is shown in Fig. 208, which illustrates the details of the bottom of the West Arlington stand-pipe, Baltimore, Md., Mr. Nicholas J. Hill, chief engineer. The inlet- and overflow-pipes are of steel.

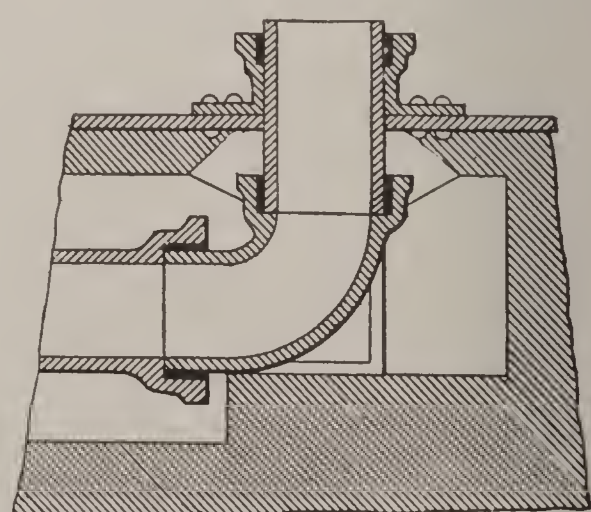

FIG. 207.—INLET-PIPE FOR STAND-PIPE.

* See Johnson's Framed Structures, p. 430.

They are riveted to steel flanged collars at the entrance to the pipe, and to similar collars bolted to the flanges on the cast-iron elbows which rest on the concrete.

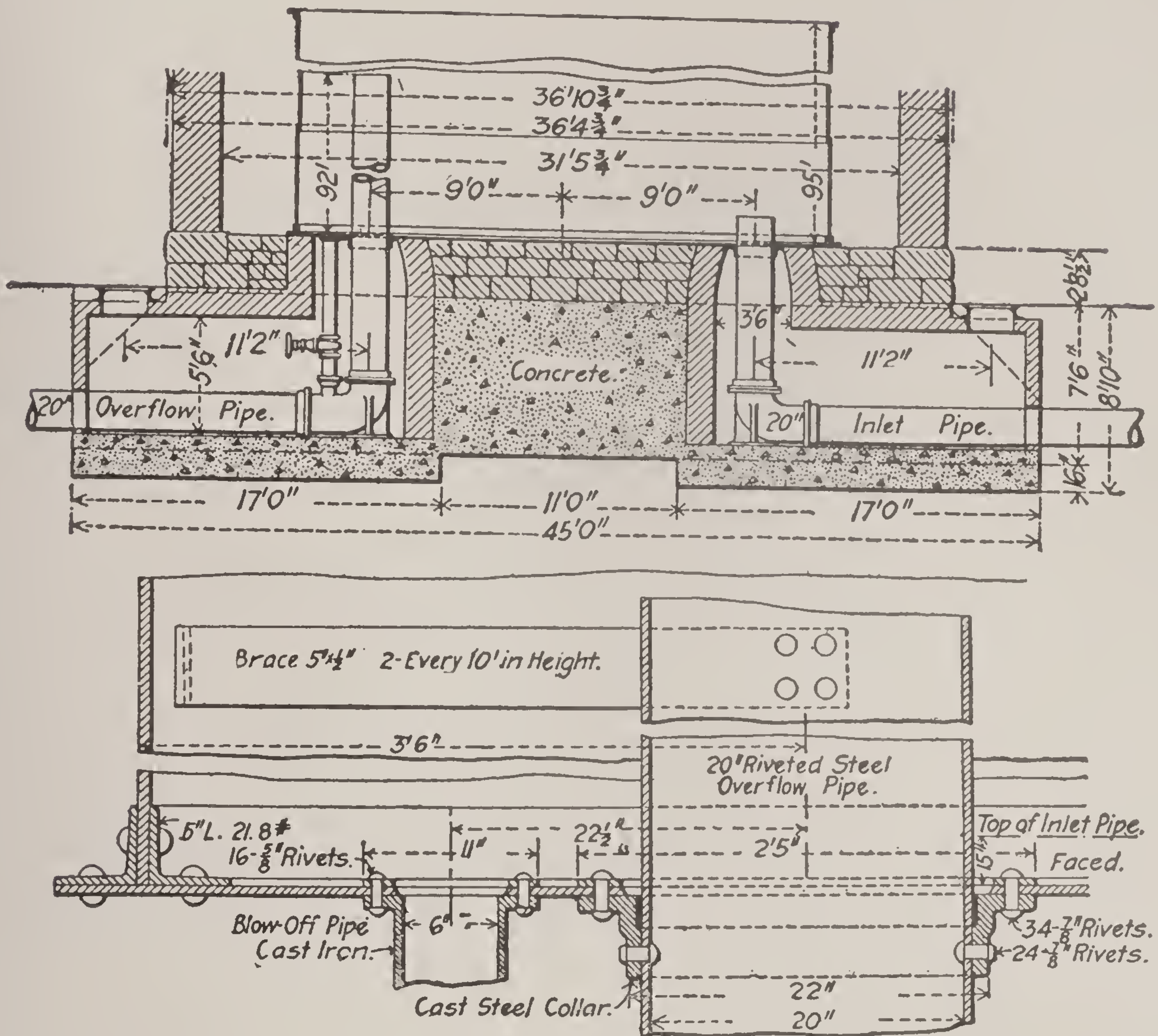

FIG. 208.—BOTTOM DETAILS, WEST ARLINGTON STAND-PIPE, BALTIMORE.

(From *Engineering Record*, Vol. XL.)

A drain-pipe through which the tank may be drained or flushed should be provided. Such is also shown in Fig. 208. Overflow-pipes are not usually provided for open stand-pipes. If used, they should be placed on the outside, the water reaching them from over a broad weir or through an orifice in the side of the tank. Valves for inlet- and drain-pipes should be placed outside the foundations.

Where the fire pressure must be furnished for the most part by direct pressure, some convenient method of shutting off the pipe must be employed, and right here is where the ordinary system is apt to be weak and out of repair. Several devices are in use for closing a valve by electrical means. A simple form of such device consists of an ordinary gate-valve, operated through suitable gearing by means of a weight attached to a drum which can be released electrically. The valve is opened by hand.* Another general form consists of a check-valve arranged to be operated by an hydraulic piston, the water for which is supplied from the force-main and controlled by a small valve operated electrically.†

Another simple form is where an ordinary hydraulic gate-valve is arranged to be operated by an electrical device. Other devices are employed which act automatically when the pressure or velocity of the water is increased. These are apt to cause heavy water-hammer, unless specially guarded against by the use of a balanced valve or by relief-valves.‡

Whatever the device used, unless the valve opens as well as closes automatically, a by-pass with check-valve should be provided to enable water to flow from the stand-pipe in case the pressure in the mains falls below that in the stand-pipe.

High-water electric alarms are advisable if the pipe be at some distance from the pumping-station. The pressure indicated at the station is not a certain guide if branch mains are led off at intermediate points. For encased pipes or tanks a simple float, arranged to close an electric circuit, may be used. For exposed pipes, ice is likely to interfere, and in this case a pressure-gauge placed in a vault and connected to the stand-pipe can be arranged to give an alarm at any desired pressure. For encased stand-pipes the balanced float-valve described on page 452 may be used to advantage to shut off the supply.

724. **Other Details.**—*Top Angle.*—The top should be stiffened against collapse by a heavy angle-iron, not less than 3 × 5 inches, and two such angles should be used for large pipes. The effect of the wind on an empty pipe is not only to cause a pressure on the outside, but to create a partial vacuum on the inside near the top. Several failures have occurred from lack of strength at this point.

Roof.—It is not customary to roof stand-pipes, and for a tall slim pipe a roof would be of little use and no improvement to its appearance. With large, low pipes a conical roof of curved profile may well be adopted. It affords considerable protection and improves the appearance of the structure. It is usually made of sheet iron or copper, supported on light angle-iron ribs or framework.

Ladder.—A ladder should be built on the outside of the pipe, but none on the inside; and in general there should be no obstructions on the inside where ice is likely to form to any extent.

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* *Eng. News*, 1889, XXII. p. 291.

† *Eng. Record*, 1894, XXIX. p. 339; 1900, XVII. p. 177.

‡ See description of automatic valves in *Eng. Record*, 1894, XXIX. p. 339; *Eng. News*, XXXI. pp. 12, 284.

Manhole.—A manhole is sometimes placed in the lower course of plates. If this is done, care should be taken to properly reinforce the cut plate. In Fig. 209 this is accomplished by an angle and a cast-steel frame.

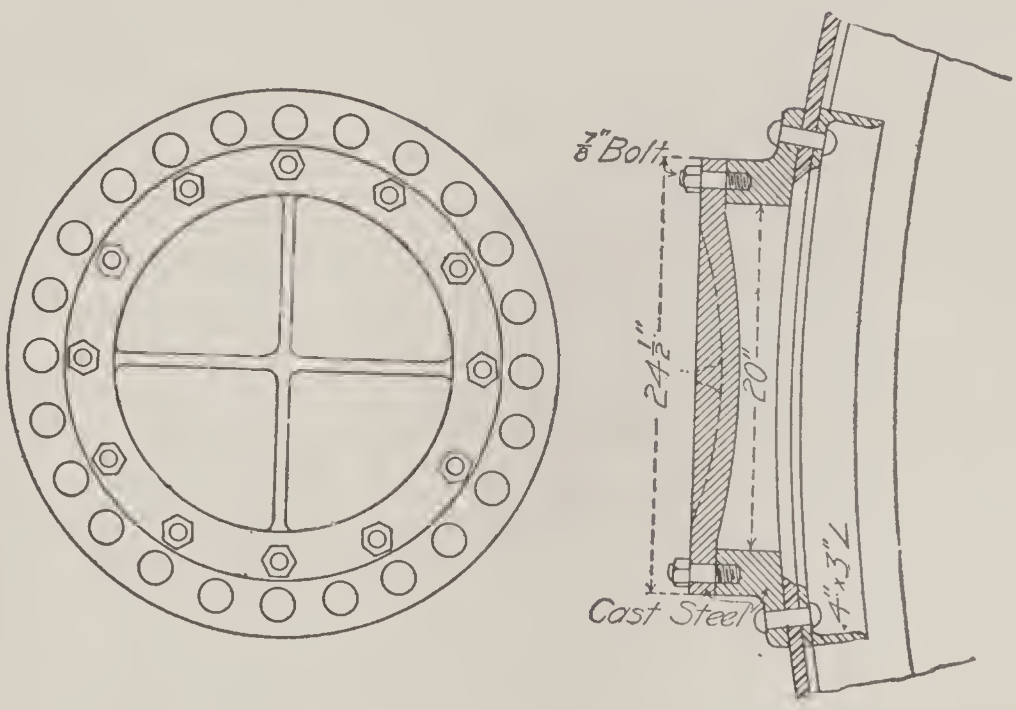


FIG. 209.—MANHOLE, WEST ARLINGTON STAND-PIPE.

Ornamentation.—Besides the use of a roof as noted above, the monotony of a low stand-pipe may sometimes be broken up by a winding staircase. For a very tall pipe little can be done, perhaps, to improve the appearance. The chief cause of the ugly appearance of such pipes is the lack of any apparent base. A massive masonry pedestal of a height proportioned to that of the tank, used in connection with a suitable cornice, would improve the appearance considerably.*

Painting.—Stand-pipes should be well painted inside and out. For the interior, asphalt is probably the best material to use. After painting the interior, the pipe should be filled to detect leaks before the outside is coated.

725. **Encased Stand-pipes.**—A stand-pipe is often surrounded with a masonry shell in order to furnish protection from cold, or to improve the appearance of the structure, or, in the case of slender pipes such as are used for pressure-regulators, to protect them from wind-pressure. The masonry shell may be of stone or brick, and is usually built enough larger than the pipe to permit of a stairway in the space between. For small towers the walls can be calculated as if the structure were a monolith, according to the principles applied to other masonry structures, the wind-pressure and the weight of masonry being

* See illustration in Johnson's Framed Structures, p. 433.

the forces considered. The resulting walls will vary considerably in thickness from top to bottom. They are usually made from 2½ to 4 feet thick at the bottom and 1½ to 2 feet at the top. With pipes of large diameters (25 to 40 feet) the results of analysis under the assumption of a monolithic structure would give walls too thin to be

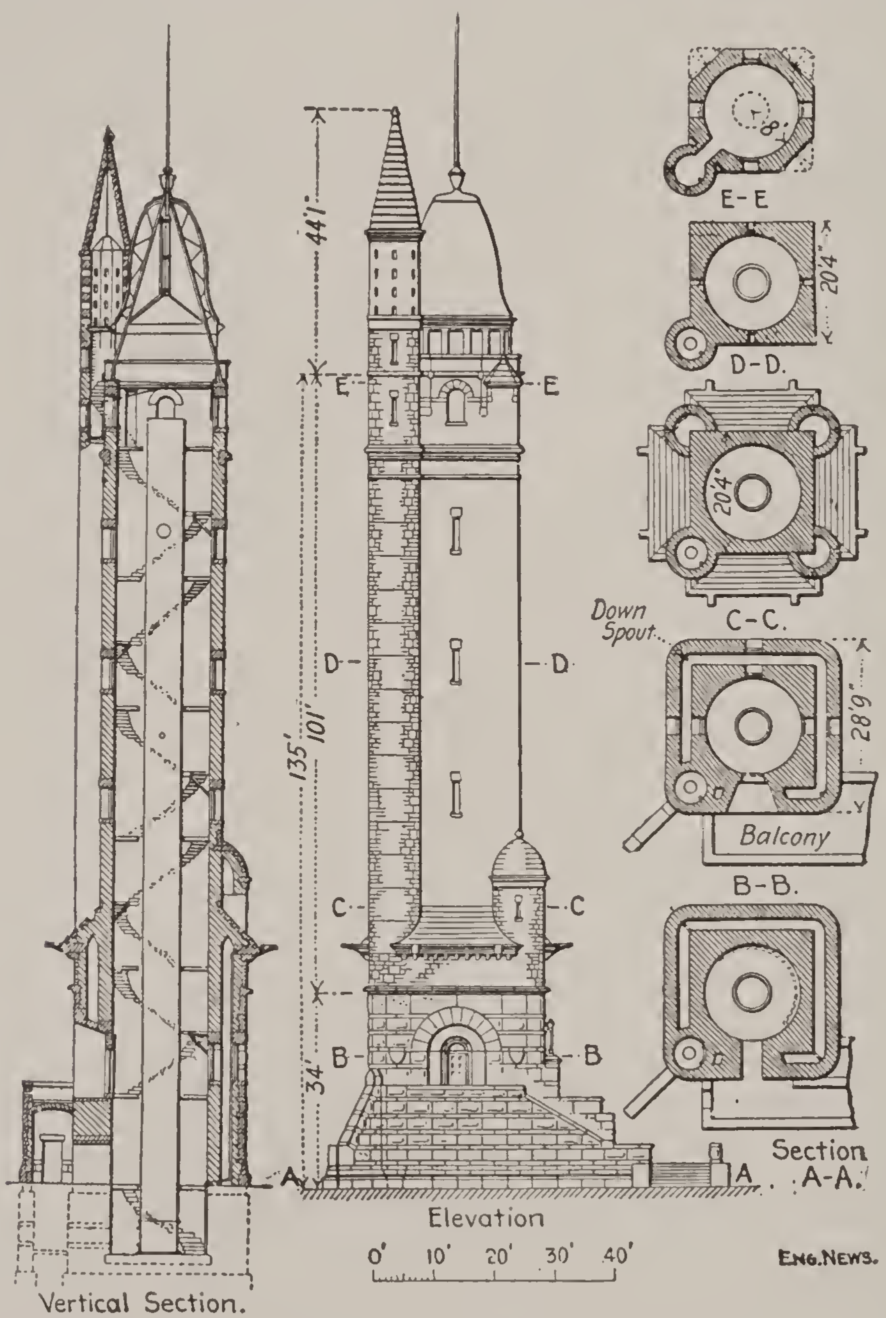


Fig. 210.—Compton Hill Water-tower, St. Louis.
(From *Engineering News*, vol. xxxix.)

stable locally, and it cannot well be assumed that such walls act as monoliths. Under such conditions it may be best to provide against wind-pressure by the tank anchorage, and then brace the walls against the tank, as was done at St. Charles, Mo. In this case, with a tank 25 × 70 feet, the walls were supported at six points by circular lattice girders 2 feet deep riveted to the tank. At the same time these girders served to strengthen the tank against buckling. The walls were from 9 to 13 inches thick.*

* *Jour. Assn. Eng. Soc.*, 1895, xiv. p. 533.

Encased pipes must be provided with overflows, which may be built either inside or outside the main pipe. For this type of structure, roofs are quite necessary, and should be carefully proportioned with respect to appearance. The masonry offers considerable opportunity for architectural treatment, and this feature should be referred to a competent architect.

A small encased stand-pipe built near a pumping-station is illustrated in Fig. 210. The stand-pipe is provided with a 2-foot overflow-pipe which is connected at two points with the main pipe. Either connection may be used, according to the pressure required. In the design of this structure the architectural features were of considerable importance, the tower being located in a prominent place. The base of the tower is of blue Bedford stone, the sub-tower of white limestone, and the main shaft of buff brick, trimmed with granite. The roof is of white tile. The tower is lighted by electricity.

Elevated Tanks.

726. **Economy of Elevated Tanks.** — If the lower portion of the water in a stand-pipe is at too low an elevation for useful pressure, its only office is to furnish support to the useful part above. Where this useless zone is of any considerable depth the support can be more cheaply furnished by a steel trestle. Assuming the safe compressive stress in the columns of such a trestle to be 10,000 pounds per square inch, the total cross-section of the columns necessary to support a tank above any given plane will be about one-half that of a stand-pipe at the same elevation. The thickness of a stand-pipe would also increase rapidly from this point down, while the column sections would increase but slightly. The economy of the trestle form is therefore very evident where the distance to the useful elevation is considerable. The cost of piping, trestle-bracing, etc., would add to the expense of the tank, but the foundation for a tank is less expensive than that for a stand-pipe. Besides being cheaper, a tank is much less objectionable in appearance than a stand-pipe, and experience indicates that trouble from ice is less likely to occur.

727. **Form and Proportions.** — For roofed tanks a height equal to the diameter would not be far from the most economical proportions, but a height somewhat greater than this will usually look better.

Formerly the bottoms of tanks were made horizontal and supported on a system of beams, but later designs use a conical or a spherical bottom supported at the periphery only, which is a better and much cheaper arrangement. The spherical form is the best, and involves no special difficulty in construction. A hemispherical bottom gives lower stresses to be provided for than the segmental form, but rather more complex details at the supports, so that the latter may be preferred, especially where the tank rests upon masonry walls. The hemispherical form has, however, been adopted as the standard by at least one large construction firm.

728. Stresses in Tank.—The thickness of side plates is the same as for stand-pipes, and the details are similar. If the bottom is spherical, the tension per lineal inch will be one-half that in a cylinder of the same radius and with the same internal pressure, or by eq. (1), page 713, will equal

$$S = 2.6hr, \qquad (10)$$

in which r = radius of bottom, and h = head of water in feet. For a hemispherical bottom, $r = \frac{d}{2}$, and hence the thickness of plates would be equal to one-half that of the lowest side course (assuming same efficiency of joint), but should not be less than $\frac{5}{16}$ or $\frac{3}{8}$ inch.

To analyze the stresses in a conical bottom it will be convenient to consider the tensile stresses along an element of the cone, and those at right angles thereto, separately.

Fig. 211 shows the portion of a conical bottom below any section

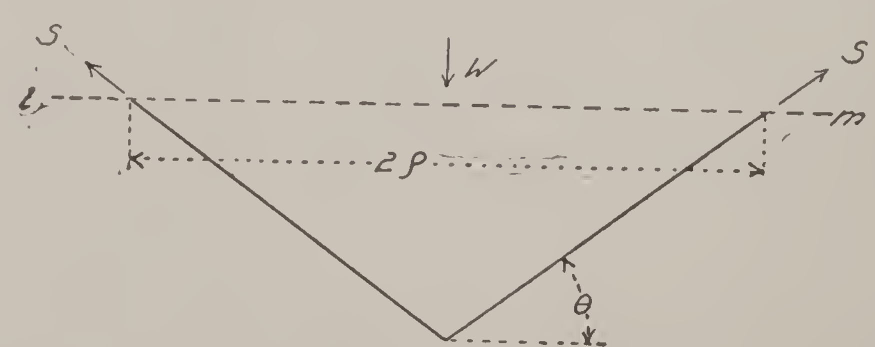


FIG. 211.

lm. W is the total weight of water directly above the section, together with the small weight of water and tank below, and S is the tensile stress per lineal inch. Assuming no bending stresses to exist, we have

$$S \sin \theta \times 12 \times 2\pi\rho = W,$$

whence

$$S = .0132 \frac{W}{\rho \sin \theta}, \qquad (11)$$

in which ρ is in feet. If h = average head of water on this portion of the bottom, we have (neglecting the weight of tank) $W = 62.5 h\pi\rho^2$, whence

$$S = 2.6\frac{\rho h}{\sin\theta}. \qquad (12)$$

At the edge of the tank S is a maximum and is equal to $2.6\frac{rh}{\sin\theta}$. For $\theta = 30°$ this would be the same as the stress in the lowest side plate (eq. (1), page 713).

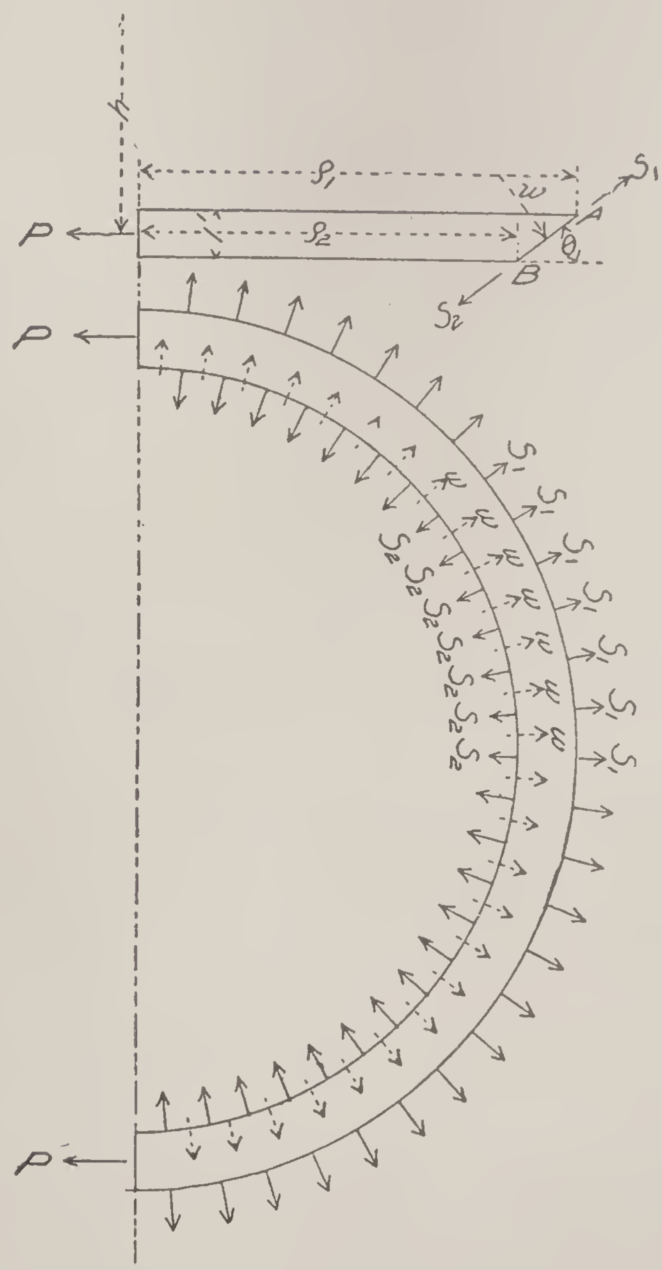


FIG. 212.

The tensile stress in a circumferential direction will now be determined. Fig. 212 shows one-half of a horizontal slice of the bottom, one foot in vertical dimension. S_1 and S_2 are the stresses per lineal unit, acting in the directions indicated, and P is the stress to be determined. The length $AB = \frac{1}{\sin\theta}$, and is the same as the length over which P acts. The average head is h, the average radius is ρ, and the water-pressure per lineal unit is w. Equating horizontal components, we have, by a summation similar to that performed in getting the bursting stress in a pipe,

$$P = S_1 \cos \theta\rho_1 - S_2 \cos \theta\rho_2 + w \sin \theta\rho. \quad . \quad . \quad . \quad (13)$$

Equating vertical components, we have

$$S_1 \sin \theta\pi\rho_1 - S_2 \sin \theta\pi\rho_2 = w \cos \theta\pi\rho; \quad . \quad . \quad . \quad (14)$$

whence we may write

$$S_1 \cos \theta\rho_1 - S_2 \cos \theta\rho_2 = w\rho \frac{\cos^2 \theta}{\sin \theta}. \quad . \quad . \quad . \quad . \quad (15)$$

Furthermore, $w = \dfrac{62.5h}{\sin \theta}$.

Substituting in eq. (13) from eqs. (14) and (15), we have

$$P = w\rho \left(\frac{\cos^2 \theta}{\sin \theta} + \sin \theta\right) = 62.5\rho h \left(\frac{\cos^2 \theta}{\sin^2 \theta} + 1\right). \quad . \quad . \quad (16)$$

The stress per lineal foot will be $P \sin \theta$, or, expressed in pounds per lineal inch, it is

$$S' = \frac{P \sin \theta}{12} = 5.2 \frac{\rho h}{\sin \theta}. \quad . \quad . \quad . \quad . \quad . \quad (17)$$

This is just twice the stress given by eq. (12), and is greater than the stress in the lower side plates in the ratio of $1 : \sin \theta$. To avoid too thick bottom plates, therefore, θ should not be made small.

729. *Connection between Side and Bottom Plates.*—With a conical or segmental bottom the inclined pull per inch at the line of connection with the side plates will be given by eq. (11). It is

$$S = .0132 \frac{W}{r \sin \theta},$$

where W = total weight of water and of bottom, r = radius of tank, and θ = angle of inclination with the horizontal of cone element, or of tangent to circular segment at outer edge. The bottom and sides are connected by means of a circular angle or shape iron, which resists the horizontal component of the force S, by compression as a ring. The compressive stress in this ring will be

$$P' = 12S \cos \theta r = .159W \cot \theta, \quad . \quad . \quad . \quad . \quad . \quad (18)$$

or approximately

$$P' = 31.2hr^2 \cot \theta, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (19)$$

where h = average depth of water. This is a very considerable stress and must be provided for by a sufficient amount of metal, but the metal of the side plates and of the bottom plates can be counted on for 5 or 6 inches from the angle. The hemispherical bottom causes no stress of this kind and is in this way preferable.

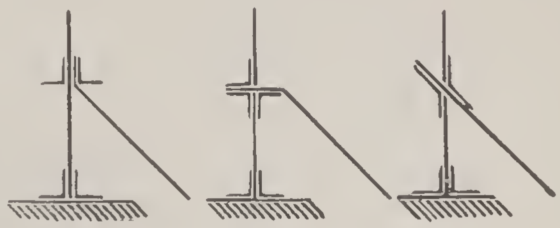


FIG. 213.

Fig. 213 illustrates three simple arrangements of this detail. The bent bottom plate should in the first two cases be supported close to the line of the bend. For other details see Figs. 214 and 217.

730. The Tower.—The tower consists of a steel trestle of four to eight legs. The material for this may be medium steel, and comparatively high working stresses may be used in its design, since the stresses are all dead- and wind-load stresses. Four legs are the smallest practicable number, but for tanks of large diameters the use of only four legs brings very heavy local stresses on the tank at the points of connection. Six or eight is a better number and presents a better appearance, but is more expensive. A design in which four posts are used and branched near the top was employed by Johnson & Flad at Laredo, Texas, and again by Mr. Flad at Murphysboro, Ill. The latter design is illustrated in Figs. 216 and 217. This arrangement gives twelve points of support without the use of an expensive tower. The tank is sheathed with wood to prevent the formation of ice.

The columns of the tower may be of channels, Z bars, or any convenient form of section. They are supported at intervals of 20 to 30 feet by lateral bracing, which also forms the wind-bracing. This bracing usually consists of horizontal struts and diagonal tie-rods. For eight or more legs radial struts should also be used to give rigidity to the tower. In high towers the columns should preferably have a broken outline for the sake of appearance, as in Fig. 215, which illustrates the large tank at the Iowa Agricultural College at Ames, Iowa, Prof. A. Marston, engineer. This was the first tank in which this feature was carried out. The details of this tank are shown in Fig. 214.

731. *Stresses in Tower.*—The stresses due to the vertical load are readily calculated, and for the four-post tower those due to wind also. In the six- or eight-post tower the wind-stresses are not so readily determined. The following method was first suggested by Prof. Marston.*

* *Eng. News*, 1898, XXXIX. p. 371.

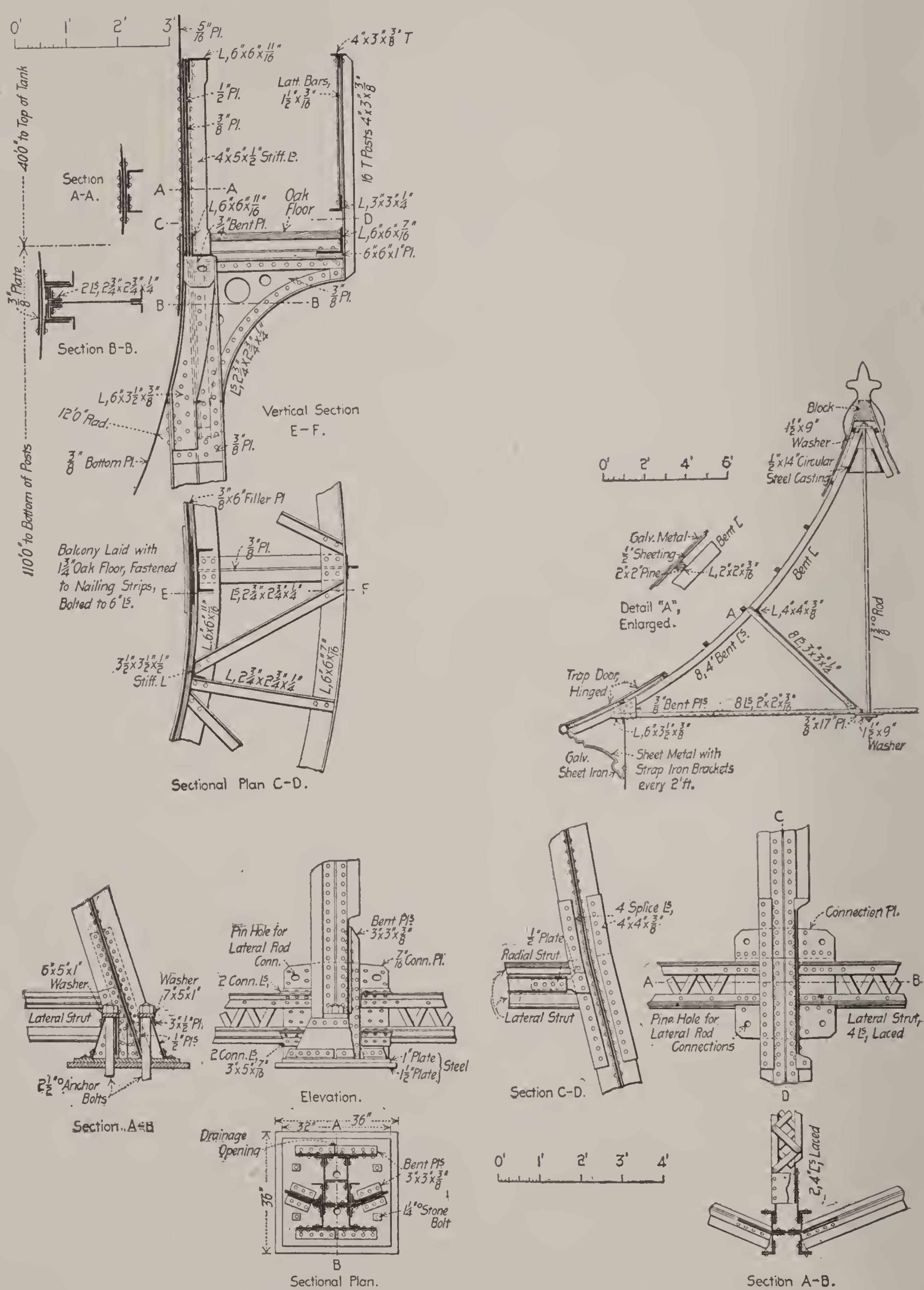
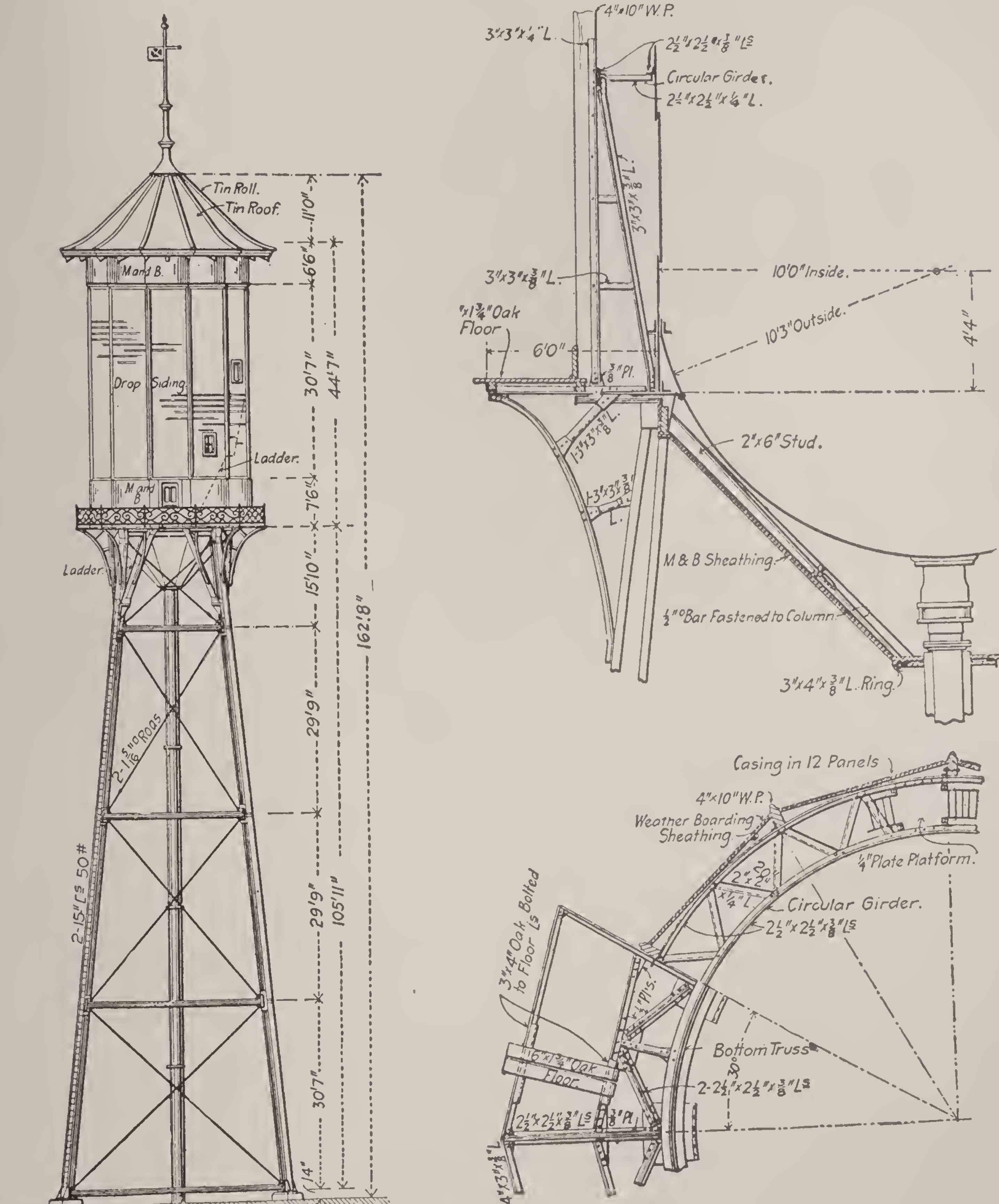


FIG. 214.—DETAILS OF ELEVATED TANK AT AMES, IOWA.
(From *Engineering News*, vol. XXXIX.)

FIG. 215.—ELEVATED TANK, IOWA AGRICULTURAL COLLEGE, AMES, IOWA.

The amount of wind-pressure on the tank may be assumed the same as given for stand-pipes. On the tower a pressure of 50 pounds per square foot of all exposed areas may be assumed. As regards wind-stresses the tower may be considered as a vertical cantilever



Elevation. Tank details.

FIG. 216.—ELEVATED TANK, MURPHYSBORO, ILL.
(From *Engineering Record*, vol. XLII.)

beam anchored to the ground. Then if we pass a horizontal section at the top of each story, cutting the posts only (between points of attachment of diagonal bracing), we can get the vertical components of post stresses as in a beam made up of parts. Thus in Fig. 218, representing such a section, let A = section of each post, and r = radius of tower. The maximum stress in post a will occur when the wind blows at right angles to axis lm. If M is the wind moment about the horizontal plane assumed, the fibre-stress in posts a will be

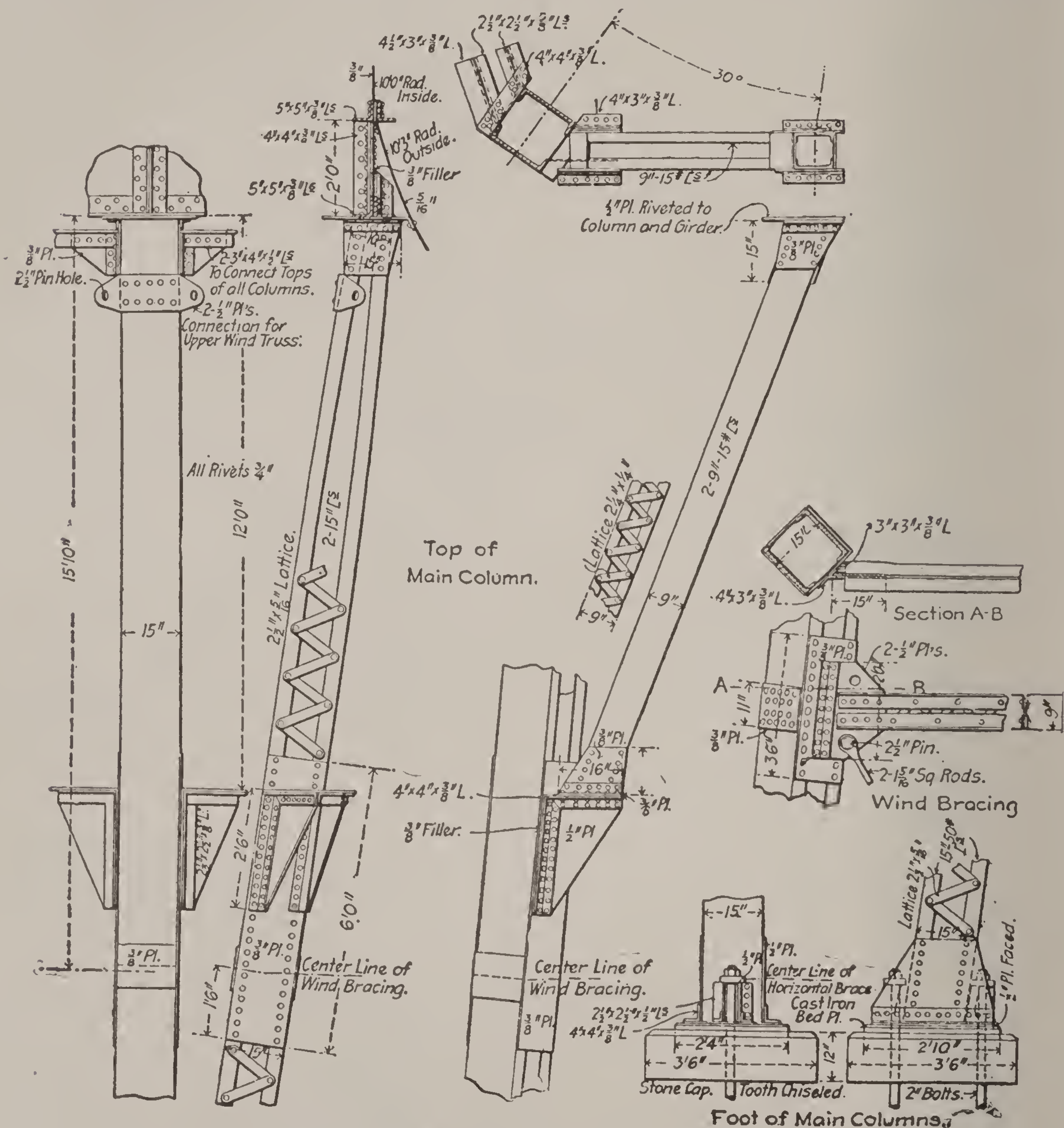


FIG. 217.—DETAILS OF ELEVATED TANK, MURPHYSBORO, ILL.
(From *Engineering Record*, vol. XLII.)

$f = \dfrac{Mr}{I}$, where I = moment of inertia of the entire tower about lm, = $4Ar^2$, if we neglect the moment of inertia of each column about its own axis. Hence $f = \dfrac{M}{4Ar}$, or the total column stress =

$$P = fA = \frac{M}{4r} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (20)$$

The stress on columns $b = .7\dfrac{M}{4r}$, and on columns c, = 0.

In a six-post tower $I = 3Ar^2$, and the stress in the most remote post is $\frac{M}{3r}$, and on each of the others is $\frac{1}{2}\frac{M}{3r}$.

By this method the vertical components acting at the top and bottom of each story at each post can be found. Then taking each story separately, the stresses in the diagonal rods can be found by equating vertical components acting at top and bottom of each post, beginning with the post *a* where the stresses on the two diagonals attached thereto are equal. The actual post stresses are then found by equating vertical components at either top or bottom joint, and finally the stresses in the lateral struts are obtained by the use of two equations of the components in a horizontal plane acting at a joint.

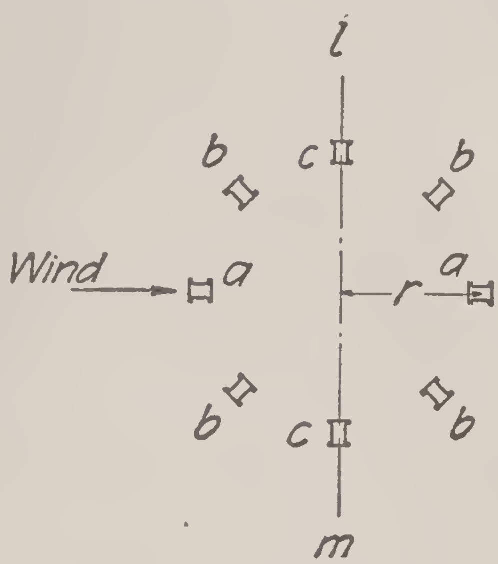


FIG. 218.

The wind-stresses should be combined with maximum dead-load stresses to get the maximum post compression, and with minimum dead-load stresses to get the tension on the windward post and the pull on the anchorage.

732. Connection of Tower and Tank. — With conical or segmental bottoms the lower side sheets are usually extended below the bottom and finished with two angles as flanges, which rest on the tops of the columns, as in Fig. 213. With hemispherical bottoms the extension of the lower sheets is unnecessary, as a central connection can readily be made to the side and bottom plates as shown in Fig. 214. Ample stiffness should be provided, and sufficient reinforcement to enable the column load to be safely distributed into the side plates. With but four posts the lower course of side plates should be thickened. Lateral stiffness is secured by riveting to the tank, at the level of the post connection, a circular plate or lattice girder supported on brackets, and which may at the same time serve as a floor or support for a balcony.

733. Anchorage. — Each column must be well anchored to the foundation, with a strength of anchorage equal to the maximum uplift due to wind acting on empty tank. The amount of this uplift is computed as explained above. The foundation should be rigid, and large and heavy enough to serve as anchorage and to give only safe pressure on the ground. There should be practically no settlement, as any unequal settlement will greatly change the stresses in the tower.

734. Inlet-pipe. — The inlet-pipe is usually made to enter the tank at the center of the bottom, and should be provided with an expansion-joint. This may consist of a brass-lined stuffing-box and gland or a joint similar to that shown in Fig. 165, *b*, page 611, may be used to advantage. In cold climates the pipe must be protected by a frost-casing, which is usually a simple wooden box with one or more air-spaces and perhaps a packing of some non-conductive material. If the tank is encased, it will be necessary to provide an overflow-pipe.

735. **Masonry Towers.**—It is the common practice in Europe to support the tank on a masonry structure, and also to enclose it with masonry or wood. This form of construction readily lends itself to effective architectural treatment and should be more often adopted in this country. The bottom details in this case are arranged as shown in Fig. 213, the tank resting upon the wall. The masonry or wood casing above must then be bracketed out. To economize in masonry a method of construction devised by Engineer Intze has frequently been employed in Germany. This is illustrated in Fig. 219, which shows a section of the tower at Kreuzberg, Berlin.* The tank has a capacity of about 100,000 gallons.

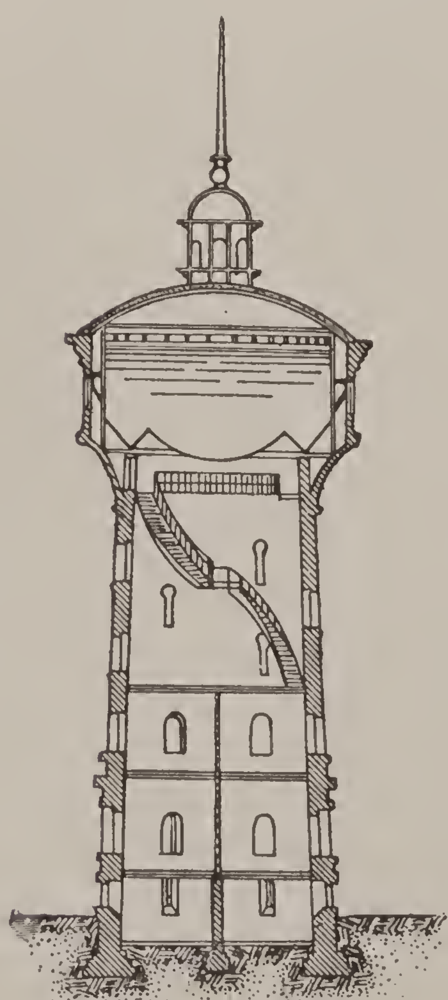


FIG. 219.—INTZE TOWER AT KREUZBERG, BERLIN.

736. **Wooden Tanks.**—Elevated tanks of wood are frequently used where low first cost is an essential element and the quantity to be stored does not exceed 50,000 to 75,000 gallons. Wooden tanks are cheap, and if well built will last fifteen or twenty years. The staves should be of good clear material and should be dressed to proper curvature on the outside. Hoops should be relatively thick to resist corrosion, and should be thoroughly coated with asphalt or other protective coating, before being put in place. Lugs and fastenings are a source of weakness. They should be carefully designed and of ample strength. The support of the floors must also be well looked after. The chief source of trouble with wooden tanks is in the weakening of the hoops by rusting from the inside. Galvanizing is now being tried as a preventive, and may prove more successful than coatings formerly

* *Zeit. d. Ver. deutsch. Ing.*, 1886, p. 28.

used. Several failures of wooden tanks have occurred by the sudden bursting of the hoops, and it is questionable policy to construct such tanks where their failure is likely to endanger life, as it is quite certain that they will not be regularly inspected as they should be.

736a. Tanks of Reinforced Concrete. — Reinforced concrete has been used to a limited extent in the construction of stand-pipes and tanks. In first cost they compare favorably with steel structures and in durability are superior. The design as to strength is simple, steel being used to supply the entire tensile strength required. Certain practical difficulties of construction arise, however, which have not been overcome with a sufficient degree of certainty to lead to the general adoption of this type of structure. These relate to the securing of complete imperviousness and a satisfactory external appearance. It is difficult to make the body concrete impervious owing to the effect of temperature changes and distortions due to tensile stresses. Usually, therefore, imperviousness is secured by means of some kind of waterproof coating. It is probable that a reliable and satisfactory method of construction will soon be developed which will lead to the general use of reinforced concrete for these structures. Figs. 219a and 219b illustrate a design of tanks built at Havana, Cuba, which are very satisfactory in appearance. The details are clearly shown in Fig. 219b. Imperviousness was secured by the use of a well-mixed wet concrete.*

737. Storage of Water under Pressure. — In direct-pressure systems, some elasticity is to be desired to lessen the shock on the pumps and mains due to sudden variations in the draught. The small stand-pipe used for regulating the pressure has already been described. Another means of furnishing a small amount of elasticity is by means of large air-chambers placed on the mains near the pumps. Such have been used in a number of cases.† The air can readily be supplied when required by means of a small auxiliary chamber placed below the main chamber and so connected that air can be admitted to it under no pressure; then by closing the inlet and opening the connection to the main tank the air may be forced into the latter by the water-pressure from the force-main.

In small works, air-chambers or their equivalent may also be used to provide a considerable storage of water and thus avoid the use of stand-pipes or elevated tanks. In the design of such storage-tanks the larger the proportion of air-space the less will be the variation in

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* *Eng. News*, 1908, LIX. p. 471.

† See *Eng. Record*, 1893, XXVII. p. 196; 1893, XXVIII. p. 155; 1899, XL. p. 55.

water-pressure as the tank is emptied. If $V =$ volume of tank, and $v =$ maximum volume of water stored, then $V - v =$ minimum volume of air. If the pressure, when containing the maximum volume of water, be P, then when the tank is just empty the pressure is $p = P\left(1 - \frac{v}{V}\right)$.

FIG. 219a. WATER TANK, HAVANA, CUBA.
(From *Engineering News*, vol. LIX.)

Thus if $\frac{v}{V} = \frac{1}{3}$, then $p - \frac{2}{3}P$, and the variation in pressure is one-third the maximum. The less the desired variation in pressure the greater must be the tank capacity for a given water capacity. The air can be maintained in the tank by the same method as previously explained.

A system of pressure-storage having several advantages over that just described is the Acme Company's system, based on patents of

Wm. E. Wortham and Oscar Darling. In this system the air is stored in a separate tank at a higher pressure than is ordinarily kept on the

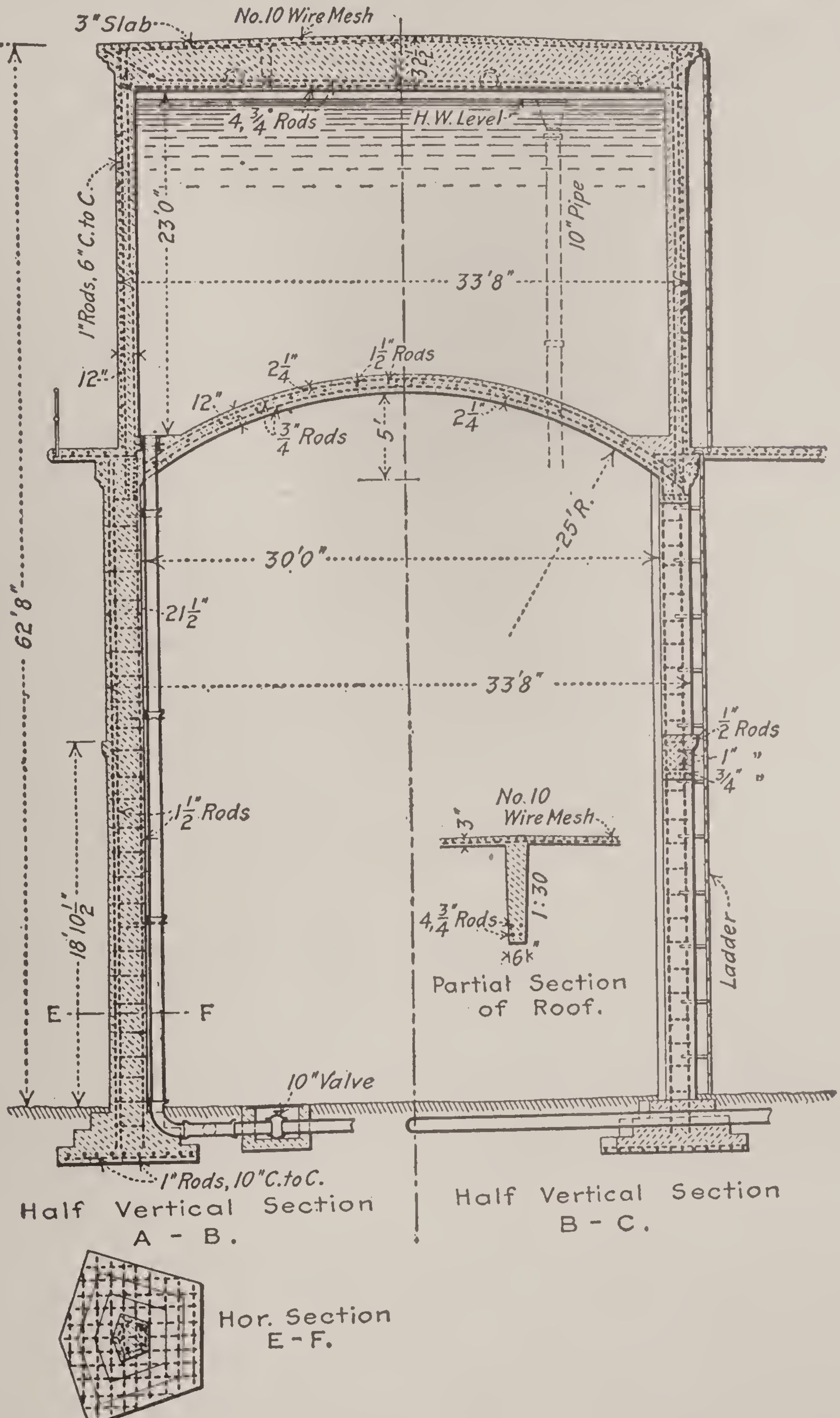

FIG. 219b. WATER TANK, HAVANA, CUBA.
(From *Engineering News*, vol. LIX.)

water. By reducing-valves in the connecting pipes, the pressure on the water may be maintained constant, or may be increased in case of fire. Air-compressors must be used here to keep up the air-supply.

A number of plants of this kind have been installed. (See references 10, 15, 18, page 741.) The use of a pressure storage system avoids all trouble from ice, and for very small quantities is cheaper than an elevated tank. A storage-tank can also be located at the pumping-station and the pressure easily controlled. For large quantities the system would be very expensive.

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CHAPTER XXVIII.

THE DISTRIBUTING SYSTEM.

738. **General Requirements.**—A distributing system should be so designed that it will be able to supply adequate quantities of water to all consumers, and that this will be accomplished with economy and with reasonable security against interruption. With respect to the design of this part of a water-works system, the uses of water naturally fall into two very distinct classes: (1) the ordinary, every-day use for domestic, commercial, and public purposes; and (2) the use for fire extinguishment. In the former case the consumption is relatively uniform over different portions of the city, and is also well distributed over many hours of the day; in the latter case the rate is likely to be extremely high for a very short period of time, but this excessive use of water will usually be confined to a comparatively small area. To supply water in the former case requires the wide distribution of moderate quantities, while in the latter case the problem is rather the concentration of large volumes within a narrow district, which district may be situated at any point in the system.

The cost of distributing-mains is usually the largest item in the cost of a water-works, and consequently much care must be taken in the design of this part of the system. In small towns it will often be impracticable to provide as large mains as would be required to furnish entirely satisfactory fire protection, and in such a case the advantages of improved fire protection must be carefully balanced against increased cost of large pipes.

To supply water to all consumers requires that a pipe be laid in each street, except in those cases where the cross-streets are not built upon. In the outlying districts, pipes are laid in those streets where the density of the population warrants it, according to the judgment of the management, but much difference in policy exists in respect to the matter of extensions.

The distributing system includes, besides the pipes, the fire-hydrants, service connections, valves, fountains, watering-troughs, meters, and occasionally other details.

739. **The Pressure Required.**—*a. Ordinary Service.*—For ordinary service the pressure at any point should be sufficient to supply water at a reasonable rate in the upper stories of houses and factories, and in business blocks of ordinary height. This will require at the street-level a pressure of from 25 to 35 pounds in residence districts, and usually from 30 to 45 pounds in business districts, according to the character of the buildings.

b. Fire Service.—For fire purposes the pressure required in the mains depends upon whether it is intended that fire-streams shall be furnished directly from the hydrants or whether steam fire-engines are to be used. In small cities and towns it is of the greatest advantage to supply fire-streams without the use of engines, and in most such places this method is adopted, fire-engines being sometimes kept in reserve, however, for extraordinary conflagrations. In pumping systems the most common arrangement is to maintain only a moderate pressure for ordinary service, and at times of fires to shut off the reservoir or stand-pipe if there be one, and to furnish the necessary fire-pressure direct from the pumps. In many plants, however, a good fire-pressure is maintained at all times, and this may be done without great expense if those buildings which demand the heaviest pressures are situated on the lower ground, and only scattered residences on the higher ground. Considerable economy is, however, usually secured by pumping against a low pressure except at times of fires.

In large cities hydrant fire-pressure is not so common, but if the supply is by gravity, and has plenty of head, a hydrant fire-pressure can profitably be furnished, at least for all except the densest portion of the city or for very large fires. If the water requires to be pumped, then only the ordinary working-pressure of 30 to 45 pounds is usually provided, and dependence is placed on fire-engines to supply the deficiency. To furnish fire-pressure direct from the pumps at all times would, in the case of large cities, be very costly, and to increase the pressure at times of fires would be impracticable.

If hydrant fire-pressure is to be supplied, it may be said that in general the pressures in the mains should be such and the hydrants so spaced that a large proportion of the fire-streams required in a business district should be of 240 to 250 gallons capacity, and in a residence district of 175 to 200 gallons capacity. If low hydrant pressures are furnished, the hydrants must be spaced closely, and if high pressures, then a wider spacing may be used. It will, however, be found that a hydrant pressure lower than 60 pounds for residence districts and 70 pounds for business districts will be undesirable, and that even these values will call for a close hydrant-spacing. Such pressures are, however, quite common. Much more preferable is a pressure of 80 to 100 pounds, as this gives good streams with a reasonable hydrant-spacing. The lower limits of 60 to 70 pounds may then be accepted where a higher pressure cannot be furnished without a large extra expense, as in the case of many gravity systems; but where the pressure is furnished by pumps, and especially where high pressure is furnished only during fires, the expense of additional head is not great and the higher values of 80 to 100 pounds should be adopted. If steam fire-engines are used, then it is only necessary to supply water to the hydrants without risk of causing the fire-pumps to operate under suction. This low pressure may result in interrupting the supply for other purposes at times of large fires, but this would not be a serious matter.

The pressures here considered are the hydrant pressures at times of maximum consumption, and refer to any point in the distributing system. If such pressures are maintained at the most remote points and at the higher elevations, the pressures on the lower ground and at points nearer the pumps or reservoir will of course be considerably higher. There will in general be certain critical points which will determine the pressures to be maintained at the source, and it will be a matter of economy to assume as low pressures as practicable for such points.

The maximum pressures allowable in a pipe system is a question of expediency, in which increased cost of heavy piping and increased danger of breakage must be offset against any advantage derived from high pressures. With a pumping system this question would hardly arise, but with a gravity system supplied from an elevated source, the head available may be greater than is desired, either for the whole or a part of the area served, in which case some method of reducing the pressure for certain districts may be used (Art. 648). As will be seen from Table No. 96, the maximum fire-pressures in common use range from about 100 to 160 pounds, this referring to the pressures at the pumping-stations. Generally speaking, pressures exceeding about 130 pounds are found to give much extra trouble in breakages, and this may be taken as about the limit which it will not often be desirable to exceed for any considerable part of the distributing system. If the elevations of different portions of the town vary widely, then two or more zones of pressure may be used (Art. 751).

In Table No. 96 are given the ordinary and fire pressures in the water-works of the United States, as stated in the Manual of American Water-works, 1888, and which serve to illustrate the practice in this respect. The pressures given refer usually to the pressures at the pumping-station. There has been a marked tendency during recent years towards higher pressures. (See Art. 757.)

TABLE NO. 96.

AVERAGE WORKING- AND FIRE-PRESSURES IN 1327 WATER-WORKS OF THE UNITED STATES.

The table gives the number of works having the pressures indicated in the first column.

Pressure per Square Inch, Pounds.	Average Working-pressure.	Fire-pressure.	Excess of Fire- over Working-pressure.
Under 20	16	....	59
20–29	60	....	57
30–39	122	....	67
40–49	270	9	106
50–59	159	11	81
60–69	204	27	93
70–79	158	36	68
80–89	126	73	39
90–99	75	70	21
100–109	47	143	21
110–119	28	32	8
120–129	27	99	11
130–139	5	30	5
140–149	5	23	2
150–159	14	54	2
160–169	4	13	2
170–179	7	11	
180–189	....	3	
190–199	....	3	
200	....	11	

TABLE NO. 97.

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

Population of Community.	Number of Fire-streams Required Simultaneously.			
	Freeman.	Fanning.	Shedd.	Kuichling.
1,000....	2 to 3	....	....	3
4,000....	....	7	....	6
5,000....	4 to 8	....	5	6
10,000....	6 to 12	10	7	9
20,000....	8 to 15	....	10	12
40,000....	12 to 18	....	14	18
50,000....	....	14	....	20
60,000....	15 to 22	....	17	22
100,000....	20 to 30	18	22	28
150,000....	....	25	....	34
180,000....	....	....	30	38
200,000....	30 to 50	....	....	40
250,000....	....	....	....	44
300,000....	....	....	....	48

740. Number and Size of Fire-streams.—The number of fire-streams which should be simultaneously available in any given town will obviously vary greatly with the character of the buildings, width of streets, etc. This subject, together with other questions relating to fire-protection, has been thoroughly discussed in valuable papers by Mr. Freeman * and Mr. Fanning,† to which reference should be made for more detailed information. The general conclusions of these engineers as to the number of streams required, and similar estimates by Mr. Shedd and by Mr. Kuichling, are given in Table No. 97.‡ The values given by Mr. Kuichling may be expressed by the formula

$$y = 2.8 \sqrt{x},$$

where y = number of streams, and x = population in thousands.

The figures given in the table relate to cities of average character, and are the total number of streams required simultaneously for the entire city. In regard to the actual number required at any one point Mr. Freeman estimates that as a general statement two-thirds of his estimated number should be capable of being "concentrated upon any one square in the compact valuable part of the city or upon any one extremely large building of special hazard." For compact residence districts one-fourth to one-half the number given in the table would usually be sufficient, and for small detached dwellings two to three good streams would answer. All these estimates should, however, be used with much caution, and should be varied to suit local conditions. Different large cities are likely to be of about the same general character and the requirements will be similar, but in small cities and towns the requirements for fire-protection may differ widely. For example, in a country town of 4000 to 5000 inhabitants in which only a small mercantile business is carried on, the fire risk is not great, while in a town of the same size whose prosperity depends entirely upon two or three large factories, located, perhaps, in one large group of buildings, a fire would be a very serious matter. In the former case four or five fire-streams would be sufficient, while in the latter case eight or ten should be supplied.

The number of fire-streams given in the table is based upon a size of stream of about 250 gallons per minute, which is generally considered to be about right as an average value for good fire-streams in

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* *Jour. New Eng. W. W. Assn.*, 1892, VII. p. 49.

† Proc. Am. W. W. Assn., 1892, p. 88; *Eng. News*, 1892, July, p. 42.

‡ From a paper by E. Kuichling in Trans. Am. Soc. C. E., 1897, XXXVIII. p. 15.

business districts. For a residence district 175- to 200-gallon streams will usually meet the requirement.

741. Location of Hydrants.—Fire-hydrants must be sufficiently numerous and so located as to meet the requirements regarding number and size of fire-streams set forth in the preceding article. Hydrants are one-way, two-way, three-way, etc., according to the number of hose-connections provided. For most purposes the two-way hydrant is considered the most convenient, but in the dense portion of a large city, where many connections must be provided, three-way and four-way hydrants can be used to good advantage. Hydrants should, in any case, be numerous enough to enable the required number of streams to be furnished with a suitable nozzle-pressure. At points where a large number of streams are required, fire-cisterns are sometimes used instead of hydrants. These cisterns are fed by large pipes, and have an advantage over hydrants in that they allow several steamers to obtain their supply at one point.

For a 250-gallon stream the required nozzle-pressure is 45 pounds, and the loss of head per 100 feet of ordinary $2\frac{1}{2}$-inch hose is about 18 pounds (see Table No. 50, page 250), so that with a hydrant pressure of 100 pounds the length of hose to supply a 250-gallon stream cannot exceed 300 feet. A 175-gallon stream, with a 1-inch nozzle, requires 35 pounds nozzle-pressure, and causes a loss of head of 9 pounds per 100 feet of hose. With a hydrant pressure of 100 pounds the length of hose in this case might be 700 feet. With a hydrant pressure of 75 pounds, which is quite common, a 250-gallon stream could not be supplied through a length of hose greater than about 200 feet, and a 175-gallon stream through a length greater than about 450 feet. Hence the general rule that hydrants should be so spaced that no line of hose should exceed 500 to 600 feet, and for at least half of the streams required at any point the length of hose should not exceed 250 to 350 feet, according to the hydrant pressure. These lengths cannot be much increased even where fire-engines are used. In outlying districts two two-way hydrants should be available at any point, with a distance of not more than 500 to 600 feet to the more remote of the two.

The most convenient location for hydrants is at the street intersections, as they are then readily accessible from four directions. In cities of moderate size the required number of streams can readily be supplied by locating a hydrant at each street intersection, but in large cities intermediate hydrants are often necessary. Thus if the blocks in Fig. 220 are 300 feet long in each direction, and a two-way hydrant is placed at each corner, then a fire at A could be served from eight hydrants, with a maximum length of hose of 450 feet, giving sixteen

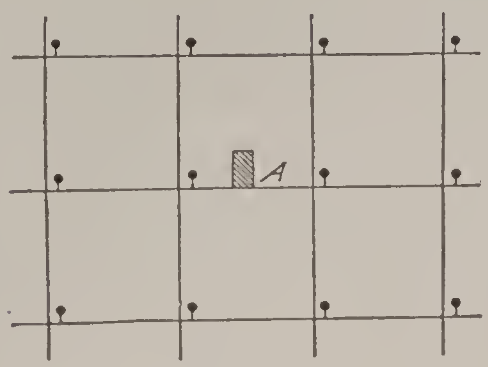

FIG. 220.

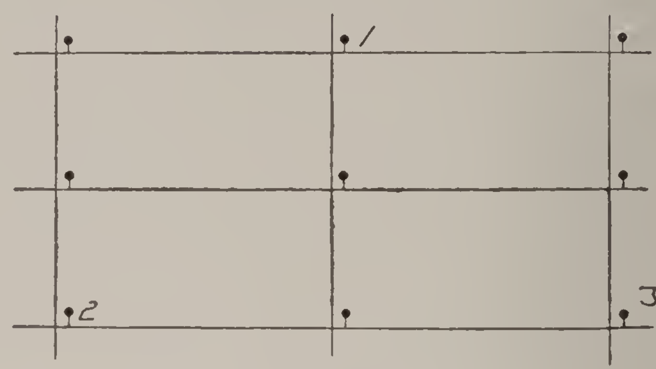

FIG. 221.

good fire-streams; while a fire at a street-corner could be served from thirteen hydrants, eight of which would, however, require hose-lengths of 600 feet. With blocks 600 feet by 300 feet, as in Fig. 221, a two-way hydrant at each intersection would supply not less than eight streams at any point, without exceeding 600 feet of hose. If only four streams are required, then one-fourth of the hydrants might be omitted, or every other hydrant in alternate streets, as hydrants 1, 2, and 3. This would just be within the requirement of a maximum hose-length of 600 feet. To omit half the hydrants, or to place them at one-half the intersections, would require the use of 750 feet of hose at certain points to supply two out of the four fire-streams. Such a spacing would therefore be inadequate. The necessary hydrant-spacing to furnish any given number of streams can be determined by the method here illustrated.

742. **General Arrangement of the Pipe System.**—From the data of Chapter II (page 32) it is evident that the fire demand will largely govern in the design of the pipe system. This is more and more true the smaller the town or district considered, and for single blocks the ordinary consumption can practically be neglected. To supply long, narrow districts, the general scheme would be to furnish the water mainly through a single large pipe of gradually decreasing size, with small parallel and branch mains supplying the side streets, somewhat as in Fig. 228 (page 766), districts 1 and 3. For broad areas, such as comprise the larger portions of most cities, the general arrangement usually adopted is to provide large mains at intervals of $\frac{1}{4}$ to $\frac{1}{2}$ mile, and to fill in between these mains with smaller pipes, thus forming a gridiron system. The smaller pipes are designed with special reference to supplying the fire-streams which are required at any point, without too great a loss of head, while the larger mains must be designed with reference to the ordinary consumption as well as to the

fire demand. The gridiron system is well illustrated by Fig. 222, which shows a section of the St. Louis distributing system.

A general principle which should be kept in mind when laying out a system is to so arrange the large mains that the smaller cross-mains may be fed from both ends, since a pipe so fed is equivalent to two pipes. It can furnish double the number of streams with the same loss of head, or the same number of streams with about one-fourth the loss of head, as when fed from one end only. This principle also makes it desirable to lay connecting pipes between separated districts, even when such pipes are not required for supplying local consumers. In the case of fire, each district may then be served from both ends. This plan is well illustrated in Fig. 228. In a gridiron system it is, for the same reason, desirable to provide large mains near the outside edges of the network. Extensions will of course make it impossible to do this at all times, but the desirability of having a circulating system, and avoiding dead ends as much as possible, should be kept well in mind. Dead ends are also objectionable on account of the stagnation which exists in the pipes and the deterioration of the water which is likely to ensue.

The size of mains and cross lines in the gridiron system will depend largely upon the number of fire-streams required at any point. In small cities, and outlying districts of large cities, 6-inch cross-mains with 8-, 10-, or 12-inch pipes at intervals of four to six blocks is a common arrangement. Four-inch pipe should rarely be used to supply hydrants. For compactly built districts many of the cross-pipes require to be 8 inches, and a more frequent use made of 12- and 16-inch pipes. A good arrangement for a comparatively large demand is to lay 6-inch pipes lengthwise of the blocks and 8-inch pipes crosswise. To supply large areas, still larger feeders, such as 24-, 36-, and 48-inch pipes, will be required. These are added to the system from time to time, as the needs of the city require and as the pressures become low through increased consumption. They should be so located and connected with the larger distributing-mains as to reinforce the pressure where most deficient.

743. Maximum Rates of Supply for Different Areas.—For the purpose of calculating the distributing system it is necessary to know the maximum rate of consumption for the entire city, and for large and small sections of the same, with suitable consideration for future growth. The rate for the entire city will enable the main supply-conduit, or the principal force-main, to be determined. For calculating the main distributing-pipes the city should be divided into

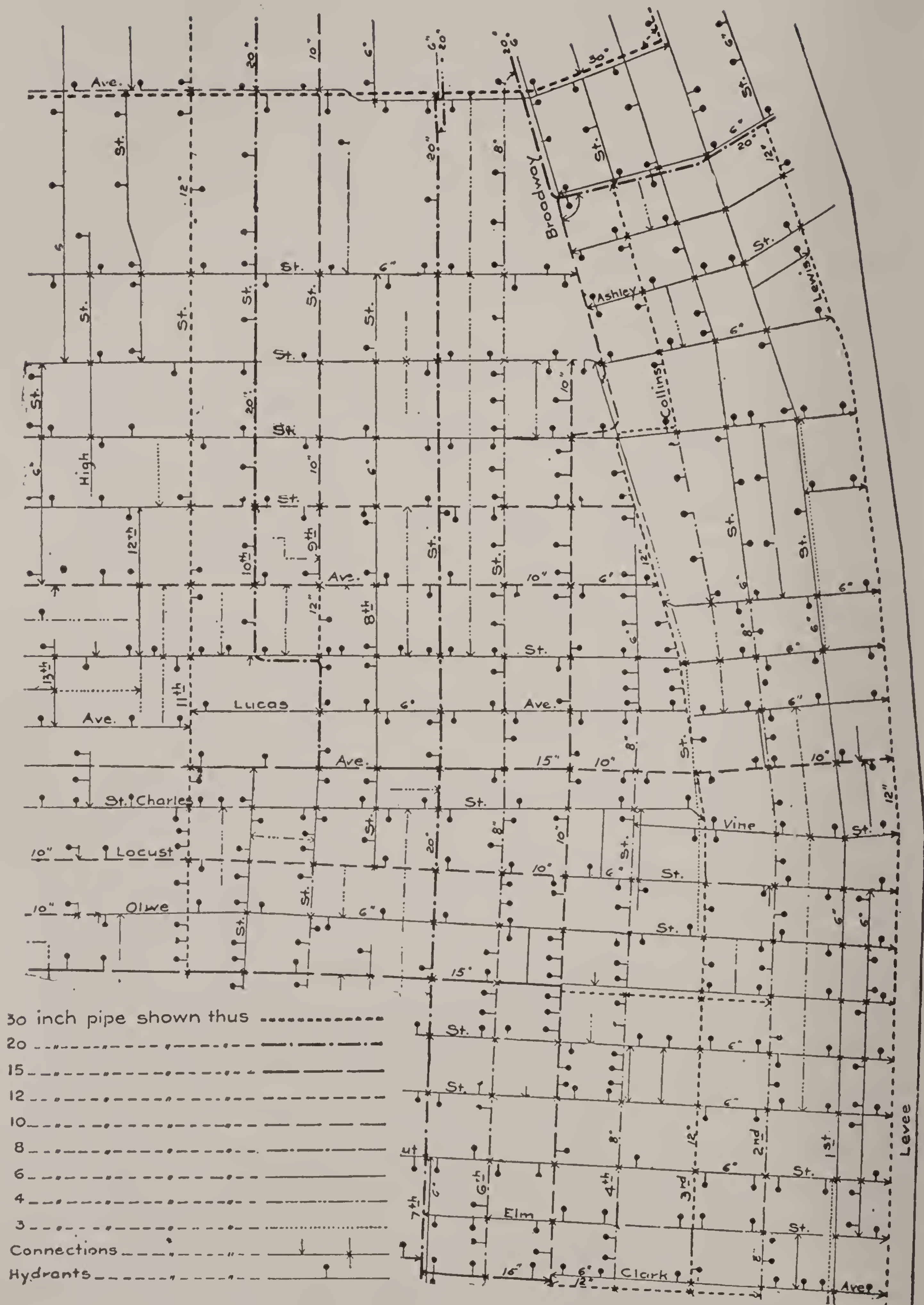

FIG. 222.—SECTION OF ST. LOUIS DISTRIBUTING SYSTEM.

relatively large districts, corresponding to the most probable location of such main arteries; then for the smaller pipes the demand for still smaller sections must be considered, and so on.

The extent to which provision for future growth should be made will be different in the various parts of the system, and will vary according to circumstances. It will not usually be necessary to design for more than fifteen to twenty years in the future, and sometimes even for less. In making extensions, large mains can readily be added from time to time, and these can often be placed where no pipes now exist. A better pressure will eventually be furnished by several good-sized mains, placed some distance apart, than by one very large main. For small cities, where the fire demand is relatively large and does not increase rapidly with the population, a small increase in size of mains will make the system serviceable for a relatively long period in the future, and in this case twenty-five or thirty years' growth might well be provided for. For that part of a system serving only a limited territory provision should be made for a fully built-up condition.

The maximum rate of consumption for the entire city has already been discussed in Chapter II, page 32. From the data there given the ordinary maximum rate is seen to be from 200 to 250 per cent of the yearly average. If the yearly average be 100 gallons per capita daily, the maximum ordinary rate will then be about 250 gallons per capita per day, or 0.17 gallon per capita per minute. The maximum fire rate by Kuichling's formula of Art. 740, assuming 250-gallon streams, is $250 \times 2.8 \sqrt{x}, = 700 \sqrt{x}$ gallons per minute, where $x =$ population in thousands. Thus for a population of 1000 the ordinary maximum rate may be about 170 gallons per minute, while the fire rate is likely to be 700 gallons, or four times as much.

After estimating the maximum rate of consumption for the city as a whole, the same should be done for the several districts, the probable future population, the maximum ordinary rate, and the maximum fire demand being estimated for each district independently. The required number of fire-streams for the separate districts should be determined in accordance with the data previously given. In combining the consumption for two or more districts, the required fire supply should be found by considering the district as a whole and not by adding the separate requirements. The fire demand will increase but little as the size of district increases.

To illustrate the points here considered, and other questions pertaining to the design of a distributing system, an arrangement of pipes will be assumed as shown in Fig. 228, page 766. This would be a suitable arrangement for the city of Madison, Wis., under certain assumed conditions which, for purposes of illustration, are different in some respects from the actual conditions. Pipes are shown where most needed, although there are a few more pipes actually laid than are shown. With respect to the natural conditions and the probable location of main arteries, the city may be divided into about ten districts as indicated. District No. 2 includes important factories. District No. 9 is a small suburb. The probable population fifteen to twenty years hence, immediately adjacent to the lines or which will be served by them, is assumed to be as given on the diagram; also the number of fire-streams simultaneously required in each district. The number for the entire city is taken at fifteen 250-gallon streams. The maximum rate of ordinary consumption is assumed to be 125 gallons per capita per day, the average rate at the present time being about 45 gallons. The maximum rate of supply required for each district for ordinary and fire supply will then be about as given in Table No. 101, page 767. For districts Nos. 2 and 3 together the maximum rate would be 380 gallons per minute for ordinary consumption plus 2000 gallons for fire purposes, = 2380 gallons per minute. Similarly, districts Nos. 4 to 10 would require altogether 2110 gallons for ordinary purposes, plus 15 fire-streams, or a total of 6560 gallons per minute. The calculation of the pipe system is further considered in Art. 750.

A larger provision for the future than here made would probably be desirable for the main pipes in the vicinity of the pumping-station and in the denser portion of the city, as no additional lines of pipes would ever be needed here to supply the local demand. At outlying districts many streets remain unoccupied, which gives opportunity for enlarging the capacity when pipes are laid in these streets.

744. Velocities of Flow for Fire Supplies.—In calculating a pipe system it is convenient to get first a good notion of the practicable and economical velocities which may ordinarily be used for the maximum fire draught. The most suitable velocities, or losses of head, will depend somewhat upon the system of supply, and also upon the location and elevation of the pipes in question. The various conditions will be included by considering the problem with respect to the following cases:

(1) A pumping system in which fire pressure is constantly maintained.

(2) A pumping system in which fire pressure is maintained only at times of fires, the pressure at other times being sufficient only for ordinary purposes.

(3) A gravity system with sufficient pressure for fire purposes.

(4) A pumping or gravity system furnishing a low working-pressure only, fire-engines being used.

(1) In this case if it be assumed that a certain minimum hydrant pressure is required at various points, this pressure, and the loss of head in the pipe system, will control the head against which the pumps operate; and as this head is constantly maintained, the disadvantage of small pipes and large frictional loss is very great. The best size of pipes, or the economical velocities, will in this case be different from those given in Chapter XXVI, page 370, but may be determined in a similar way.

Let Q = average yearly rate of flow, or the average rate of consumption for any given district served by a given pipe. Let Q_1 = maximum rate of fire demand, plus the rate for ordinary purposes. In the case under consideration the pressure-head at the pumps will be determined by Q_1, but the actual yearly expense of pumping will be proportional to the volume Q, since the total amount pumped for fires is very small. We then have, as in eq. (5), page 604,

$$s = 100 \frac{Q_1^{\frac{7}{4}}}{d^{\frac{19}{4}}},$$

and, as in eq. (6), the yearly cost =

$$A = bsQ + 20r + 2ard^{1.55}.$$

Substituting the value of s, differentiating, etc., as on page 605, we find that the economical velocity

$$v = 30\left(\frac{Q_1}{Q} \cdot \frac{ar}{b}\right)^{.36} d^{.27}. \qquad (1)$$

The economical velocity in this case is thus equal to the economical velocity for the average rate of consumption, as given on page 605, multiplied by $\left(\frac{Q_1}{Q}\right)^{.36}$, or approximately by the cube root of the ratio of the maximum to the ordinary rate. For small areas, such as two or three blocks, the maximum rate is likely to be fifty or more times the ordinary rate, but for areas consisting of several blocks the ratio is much less. Thus in the table on page 767, district No. 5, the ratio of the maximum to the average rate is about 12, and in No. 2 it is about 23, while for districts Nos. 4 to 10 combined it is only 4.

To illustrate the variation in economical velocity with varying conditions, such velocities have been calculated for certain cases, taking the upper figures of Table No. 77, page 606, as a basis. The results are as follows:

		2	4	10	25	50
Ratio of maximum to ordinary rate......		2	4	10	25	50
Economical velocities...	6-inch pipe...	2.3	3.0	4.2	5.8	7.4
	8- " " ...	2.5	3.2	4.5	6.2	8.0
	10- " " ...	2.7	3.4	4.7	6.6	8.5
	12- " " ...	2.8	3.6	5.0	7.0	
	16- " " ...	3.0	3.9	5.4		

For still larger pipes the ratio will usually be quite small, and the velocities therefore not much greater than given in Table No. 77.

(2) Where fire pressure is furnished only when needed, and a comparatively low pressure is used at other times, the pipes are to be designed, first, to give economical velocities for ordinary service, and, second, to give practicable velocities and losses of head for fire service. It is desirable to limit the loss of head so that the fire pressure at the pumps will not need to be excessively high, both to avoid the use of extra thick pipe and heavy plumbing, and to avoid too large variations in the pump pressures. Ordinarily about 120 to 130 pounds is as high a pump pressure as is desirable to use, but sometimes a greater pressure is necessary to furnish fire-streams on the higher areas of the city. With a hydrant pressure of 80 to 100 pounds, and a pump pressure of 120 to 130 pounds as a desirable maximum, the allowable velocities of flow will not much exceed those given in case (1), and may in fact be lower. The smaller mains would seldom be affected in any event, as a change of 2 inches in the diameter of a small main so greatly changes its capacity, but some of the larger mains might be reduced somewhat in size.

(3) Where a gravity system furnishes a certain definite pressure at the distributing-reservoir, the loss of head allowable in the pipe system is more or less closely defined. If the available pressure is low, the distributing-pipes will need to be made large in order to obtain as much head at the hydrant as possible, but for certain areas and for large fires it may be cheaper to employ fire-engines than to go to a large expense to save loss of head in conduit and distributing-pipes. Where the available pressure is high, then the loss of head in the pipe system may equal the difference between this and a hydrant pressure of, say, 100 pounds.

Whenever a certain definite loss of head is allowable between a reservoir and a certain section of the city, the proper distribution of this loss among large and small mains is a matter of considerable importance. A general solution of the problem will be of some aid.

In Fig. 223, let d_1, Q_1, v_1, l_1, h_1, and s_1 be, respectively, the diameter, discharge, velocity, length of pipe, loss of head, and hydraulic slope with respect to a main, AB; and let d_2, Q_2, etc., refer to corresponding quantities in a branch BC, of which there are several of the same length and size. Let n = number of branches; H = total loss of head from A to C, $= h_1 + h_2$. The total cost of the system is, by eq. (1), page 604,

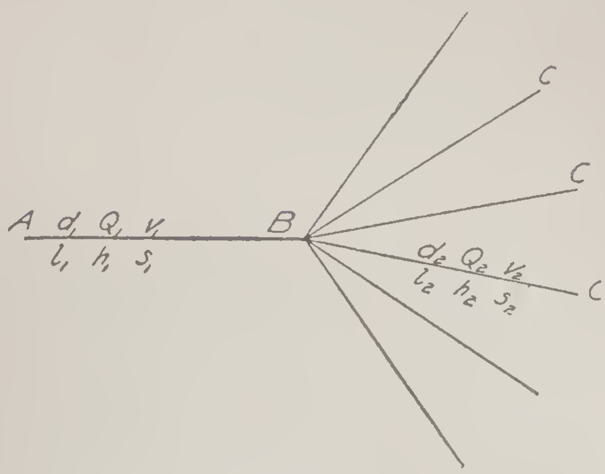

FIG. 223.

$$A = 20(l_1 + nl_2) + 2a(l_1 d_1^{1.55} + nl_2 d_2^{1.55}). \quad . \quad . \quad . \quad (2)$$

Substituting the value of d as derived from eq. (5), page 604, we have

$$A = 29(l_1 + nl_2) + 9a\left(\frac{Q_1^{.57} l_1^{1.325}}{h_1^{.325}} + nQ_2^{.57} \frac{l_2^{1.325}}{h_2^{.325}}\right). \quad . \quad (3)$$

If we differentiate with respect to h_1, or add an increment dh_1, we at the same time subtract the same amount from h_2, since $h_1 + h_2 = H =$ a constant. Hence $dh_1 = -dh_2$. Differentiating then, and equating to zero, etc., we have for a minimum cost

$$\frac{Q_1^{.57} l_1^{1.325}}{h_1^{1.325}} = n \frac{Q_2^{.57} l_2^{1.325}}{h_2^{1.325}},$$

or since, in general, $\frac{h}{l} = s$, we have

$$\frac{s_1}{s_2} = \frac{Q_1^{.42}}{n^{.74} Q_2^{.42}},$$

or, practically,

$$\frac{s_1}{s_2} = \left(\frac{Q_1}{n^2 Q_2}\right)^{.4} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

In the case where $nQ_2 = Q_1$, or where all the branch pipes are discharging equally, then $\frac{s_1}{s_2} = \left(\frac{1}{n}\right)^{.4}$; that is, the hydraulic slopes for AB and each of the branches BC should be as $1 : n^{.4}$. Thus if a single large pipe branches into ten smaller ones, each designed to carry one-tenth the total volume, then the hydraulic slopes should be as

$1 : 10^{.4} = 1 : 2\frac{1}{2}$. If four of the smaller pipes be designed to carry the entire volume, as for fire purposes, then $Q_2 = \frac{1}{4}Q_1$, and we have

$$\frac{s_1}{s_2} = \left(\frac{4Q_1}{10^2Q_1}\right)^{.4} = \frac{1}{3.6}.$$

The actual sizes of pipes for any given total loss of head and discharge can readily be found by trial by the aid of the diagram on page 243.

The general principle here brought out is that in a distributing system containing a large number of small pipes, only a few of which are ever discharging at their full capacity at the same time, most of the loss of head at times of fires should occur in the near vicinity of the fire, and relatively little in the large mains leading thither.

As already stated, the possible variation in size in the smaller pipes will be very little, but in the larger and more expensive mains considerable economy can be secured by a careful study of the problem, and by calculations of two or more possible arrangements.

(4) Where a pressure sufficient only for ordinary purposes is provided, the pipes must still be designed largely with reference to fire consumption, so that they will at all times be able to furnish full supplies to the fire-engines without suction. The problem is essentially the same as that discussed under (3).

745. **Loss of Head in Distributing-pipes.**—To aid in the selection of the smaller sizes of pipes it will be convenient to tabulate the losses of head in such pipes, when supplying various numbers of fire-streams. Table No. 98 is made up in this way, the values given being based on the diagram of page 243.

It will be readily seen that for any given number of streams the choice of pipe will usually be confined to two or possibly three sizes, since the loss of head varies so rapidly with change of size. The ordinary consumption may be neglected for short pipes supplying only one or two blocks, while for larger areas the ordinary rate may be converted into an equivalent number of fire-streams.

For example, if ten streams are required, the choice would probably be either a 12-, 14-, or 16-inch pipe. In the case of a city where 12-inch pipes are used for comparatively short submains, such a size might be employed, but where serving larger districts, or where the available head is small, a 12-inch would be too small, and a 14- or 16-inch should be used. The 16-inch pipe would probably be the best size if the district comprised a large portion of a small city, where the large main would be relatively long and the ratio of fire to ordinary consumption not very large. In the same way a supply of six fire-

TABLE NO. 98.

VELOCITY OF FLOW AND LOSS OF HEAD PER 1000 FEET IN DISTRIBUTING-PIPES WHEN DELIVERING GIVEN NUMBERS OF 250-GALLON FIRE-STREAMS.

Loss of head, in feet, given by light-faced type. Loss of head, in pounds, given by bold-faced type.

No. of 250-gal. Fire-streams.	4-inch.		6-inch.		8-inch.		10-inch.		12-inch.		14-inch.		16-inch.		20-inch.		24-inch.		30-inch.		36-inch.	
	Veloc-ity.	Loss of Head.	Veloc-ity.	Loss of Head.	Veloc-ity.	Loss of Head.	Veloc-ity.	Loss of Head.	Veloc-ity.	Loss of Head.	Veloc-ity.	Loss of Head.	Veloc-ity.	Loss of Head.	Veloc-ity.	Loss of Head.	Veloc-ity.	Loss of Head.	Veloc-ity.	Loss of Head.	Veloc-ity.	Loss of Head.
1	6.4	50 **21.8**	2.9	7.4 **3.2**																		
2	.....	.....	5.8	25 **10.9**	3.2	6.4 **2.8**																
3	.....	.....	8.6	52 **22.6**	4.9	12.8 **5.6**	3.1	4.5 **1.9**														
4	.....	.....	.....	.....	6.5	22 **9.6**	4.2	7.4 **3.2**	2.8	3.2 **1.4**												
5	.....	.....	.....	.....	7.9	32 **13.9**	5.1	10.9 **4.7**	3.5	4.5 **1.9**	2.6	2.2 **1.0**										
6	.....	.....	.....	.....	9.6	43 **18.7**	6.2	15 **6.5**	4.2	6.1 **2.6**	3.0	3.1 **1.3**	2.4	1.6 **.69**								
8	.....	.....	.....	.....	.....	.....	8.2	25 **10.9**	5.7	10.5 **4.6**	4.2	5.0 **2.2**	3.2	2.7 **1.2**								
10	.....	.....	.....	.....	.....	.....	10.2	37 **16.1**	7.1	15.4 **6.7**	5.2	7.3 **3.2**	4.0	3.9 **1.7**	2.6	1.4 **.59**						
12	.....	.....	.....	.....	.....	.....	.....	.....	8.6	22 **9.6**	6.3	10.2 **4.4**	4.8	5.3 **2.3**	3.1	1.9 **.8**	2.1	0.8 **.35**				
16	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	8.4	17 **7.4**	6.4	9.0 **3.9**	4.1	3.2 **1.4**	2.8	1.3 **.57**				
20	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	8.0	13 **5.6**	5.1	4.7 **1.6**	3.6	2.0 **.87**	2.3	0.7 **.29**		
25	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	6.5	7.0 **3.0**	4.5	3.0 **1.3**	2.9	1.0 **.43**		
30	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	7.8	9.5 **4.1**	5.4	4.0 **1.7**	3.4	1.4 **.61**		
40	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	7.0	6.5 **2.8**	4.5	2.3 **1.0**	3.1	0.9 **.39**

streams would in most cases call for either a 10- or 12-inch pipe, and four streams an 8- or 10-inch, etc. It is to be particularly noted that a 4-inch pipe is hardly suited for even a single stream, and a 6-inch pipe for not more than two streams.

The table and computations refer of course to the actual flow in the pipe. A pipe fed from both ends and supplying intermediate hydrants is equivalent to two pipes and should be so calculated. The diagram on which the table is based allows about 20 per cent increase in loss of head for corrosion, but in many cases a considerably greater allowance should be made for the smaller sizes, and unless it is quite certain that corrosion will not be great, 4-inch pipes should not be used at all to supply hydrants.

As it is frequently desirable to ascertain the size of a single large pipe equivalent to several small pipes, the relative discharging capacities of pipes of different sizes for the same loss of head are given in Table No. 99, the capacity of a 4-inch pipe being taken as 1.

TABLE NO. 99.

RELATIVE DISCHARGING CAPACITIES OF PIPES FOR THE SAME LOSS OF HEAD.

Size of pipe	4	6	8	10	12	14	16	20	24	30	36
Relative capacity	1	3	$6\frac{1}{2}$	12	20	30	43	80	130	235	390

746. General Problems Pertaining to the Flow through Compound Pipes.—In calculating the flow through a system of pipes, several problems will arise. Some of these can be solved only by rough approximations, but there are two classes of problems for which simple general solutions can readily be found. Where a distributing system consists of but a few pipes, or is in the form of a long narrow district, the formulas derived can often be easily and directly applied. Where, however, the pipes spread over a broad area it is impracticable to obtain anything more than a very rough approximation to exact results, but the general relations brought out by the solution of these two cases will assist in making reasonable assumptions in the more complicated case.

The two general cases to be considered are:

(1) The discharge from, or loss of head in, a single pipe-line of varying cross-section.

(2) The discharge from, or loss of head in, a line of two or more pipes extending between any two given points.

In both cases an algebraic expression could readily be derived giving the exact relation between discharge and loss of head, but practically the problem is best solved by determining the size of a single pipe which shall be equivalent to the given combination, that is, such a size as will give the same loss of head for a given discharge. The method of solution can best be explained by solving two examples.

1. As an example of the first case let the sizes and lengths be as given in Fig. 224. To get the size of a pipe 1600 feet long which will give the same loss of head for the same discharge, assume any convenient discharge, such as 400 gallons per minute. Then by the diagram on page 243 we have the following losses of head:

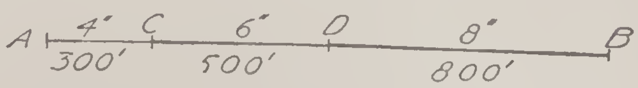

FIG. 224.

$$\begin{aligned} \text{For } AC, \text{ loss} &= 120 \times .3 = 36 \text{ feet} \\ \text{`` } CD, \text{ ``} &= 17 \times .5 = 8.5 \text{ ``} \\ \text{`` } DB, \text{ ``} &= 4.5 \times .8 = 3.6 \text{ ``} \\ \text{Total loss of head} &= 48.1 \text{ feet} \end{aligned}$$

The total loss of head is at an average rate of 30 feet per 1000. The size which will discharge 400 gallons per minute at this loss of head is then found by the diagram to be 5.3 inches in diameter, which size can be substituted for the given combination in all calculations relating to the section *AB* as a whole.

2. In the other case assume the arrangement shown in Fig. 225. The problem is to get the size of a single pipe from *A* to *B*, equivalent to the given combination. Get first, by the method just described, the size of a uniform pipe *ACDB*, 1200 feet long, which shall be equivalent to the pipes *A C D B* as shown. This size will be 4.5 inches. Now the loss of head between *A* and *B* must be the same by both routes. Assume any loss of head, as 10 feet, and find the discharge by each route. For the 6-inch pipe the loss is $\frac{10}{.6} = 16.7$ feet per 1000 feet, and the discharge = 390 gallons per minute. For the 4.5-inch pipe 1200 feet long the loss is $\frac{10}{1.2} = 8.33$ feet per 1000, and the discharge = 130 gallons per minute. The total discharge = 520 gallons, and the size of pipe 600 feet long which will deliver 520 gallons at a loss of head of 10 feet is found to be about 6.7 inches, which is the equivalent size desired. If three or more pipes extend from *A* to *B*, the problem can be solved in a similar manner. Where

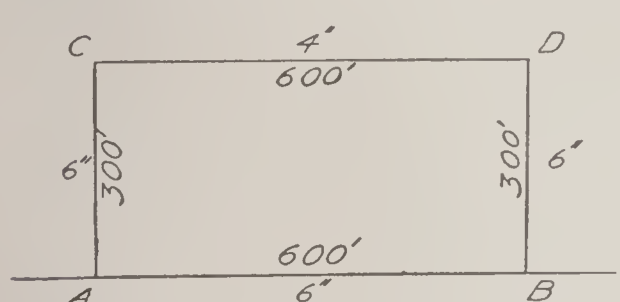

FIG. 225.

the pipes are of the same length the relative discharges can be determined from the table of relative capacities on page 758.

In practice there will usually be complications from the fact that two routes may be connected at more than two points, in which case no simple exact method of calculation can be used; but by making certain reasonable assumptions as to the direction of flow, and eliminating some of the cross-connections, the problem may be reduced to the simple form just discussed. A more extended example is given on page 768. In making assumptions as to the relative flow in different lines, there should be kept in mind the very great effect of diameter (about as $d^{\frac{5}{2}}$) and the comparatively small effect of distance (about as $\sqrt{l}$).

747. Calculation of the Pipe System.—Before beginning the calculation of a distributing system, a map should be prepared showing thereon the streets where pipes are required, probable lines of future growth, character of buildings in various districts, etc. On the map can also be recorded the population of various districts, ordinary rates of consumption, and number of fire-streams required simultaneously at different points. There should also be shown on this map the desired hydrant pressure at various points, referred to a horizontal plane as well as to the ground-surface. This pressure will of course be selected with reference to the head available, and may need to be altered before the plans are finally completed.

In designing a pipe system it will be well to first lay out in a tentative way certain main lines of pipes, or arteries, to supply certain large districts, which may be more or less separated by undeveloped territory. Then it will be convenient to determine upon the size and arrangement of the smallest cross-mains, according to the number of fire-streams needed in any given small area. The arrangement of submains feeding these smaller ones, and connections with the main arteries and the submains in other districts, can then be arranged, provision being made at all points for the ordinary consumption as well as the fire supply. Then, with a tentative plan, the maximum number of fire-streams should be assumed in use at various points in the system, and the loss of head between the source and the hydrants in question estimated as closely as practicable. This loss should not exceed the desired limit, and for economy should be adjusted in accordance with the principles of the preceding articles. Several arrangements should be tried and comparative estimates made. As already shown, the possible variations in size will not be large.

The calculations involved will be only roughly approximate, and to enable them to be made at all, certain assumptions may be necessary as noted in the preceding article. If the area is broad, the calculations are much more difficult than where it is long and narrow. It is to be noted that any large system will be built up gradually, and will have to be reinforced from time to time by additional larger mains or by replacing small ones by large ones. The actual loss of head which obtains may then be known by actual measurements, and the effect of additional mains can be quite easily estimated. However, in laying out new systems, and often in investigations of old systems, certain calculations need to be made.

748. *Calculation of Small Service Mains.*—In long narrow districts the pipes can be calculated by the methods already described, but where the system covers a broad area the problem is a very indefinite one. In such a case we will usually have, as noted on page 688, a general scheme of large pipes filled in between with smaller pipes, forming a sort of gridiron system.

The size and arrangement of the small mains can be determined conveniently by the following approximate method: In Fig. 226 is represented a system of small cross-mains where the streets are 250 to 300 feet apart in each direction. It is assumed that the pipes are fed from both directions. Suppose it is desired to concentrate 20 fire-streams at *A* without exceeding 600 feet of hose, assuming that the hydrants are suitably spaced to render this possible. Draw a circle with a radius of about 500 feet with *A* as a center. It will be found to cut fourteen lines of pipe. It will then be approximately correct to assume these fourteen lines of pipe tributary to the fire, without reference to the exact location of the hydrants. Each pipe where cut by the circle will then on the average have to supply 1.7 fire-streams, or 420 gallons per minute, and if 6-inch pipes be used, the loss of head at this point will therefore be about 20 feet per 1000 feet, assuming all pipes to discharge at the same rate. This assumption will be approximately correct for a small area situated in a large system and surrounded by large pipes, as the loss of head near the fire will be much greater than at points more remote. The loss of head at points nearer *A* will be at a rather less rate than at the given circle, if the hydrants be evenly distributed; and the loss of head outside this circle will rapidly decrease as other vertical and horizontal lines are crossed.

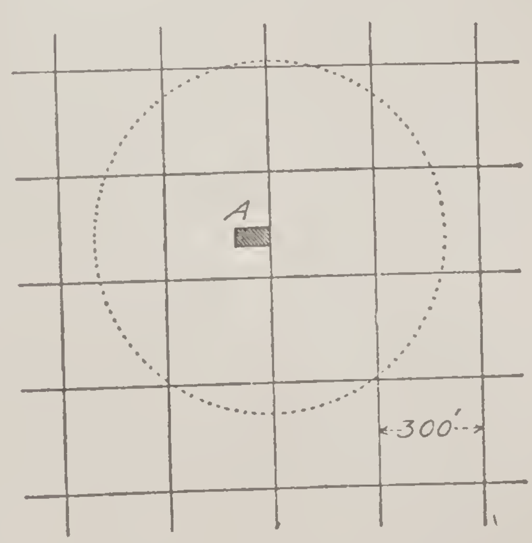

FIG. 226.

If thirty streams were needed in the same area, each pipe would have to supply 2.1 streams, or 520 gallons. If 6-inch pipes be used, the loss of head would be about 28 feet per 1000, which might be a greater loss than desirable. In this case the supply could be furnished by the use of 8-inch pipes running one way and 6-inch the other. Then at least six 8-inch pipes would be available, and eight 6-inch. By the table on page 758 it is found that six 8-inch pipes are equivalent to thirteen 6-inch pipes, hence we have an equivalent of twenty-one 6-inch pipes, giving a loss of head of about 14 feet per 1000. In this case every alternate pipe might perhaps be made an 8-inch. For still larger supplies all pipes can be made 8-inch, or a 10- or 12-inch pipe placed in every second or third street. In general it is somewhat more economical to provide volume in one or two large pipes than to increase the size of all, but care should be taken that the smaller pipes are not too long for the number of hydrants placed upon them.

The approximate loss of head, locally, can be found in the way described for any given arrangement, where the location is in the central part of a large network of pipes or where large pipes surround the territory.

For residence districts the blocks are usually about 250 to 300 feet by 500 to 600 feet, and larger pipes must be used to furnish the same number of streams with the same loss of head.

To aid in estimating the value of any particular arrangement of cross-mains, the approximate number of fire-streams which may be supplied by different arrangements of pipes at a loss of head of about 10 pounds for the first 1000 feet from the center, is here given.

TABLE NO. 100.

APPROXIMATE NUMBER OF FIRE-STREAMS SUPPLIED BY DIFFERENT ARRANGEMENTS OF PIPES FOR A LOSS OF HEAD OF 10 POUNDS IN FIRST 1000 FEET.

Arrangement of Pipes.	Approximate No. of Streams.
Blocks 300 *feet by* 600 *feet.*	
All 4-inch	6
4-inch lengthwise, 6-inch crosswise	10
All 6-inch	20
6-inch lengthwise, 8-inch crosswise	25
All 8-inch	40
6-inch lengthwise, 10-inch crosswise	40
6-inch lengthwise, 12-inch crosswise	50
Blocks 300 *feet by* 300 *feet.*	
All 4-inch	9
4-inch and 6-inch	18
All 6 inch	25
6-inch and 8-inch	40
All 8-inch	60
6-inch and 10-inch	65

The loss of head here considered refers to the most centrally located hydrant; the loss at other hydrants will be less. In many cases a considerably larger loss than here given would be permissible, and the possible number of fire-streams could be increased, but not often more than 25 per cent.

749. *Calculation of Large Mains.*—The arrangement of mains and submains must be made with reference to the ordinary consumption as well as the fire demand, and proportioned in accordance with the principles already discussed. The best velocities will be much lower than in the small pipes. If nothing but fire demand existed, an ideal system would consist of a network of pipes of the sizes determined in the last article, surrounded by a large feeder so as to maintain a nearly uniform pressure at the periphery. The water could then be concentrated for fire purposes with the least loss of head, and no other large mains would be required. But to provide adequate pressure over large areas the ordinary consumption must be taken account of. A certain number of large mains will be required, and these will increase in size as we approach the source of supply. It is in these large mains and branches that a great saving can be effected by having two or more reservoirs located at different points in the city. The possibility of one of the large mains being shut off in time of fire should be considered, and the system so arranged that the small mains may be fed from two or more larger ones.

Let Fig. 227 represent a part of a large network of pipes, in which the lines AB and BC are at or near margins of the system. With the arrangement shown let it be required to determine approximately the maximum loss of head between D and any other point, such as Z, in the section EH, and to adjust the size of the large mains. Suppose that twenty fire-streams, or 5000 gallons per minute, are required in the vicinity of Z, and, further, that the maximum rate of the ordinary consumption is 400 gallons per minute in each of the large divisions.

With twenty fire-streams in action near Z, the loss of head in the 6-inch pipes between Z and the surrounding large pipes will be found to be about 10 pounds, although the pressures in these large pipes will vary considerably at different points. The line EB feeds five 6-inch pipes, three of which are likely to be called upon simultaneously to supply about two fire-streams each; hence EB would have to supply about six streams. From the table on page 757, we would evidently need about a 12-inch pipe, and this is the size which would result by the application of the method of Art. 744. BH will also be made a 12-inch pipe. Of the twenty fire-streams demanded at Z, six or eight may be assumed to come from *GH* together with the five 6-inch pipes between *GH* and *JK*. The line *GH* will also partly supply *BH*, and as the capacity of five 6-inch pipes is not much more than one 10-inch,

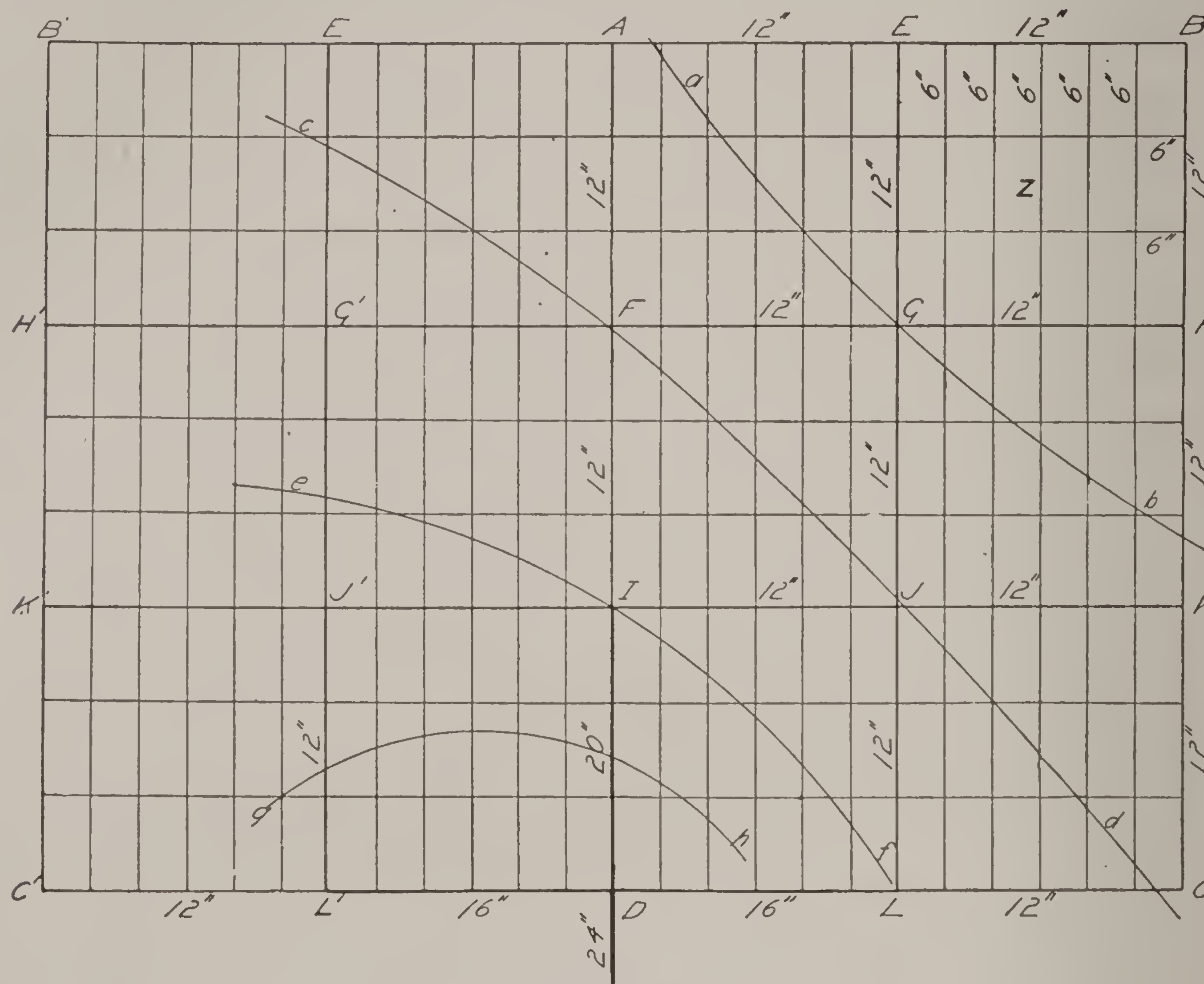

FIG. 227.

we may assume that five or six streams will be carried by *GH*. This line will then need to be again a 10- or 12-inch pipe.

Farther away from *Z* the proportion carried by each large main becomes very difficult of estimation. In the arrangement here assumed the small pipes have about one-half the capacity of the large ones, except in the vicinity of *D*. It will be reasonable, then, to design the large pipes to carry two-thirds of the total required quantity, and to provide for contingencies by assuming at the same time one of the large pipes tributary to any district to be out of service. Another method which may be used is to assume all the water carried by the large pipes, leaving the contributions of the smaller pipes as a margin of safety. Whichever assumption be made, the approximate loss of head in the large mains from *Z* towards the source *D* can then be found in the same general way as employed for service-mains. To do this we may sketch the lines *ab*, *cd*, etc., across the system in such a direction that they will represent, as nearly as can be judged, lines of equal pressure; then note the number and size of large mains cut by these lines. The relative flow in each main can then be estimated in proportion to its capacity and the directness of the route, and the approximate loss of head per 1000 feet determined at several points, and finally the total loss between *Z* and *D*. The maximum rate of the ordinary consumption should be taken account of at each section. Very roundabout routes should be omitted from the calculation, and a margin allowed for contingencies in one of the ways mentioned above.

The loss of head between *Z* and *D* may in the present case be estimated as follows: On section *ab* the rate of flow is about 5700 gallons per minute, two-thirds of which is 3800 gallons. Four 12-inch pipes are intersected, and, omitting one for contingencies, the flow through each of the others will average about 1300 gallons per minute, which will give a loss of head of about 5 feet per 1000. For section *cd* the maximum rate will be about 6800 gallons, with about four pipes available for two-thirds this amount, which will involve a loss of head of about 3.8 feet per 1000. Similarly on section *ef* the volume is about 8500 gallons, with four pipes in service, giving a loss of head of 5.5 feet per 1000. On section *gh* we will assume available for the total volume, one 20-inch pipe, one 16-inch, and seven 6-inch pipes. The volume equals about 9600 gallons per minute. The loss of head for this combination is about 5 feet per 1000. Farther towards *D* the supply may be assumed to come through the 20-inch pipe and one 16-inch, which will also give about 5 feet loss of head per 1000 feet. In the 24-inch supply-pipe, with a rate of 98,000 gallons per minute, the loss of head would be about 6 feet per 1000.

Considering the average distance traveled from section to section, assuming blocks 300 by 600 feet, the actual loss of head from *ab* to *cd* is approximately 8 feet, from *cd* to *ef* 8.5 feet, from *ef* to *gh* 5 feet, and from *gh* to *D* 4.5 feet. Adding the loss of head in the small pipes and mains near *Z*, we find a total loss of head of 50 to 55 feet, or about 23 pounds, which would ordinarily be a reasonable allowance.

With a reservoir at *A*, or beyond, nearly all the 12-inch submains could readily be reduced to 10-inch or 8-inch pipes, or perhaps most of them to 6-inch. The volume flowing through any pipe would be reduced about one-half, and the distance traveled also about one-half, thus reducing the loss of head very greatly. A large main of about 20 inches in diameter, extending from pumps to reservoir, would, however, be required.

750. *Example.*—In Fig. 228 is shown a possible arrangement of pipes and hydrants to meet the conditions stated on page 752 and in Table No. 101.

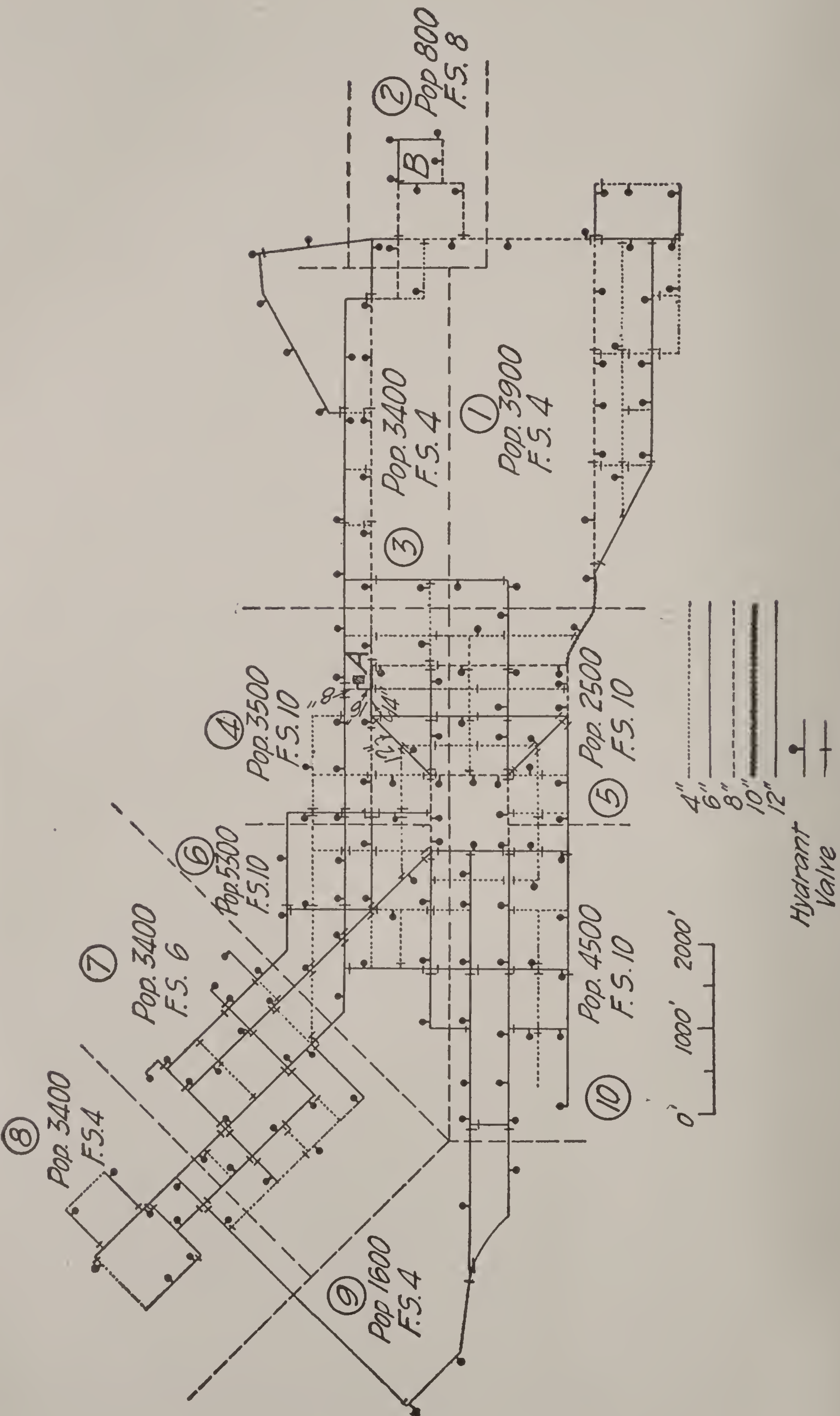

Fig. 228. — Example of a Distributing System.

A considerable use is made of 4-inch pipe, as experience has shown that well-coated pipe will corrode very slowly with the Madison water. The fire-pres-

TABLE NO. 101.

ESTIMATED POPULATION, AND MAXIMUM ORDINARY AND FIRE-RATES FOR DIFFERENT DISTRICTS OF FIG. 228.

District.	Estimated Future Population.	Maximum Ordinary Rate, Gallons per Minute.	Maximum Fire-rate, Gallons per Minute.	Total Maximum Rate, Gallons per Minute.
1	3900	340	800	1140
2	800	90	2000	2090
3	3400	290	800	1090
4	3500	300	2500	2800
5	2500	220	2500	2720
6	5300	460	2500	2960
7	3400	300	1200	1500
8	3400	300	1200	1500
9	1600	140	800	940
10	4500	390	2500	2890
Entire city	32300	2830	3750	6580

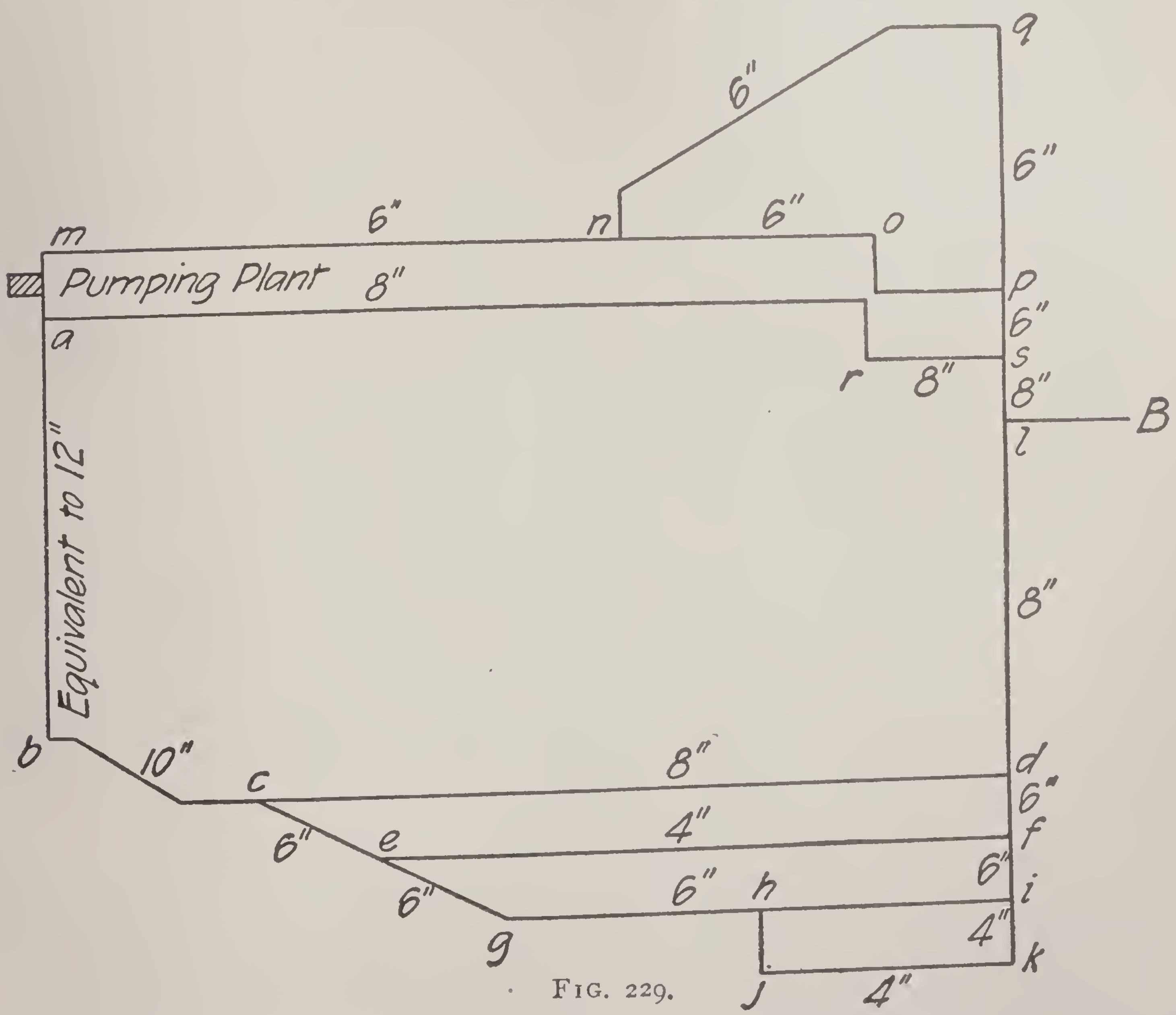

FIG. 229.

sure at the pumping-station is assumed to be 120 pounds, and the pipes made of such size that the loss of head to the most remote hydrant will not exceed

40 pounds. The ground is assumed to be level. Many other arrangements could be made, some of which might be more economical than that given.

As a further example of the application of the method of calculation given in Art. 746, page 758, we will here compute the approximate loss of head from the pumping-station *A*, Fig. 228, to the point *B*, where it is assumed that eight fire-streams are in use. We will for the present neglect the ordinary consumption and, to make the solution possible, will omit certain cross-lines and modify the arrangement as shown in Fig. 229. We will also estimate that the various pipes from *a* to *b* are equivalent to a single 12-inch pipe. The problem is to determine the loss of head from the pumps to the point *l* by finding the size of a single pipe which will be equivalent to the system shown. The blocks of Fig. 228 are assumed to be 600 feet long by 300 feet wide. Beginning with the loop *hki* we first find a single pipe, *hi*, equivalent to the two pipes shown, then a single pipe *ef* equivalent to the given pipe *ef* together with the new pipe *egif* just found, etc. The calculations are very quickly made as shown in Art. 746. The results in detail are as follows:

Line.	Diameter.	Length.	Equivalent Pipes. Line.	Equivalent Pipes. Diameter.	Equivalent Pipes. Length.
hi	6	1200	*hi*	6.5	1200
hjki	4	1800			
egh	6	1870			
Equiv. *hi*	6.5	1200	*ehf*	6.7	3370
if	6	300			
Equiv. *ehf*	6.2	3370	*ef*	6.7	3000
ef	4	3000			
ce	6	370			
Equiv. *ef*	6.7	3000	*cefd*	6.5	3670
fd	6	300			
cd	8	3600	*cd*	9.5	3600
Equiv. *cefd*	6.5	3670			
ab	12	1800			
bc	10	980	*abl*	9.4	7880
Equiv. *cd*	9.5	3600			
dl	8	1500			
nqp	6	3300	*np*	7.4	2100
nop	6	2100			
amn	6	3000			
Equiv. *np*	7.4	2100	*ans*	6.4	5400
ps	6	300			
Equiv. *ans*	6.4	5400	*as*	9.3	4800
ars	8	4800			
Equiv. *as*	9.3	4800	*asl*	9.2	5100
sl	8	300			
Equiv. *abl*	9.4	7880	*al*	11.3	5100
Equiv. *asl*	9.2	5100			

Thus the system *abl* is equivalent to a single pipe 9.4 inches in diameter and 7880 feet long, and the system *asl* is equivalent to a 9.2-inch pipe 5100 feet long. Finally, these two are found to be equivalent to a single pipe 11.3 inches in diameter and 5100 feet long. The loss of head in such a pipe, for eight streams of 250 gallons each, would be 13.6 feet per 1000, or a total loss of about 70 feet or 30 pounds. The volume carried by the system *abl* will be equal to that which would be carried by a 9.4-inch pipe 7880 feet long, with a loss of head of 70 feet. This will be about 900 gallons per minute. The volume carried by the other system will be 1100 gallons per minute. Corrections can be approximately made for the amounts consumed locally by adding such amounts to the above quantities at a few points along the pipe system.

751. Separate Services for Different Zones of Elevation.—Where the different parts of a town vary considerably in elevation, it is frequently advisable to divide the distributing system into two or more independent portions, each serving an area or zone situated between certain limiting elevations. It often happens that only a small portion of a city is at a high elevation, and by thus separating the systems of distribution a comparatively small amount of water will need to be raised to the maximum height, the greater portion being pumped against a much lower pressure. By this arrangement a large saving can be effected in the expense of pumping, and the use of excessive pressures in the lower districts will also be avoided.

Various arrangements may be made for supplying the different zones. Each zone may be practically an independent system, with its own pumping-station and perhaps its own source of supply; or the pumps of a higher zone may be supplied by a reservoir located at a high point in the next lower zone; or the pumps of the different zones may all be located at the same station and obtain their supply from the same source. In the gravity system a division is often made so that the lowest zone is supplied by gravity, while the upper zones are supplied by pumps. The most favorable arrangement will be determined chiefly by the cost of operation and the cost of the necessary piping. For small plants separate pumping-stations would rarely be an economical arrangement. Separate pumps placed in the same station would probably be employed; or, where the difference in pressure is not great, the same pump may be designed to supply the two services alternately. The latter arrangement will require some storage capacity in each system.

The advantage of two or more services depends largely on how great a proportion of the supply can be furnished at the lower pressure. If any considerable amount of storage is provided in the higher of two systems, advantage may be taken of this to furnish water at a high pressure for fire purposes in the lower system. For this purpose, connections controlled by suitable valves should be made between the two systems at one or more points. If the lower system contains a reservoir, it can be shut off in the same way as described for stand-pipes (page 719).

Separate systems for different pressures have a disadvantage in the fact that at their margins the two networks of pipes are not connected, and, as a result, somewhat larger pipes are required for the same efficiency than in the single system.

752. **Location of Pipes and Valves.**—The distributing-pipes should be so located with respect to street lines as to be readily found and to avoid other structures as far as practicable. The center of the street being usually reserved for the sewer, the water-pipes are placed at some fixed distance, usually from 5 to 10 feet, from the center. The side chosen should be the same throughout. The north side of east and west streets will be warmer than the south side.

Valves should be introduced in the system at frequent intervals so that comparatively small sections can be shut off for purposes of repairs, connections, etc. As a general rule, wherever a small pipe branches from a large one, the former should be provided with a valve. Thus with 10- or 12-inch pipes feeding 6-inch pipes, each of the latter should have a stop-valve at each end. At intersections of large pipes a valve in each branch is usually desirable. In a network of small pipes of uniform size, a valve in each line at each intersection, or four in all, is rather more than necessary, but two at each intersection, or a valve in each line every two blocks, answers very well. The map of Fig. 228 shows a suitable arrangement of valves for the case in question.

Valves, like pipe-lines, should be located systematically. They are usually located in range either with the property-line or the curb-line, but sometimes are placed in the cross-walks. A form of three-way and four-way valve, placed at the intersection of two pipes, has been used to some extent. This arrangement reduces the number of valve-boxes and is reported to be quite satisfactory.

753. **Hydrants.**—The general location of hydrants has already been considered in Art. 741. In fixing upon the exact location, and the side of the street on which each should be placed, a detailed examination should be made and the location determined with reference to important buildings, convenience of access in case of fires, etc. Generally the hydrant is placed on the same side of the street as the pipe, and is connected to the larger of two pipes where there is a choice.

Hydrants are of two general types: the post hydrant, in which the barrel of the hydrant extends 2 or 3 feet above the ground-surface, and the flush hydrant, in which the barrel and nozzle are covered by a cast-iron box flush with the surface. The former is more commonly used, and as it is much more readily found and more conveniently operated, it is to be preferred, except perhaps in the congested districts of large cities, or on narrow streets where all obstructions should be avoided. Post hydrants are set just back of the curb-line; flush hydrants, either in the sidewalk or in the street. In Boston and some other Eastern cities, extensive use is made of a flush hydrant placed directly over the main, or at the intersection of two mains.

The branch supplying the hydrants should be of a size corresponding to the number of streams to be carried, and should be designed on the same principle as other pipes. For one fire-stream the branch may be 4-inch, and for two streams 6-inch, etc. The hydrant-barrel should be nearly as large.

Many styles of hydrants are on the market, most of which will give reasonably good service if properly handled. Reliability of operation is the first essential, but next in importance is the requirement that the frictional loss in the hydrant shall be small. All waterways should be ample, and sharp angles and sudden changes in size should be avoided as much as possible. Considerable difference exists in different hydrants in this respect, with a corresponding difference in the amount of pressure lost.* The main valve, which is located at the base of the hydrant, should seat accurately and remain tight, and when open should provide ample waterway. Valve-stems should be made of extra strength, as they are likely to be subjected to rough usage. Valve and stem should be removable without the necessity of digging up the hydrant. In Fig. 230 are shown two forms of hydrants which illustrate the two general types of valves used,—the gate-valve and the compression-valve. Small independent valves controlling the nozzles are useful in multiple-nozzle hydrants, as they enable hose-connections to be more conveniently made. In ordering hydrants care should be taken to have the nozzles of the same standard as those used in adjoining large

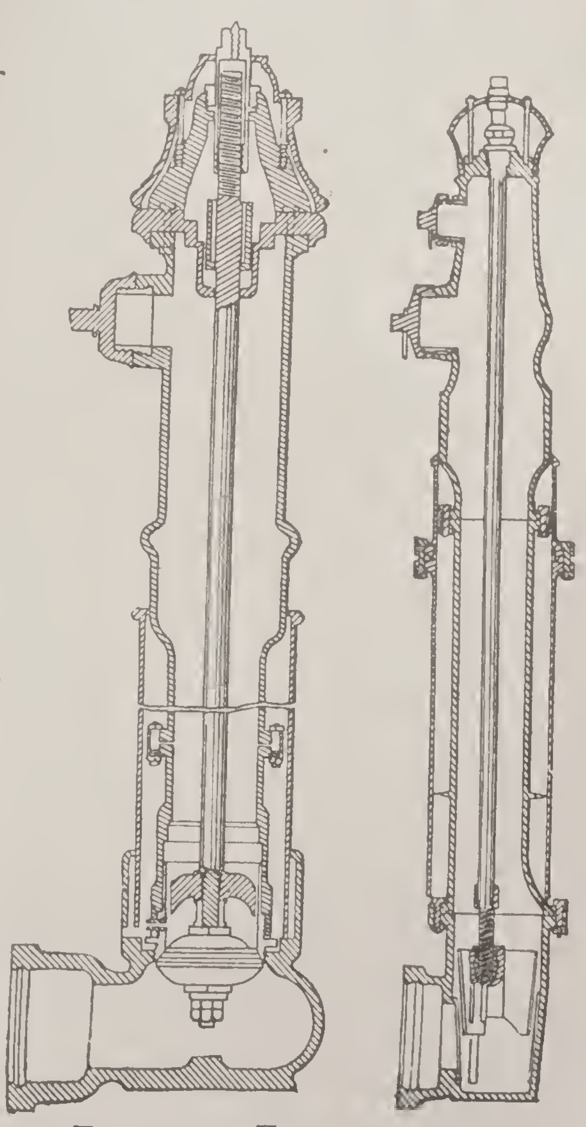

FIG. 230.—FIRE-HYDRANTS.

* See results of experiments on hydrants by Newcomb in Trans. Am. Soc. M. E., 1899, xx. p. 494.

cities, so that connections can readily be made to fire apparatus which may be borrowed in emergencies.

When a hydrant is closed after use, the water remaining in the barrel must be drained out through a drip, so arranged as to open when the main valve is closed. This is an important feature of the design, as a hydrant is likely to freeze if not thoroughly drained. The escaping water may be led away through a small drain-pipe to a sewer, or a considerable body of broken stone and gravel may be filled around the base, into which the water may be allowed to drain. If the hydrant base is below ground-water level, the drip should be plugged and the hydrant pumped out after use. Hydrants are frequently provided with an outside shell or frost-case, but the use of this has been found of little advantage. In setting hydrants care should be taken to provide a firm base and to ram solidly back of the barrel. The hydrant branch should be covered at least as deep as the main, as this branch is essentially a dead end and is much more likely to freeze than the main itself.

754. **Depth of Covering for Distributing-pipes.**—In constructing the pipe system one of the most important points to settle is the depth at which the pipes should be laid. In warm climates a covering of 2 to 3 feet is sufficient. In cold climates the depth to be adopted is that which will be sufficient to prevent freezing. In the Northwestern States the common practice of a depth of 5 to 6 feet proved insufficient during the severe winter of 1898–9, and many small mains as well as service-pipes froze. The experience at that time indicated that in this region 7 feet should be about the minimum for small pipes. In a general way it may be stated that in New York and New England the depth of cover should be 4 to 5 feet for latitude 42°, and 6 to 7 feet for latitude 45°. Between Lake Michigan and the Rocky Mountains the corresponding minimum depths should be not less than the larger of these figures. In sandy soil the depth should be a maximum. Large pipes are not likely to freeze, but should be placed at about the same depth as the smaller pipes to aid in maintaining the water above a freezing temperature.

755. **Service Connections.**—Service-pipes are usually from $\frac{3}{4}$ inch to 1 inch in diameter. The question of the most suitable material for these pipes has been discussed in Chapter XXIV. In making the connection between service-pipe and main, the latter is tapped and a brass "corporation" cock screwed in. This cock is then usually connected to the service-pipe by means of a goose-neck, or U-shaped piece of lead pipe, in order to avoid breakage from settlement of

main, although this detail is omitted by some, with apparently no bad results. At the curb is usually placed another stop-cock, with a suitable valve-box, at which point the supply to the consumer is controlled. Service connections can be made without shutting off the water, by the use of special tapping-machines, several of which are on the market.

756. **Other Details.** — In laying out lines of pipe, small depressions should be avoided, but as a rule the line may follow the street grade closely. Hydrants can usually be placed at low and high points and thus can act in a measure as blow-offs for clearing out sediment, and as air-valves. For draining large mains, small drain-pipes connecting with the sewer should be constructed at the lower points of the system.

The construction of the pipe-lines has already been described in Chapter XXV. Where pipes are laid in city streets, special care must be taken in backfilling and replacing the pavement. There is a wide difference of opinion as to the best method of backfilling, but probably the most certain way of getting the earth back without trouble from future settlement is by very thorough ramming of the material in a moist condition, but not wet. Such thorough ramming is difficult to secure, and it will usually be advisable to adopt the method of backfilling through a good depth of water. Hydrants are often deranged by being used for filling sprinkling-carts. It is much preferable to provide water-cranes for this purpose, numerous forms of which are on the market.

757. **Special Fire-protection Systems.** — Special high-pressure fire-protection systems were constructed in a few cities, conveniently located, previous to 1900, but since that time this method of improving the fire protection has been given much attention and has been adopted in several of the largest cities of the country. Very high pressures can profitably be used in these systems, a common value being 300 pounds per square inch. The pipe system should be of ample size and must be designed with especial reference to the high pressure employed. Joints are commonly hub and socket pattern, but with especially deep sockets and double grooves for the lead packing. Specials for sharp angles may well be made of steel castings. Hydrants must be of ample size and no connections other than to hydrants should be allowed.

Originally these special fire systems were laid so as to be fed from fire-boats stationed along the water front. These boats are in general use in large cities for fighting fires along the shore and among the shipping, and by laying special lines of pipe of comparatively short length, they can be made of great use in fighting fires farther inland.

TABLE NO. 102.

FIRE PROTECTION IN 21 CITIES OF THE UNITED STATES.

City.	Estimated Population.	Date of Installation.	Source of Pressure.	Gals per Min.	Maximum Pressure, Lbs.	Lineal Ft. of Mains.	Size of Mains in In.	No. of Hydrants.	Total Cost of System.	No. of Acres.	Cost per Acre.	Connection with Buildings.	Effect on Insurance Rates.
Atlantic City	40,000	Proposed	1 Station Electric Turbine Pump	7,000	225	38,590	8–14	82	$187,272	306	$612.	...	...
Baltimore	575,000	Proposed	Pumping Station	...	...	75,900	10–20	...	*397,999	360	...	Standpipes on Buildings	...
Boston	620,000	1898	Fire-boat	6,000	200	4,700	12	14	30,080	65	463.	...	...
Brooklyn	1,400,000	Constructing	2 Pumping Stations Electric Turbine Pumps	32,000	300	...	8–20	...	1,384,500	1420	975.	...	...
Buffalo	420,000	1897	3 Fire-boats	...	300	12,756	12	...	...	...	...	...	Reduction of 30 cts. per $1000
Chicago	2,100,000	Proposed	1 Station Gas Triplex Pumps	30,000	300	268,900	8–36	850	3,203,480	1280	2503.	No Open Connection	Reduction of 25% assumed
Cleveland	480,000	Constructing	To have Pumping Station. At present 2 Fire-boats	10,000	300	32,524	8–20	96	*170,000	338	...	May have Connection with Auto. Sprinklers	Reduction of 80 cts. per $1000 proposed
Coney Island	...	1905–6	1 Station Gas Triplex Pumps	3,600	150	...	8–16	...	90,000	147	612.	...	Reduction of 25%
Detroit	380,000	1893	2 Fire-boats	10,000	210	25,831	8–10	95	...	356	135.	...	Probably has prevented an increase
Fitchburg	33,000	...	Gravity †	...	180	28,250	8–16	...	50,000	346	144.	Boiler Feed, Elevators and Sprinklers	Prevented an increase
Hartford	98,000	Proposed	1 Station	10,000	300	53,430	8–24	198	796,277	731.3	1089.	No Open Connection	...
Lawrence	75,000	1906	Gravity †	...	134	10,200	10–12	39	...	120	...	...	No Change
Milwaukee	340,000	1889	3 Fire-boats	15,000	250	45,717	6–12	183	...	630	...	...	10% Reduction
Newark	290,000	1905	Gravity	3,500	165	15,000	20–30	52	135,000	303	446.	Some Connections	10% Reduction
New York (Manhattan).	2,100,000	Constructing	2 Stations Electric Turbine Pumps	30,000	300	...	12–24	1200	3,950,400	1430	2763.	Water Curtains provided for	No Change
Philadelphia	1,500,000	1903	1 Station Gas Triplex Pumps	9,100	300	35,300	8–16	166	700,000	512	1367.	None on or in Buildings	Penalty of 25% removed ‡
Providence	200,000	1897	Gravity	3,472	116	29,409	12–24	89	143,136	358	400.	5 Automatic Sprinklers	No Change
Rochester	185,000	1874	2 Stations — Elec. Tur. Pump, Steam Tur., Tur. Pump	9,000	140	102,960	4–20	...	...	...	...	Some Connections	Graded Reduction
Toronto	215,000	Constructing	1 Station Electric Turbine Pump	14,000	300	40,000	8–20	...	500,000	287	1742.	Considering Connection	Uncertain
Winnipeg	110,000	Constructing	1 Station—Gas Producer Gas Eng., Triplex Pumps	10,800	300	15,840	8–20	...	650,000	275	2364.	Connection with Automatic Sprinklers	Uncertain
Worcester	138,000	...	Gravity †	...	165	100,320	8–30	...	...	1380	...	Elevators	No change

* Exclusive of Pumping Station and Equipment.
† System consists of extension of pipes from high service into district covered by low service.
‡ Board of Fire Underwriters have voted to reduce rates to the amount of 10 cents per $100. = A total of $40,000 if extensions costing $150,000 are made to the system.

They are usually of large capacity, — equal to from ten to thirty fire-engines, — and so can supply very large and efficient fire-streams. By means of telephonic or telegraphic communication, one fire-boat can serve any one of several pipe-lines. Connections between boat and pipe-line are made by means of several lines of hose. Generally such lines are emptied after fires and consequently must be provided with ample air and waste valves, and often with relief valves.

The increasing importance of high-pressure systems has led to their adoption in many places independently of fire-boat service, and special pumping-stations are provided for supplying the necessary water. For such purposes gas or electricity will usually be the most convenient motive power and the triplex or multiple-stage turbine pump the best form of pump.

Table No. 102 gives general data of special fire systems in 21 cities of the United States taken from a report on an auxiliary high pressure water supply for Hartford, Conn.*

In the Hartford plan the length of hose necessary was given more weight than the probable height of buildings. The system was planned to limit the general hose length to 600 feet, and to 400 feet in the case of the more important city blocks. A pressure of 300 pounds was adopted. The plans provide for 10.12 miles of 8 to 24-in. cast-iron pipe ranging in thickness from 7/8 to 1 7/8 inches, and giving factors of safety of 15.3 to 10.8. The valves are to be of the double-disk type, tested to 500 pounds per square inch. The fire-hydrants are of the gate type with four 2½-inch outlets and 8-inch barrels. The hydrant spacing is fixed at 150 feet in the congested district, decreasing to 200 feet and finally to 300 feet. For data relating to other plants see numerous references at the end of the chapter.

Where salt water is used in such systems, special care must be taken in the design of valves, etc., to avoid the combination of dissimilar metals without insulation of rubber or other like material. Otherwise galvanic action will be set up and rapid corrosion will result. Besides this feature, and the slightly increased rate of corrosion of pipes, there is no objection to the use of salt water.

758. **Records and Maps.**— All constructive features pertaining to the distributing system should be carefully recorded on maps of adequate size and suitably indexed. The exact location of pipes, hydrants, and valves is of special importance. It will be convenient to have two sets of maps for this purpose: one on a small scale showing arrangement

* See abstract in *Eng. News*, 1907, LVIII, p. 53.

and size of piping and points of connection, and a set of large-scale maps, each one showing a comparatively small section of the system, on which the detailed information can be recorded. It is of the greatest importance that valves on large mains be quickly accessible in order that great damage may be prevented in case of breakage and also to facilitate repairs. Rigid discipline and constant drill of employees is of great value in this connection.

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CHAPTER XXIX.

OPERATION AND MAINTENANCE.

759. **Conduits and Pipe-lines.** — The maintenance of conduits and large pipe-lines involves chiefly the work of cleaning and repairing. The various special structures should be frequently inspected to detect any sign of weakness, and in the case of large aqueducts the entire line should be regularly patrolled. Exposed masonry will need occasional repointing, and at points of excessive wear may need renewal at intervals. The right-of-way should be properly taken care of, and slopes of earthen embankments kept in good form. Culverts and other waterways must be looked after to see that they are not obstructed. Air-valves of pipe-lines must be frequently inspected and be kept in working order; other valves require less frequent inspection. If the water carries sediment and has a low velocity, the pipe-line should be occasionally flushed by opening the blow-off valves. Gates at terminal points and at intermediate reservoirs require frequent adjustment to regulate the flow in accordance with the demand. A telephone or telegraph line is almost indispensable in the operation of a long conduit.

Masonry conduits are likely to become coated with slime and organic growth, which will cause a large diminution of their carrying capacity, and if allowed to remain may affect the quality of the water. In such a case the aqueduct should be cleaned regularly once or twice a year, or at longer intervals, depending on the rapidity of the accumulations. In cleaning the aqueduct it is emptied and then swept with brushes and scrapers, or the work may be done by mechanical brooms mounted on cars, as was at one time the practice with the Sudbury aqueduct. Experiments made on this aqueduct, and also on the new Wachusett aqueduct of Boston, show that the carrying capacity is in each case reduced about 10 per cent by a few months' accumulation of slime. The original capacity is very nearly restored by the cleaning which the aqueducts regularly receive.*

* See paper by Patch in *Eng. News*, 1902, XLVII. p. 488, for diagrams showing effect of growths on capacity of aqueducts.

Cleaning and repairing should be done, if possible, at a favorable season, so as to render the risk of an interruption in the supply as small as may be. In a well-constructed aqueduct or pipe-line the expense of repairs should be very slight.

Large steel and cast-iron pipe-lines will rarely need to be emptied for cleaning; but in some cases accumulations of organic growth have formed, which greatly obstructed the flow and which could not be removed by blowing off. In such a case the pipe should be cleaned in the same way as a masonry structure, or by the use of mechanical scrapers as described below. The tuberculation, which occurs to a greater or less extent from the corrosion of the iron, often seriously reduces the carrying capacity of the pipe. The removal of such incrustation will restore a large part of the lost capacity, and may be a much more economical method of increasing the pressure in a system than by adding new pipes.

760. *Use of Mechanical Scrapers in Cleaning Small Pipes.*—A method of cleaning cast-iron pipes which has been used extensively in

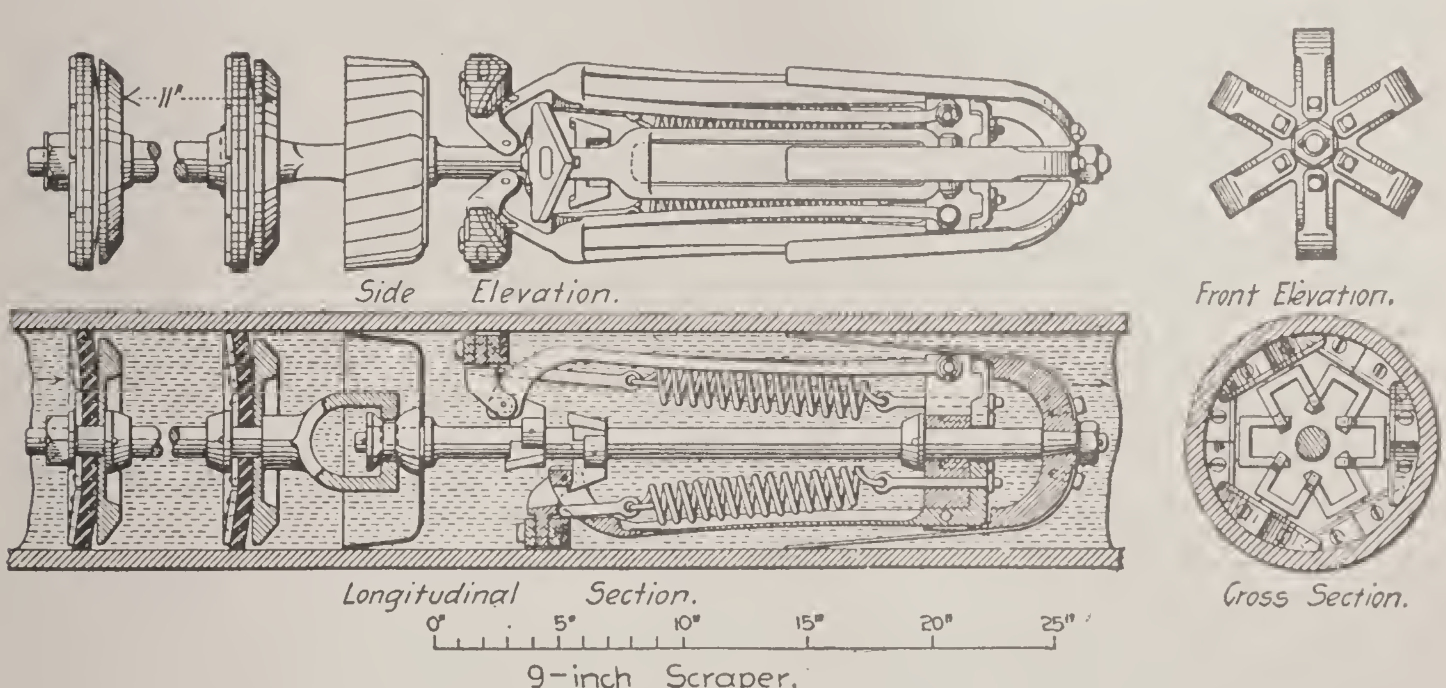

FIG. 231.—PIPE-SCRAPER.
(From *Engineering News*, vol. XLIV.)

England, and at a few points in the United States and Canada, is by the use of mechanical scrapers propelled by the water-pressure. These have been employed chiefly to remove hard incrustation, but a similar device can be used for removing less resistant obstructions. The general form of such scrapers is shown in Fig. 231, an illustration of one recently used at Torquay, England.* It consists of a scraping

* See paper by Wm. Ingham before Inst. M. E. Abstract, *Eng. News*, 1900, XLIV. p. 154. The paper also contains a description of a mechanical brush for removing peaty deposits.

mechanism fastened, by means of a jointed rod, in front of two propelling pistons which are rigidly connected together. The scraping-blades are held against the pipe by heavy springs. The scraper is introduced into the pipe through a long hatch-box, or through an opening made by removing a section of pipe. After closing the pipe a blow-off valve is opened at a point in advance, and the scraper is pushed along by the water. The apparatus may be followed by the noise it makes, and this should be done in order to locate it if it should stick. The velocity can be regulated by the blow-off valve. At Torquay such an apparatus was used as early as 1866 in cleaning an uncoated cast-iron pipe. The operation is now carried out at that place every year. The scraper here described will pass around a curve of a radius equal to about fifteen times the pipe diameter. The cost of cleaning 9- and 10-inch pipes, at Torquay, was, for labor alone, $3.50 per mile.

Scrapers similar to the one here described have been used at Halifax since 1880, certain mains at that place being cleaned twice a year, on account of the necessity of reducing the loss of head to the lowest possible limit. In 1898, 112,803 feet of mains was cleaned at Halifax at an average cost of about 0.4 cent per foot. For some of the pipes the cost was as low as 0.3 cent per foot.*

761. Pumping-stations.—Where a water-supply has to be artificially elevated, the pumping-station expenses constitute by far the largest portion of the operating expenses of the water-works system. It is therefore of the greatest importance that the highest efficiency be maintained in this part of the service. This can be secured only through skillful attendance, and the best results will be obtained by paying good wages to good men. The item most susceptible of variation is the cost of coal, and every effort should be made to reduce this to the lowest practicable limit. A daily record should be kept of the weight of coal and of ashes, so that the efficiency of the service can be known at all times. Frequently a premium paid for low coal consumption is of much aid in this matter. Sufficient reserve boiler and pumping capacity must be provided to enable repairs to be made and the boiler to be regularly cleaned and overhauled. Reserve machinery should be operated frequently to make sure it is in good condition and can be started when called for. This is especially important where it is depended upon for fire-pressure. Careful attention must be given to pump-valves and plunger-packing in order to keep the leakage or slip a minimum. Air should not be allowed to get too low in air-chambers or to accumulate too much in vacuum-chambers. Suction-pipes should be kept air-tight and a free entrance provided at all times

* See further data in *Eng. Record*, 1904, L. p. 623.

for the water. The motive power, whatever it may be, should be maintained at a high efficiency, and should have the same careful attention as is given to other high-class machinery.

Records should, of course, be kept of the amount of water pumped per day, and the pressure maintained; also of the time during which special fire-pressure is furnished, and the amount of water pumped at this pressure. Recording pressure-gauges are of the greatest value in maintaining the efficiency of a plant. By the use of several such gauges, placed in different parts of the city, a valuable record may be obtained of the actual working-pressures under different conditions. Such records will be of especial value at times of fires, and, if the pressure is insufficient, will determine whether it be due to low pressure at the pumps or to inadequate size of mains. Recording-gauges serve also as quick detectors of pipe breakages and the occurrence of stoppages, so that means can be at once taken to remedy the trouble. Probably no other detail of equal cost is of such great value to the superintendent as is a reliable recording-gauge.

762. **Distributing-reservoirs, Stand-pipes, and Tanks.**—The maintenance of earthen reservoirs calls for little more than has already been mentioned (page 337). The cleaning of such reservoirs may need to be done frequently. It is usually accomplished by flushing out the mud through the waste-pipe by means of a hose, as in the cleaning of settling-basins. Stand-pipes and tanks may require occasional flushing or blowing out, and will need to be repainted at intervals of a few years. They should also be inspected for signs of excessive corrosion or of electrolysis, and for any indication of weakness or wear at the base. Wooden tanks need rigid and frequent inspection to ascertain the condition of the wood and of the hoops. One or two of the latter will probably need to be occasionally removed to determine this point. Details should be inspected for leaks. Any automatic or quick-acting valve should have special attention to make certain that it is in working condition.

THE DISTRIBUTING SYSTEM.

763. In the operation and maintenance of a distributing system there are to be considered, besides the questions of construction already discussed, the cleaning of pipes, detection of leaks, repairs of pipes, prevention of corrosion, provision against electrolysis, thawing of frozen pipes, care of valves and hydrants, detection and prevention of waste, and the use of meters.

764. Mains and Service-pipes.—A method of removal of incrustation has already been described in Art. 760. To remove sediment from the pipe system use is made of blow-off valves or hydrants. Dead ends may need quite frequent flushing on account of odors and bad tastes developing in the stagnant water. Large leaks in mains will quickly make themselves known, especially if a recording pressure-gauge is in use. Prompt action in shutting off the supply is often necessary to prevent heavy damage. Small leaks, if occurring in clay soil, will usually be indicated by the appearance of water at the surface, but in porous soils, and especially near sewers or drains, quite large leaks may go unnoticed for years. These may, however, be detected by the method described in Art. 768 for the detection of waste. Leaks in services, between the curb-cock and the house, can also be likewise detected. Broken sections of pipe must be cut out and replaced, either by cutting the pipe and putting in a short piece by means of sleeve-joints, or by melting out the lead joints of three sections and introducing a new length of pipe.

765. *Electrolysis.*—A serious form of corrosion which has given trouble in many cities is the electrolysis which is caused by return currents from single-trolley electric railways. In this system the return current is supposed to pass through the rails, but as these are not insulated, a portion passes through the earth to neighboring pipes or other conductors leading in the right direction. This current then flows along the pipe with more or less resistance until it reaches a neighborhood where the rails or some other conductors are of lower potential than the pipe, this being usually in the vicinity of the power-station. The current then leaves the pipe, and in so doing sets up corrosive electrolytic action. Such action takes place only where the current passes from the pipe to the ground, and not where the current passes from the ground to the pipe. It depends in amount upon the strength of the current, and upon the character of the salts in the soil. If the current in the pipe is strong, corrosion will also take place near the joints. This is due to the fact that the joints offer relatively high resistance, thus causing a part of the current to leave the pipe and pass around the joint through the soil or the water and back to the pipe on the other side. This corrosion near the joint is apt to be much less than at other points, but recent reports indicate that it is likely to be serious.

Electrolytic corrosion is in some cases so rapid that pipes are practically eaten through in three to four years, and some of the worst cases have occurred where the pressure is but $1\frac{1}{2}$ volts. At Peoria, Ill., a stand-pipe which failed had been badly corroded by electrolysis, and this was doubtless a prime cause of its failure.

The remedies for electrolysis should apparently rest entirely with the railway companies. The double trolley is the only complete remedy, and it has been applied extensively, and with success, in one or two places. A very important aid in preventing electrolysis is the construction of a good return conductor by means of good rail-bonding and the use of adequate return wires. Then in those districts where the pipes are of higher potential than the rails, if good, low-resistance connections are made between rails and pipes, or from pipes to special return wires, the current will leave the pipes without passing into the ground and without causing trouble. Voltmeter tests between pipes and rails, at various points over the city, will determine the danger area, but such tests should be made under a variety of conditions and at occasional intervals. Pipes and rails should not be connected outside of the danger area, as this only aggravates the trouble by conducting more current into the pipes. This method of making connections to the pipes does not obviate the trouble at the joints, but rather increases it, as it adds to the conductivity of the pipes.

It has been proposed to insulate the pipes by the occasional use of a wooden section, or by the use of wooden joints, so as to render the pipe a poor conductor. The success of such means would depend upon whether the current could be sufficiently reduced to avoid electrolysis at the end of each individual section, as now occurs at the joints.

766. *Thawing Frozen Pipes.*—Not infrequently considerable trouble arises from the freezing of service-pipes which are not placed at a sufficient depth. Occasionally, also, small mains are frozen. Where the proper facilities exist the best way to thaw frozen pipes is by warming them with an electric current, a method applied for the first time by Professors Jackson and Wood at Madison, Wis, in 1898–99, and which has been used in many places since then.

For thawing service-pipes a current of 200 to 300 amperes at a pressure of 50 volts is satisfactory, and will ordinarily thaw a pipe in from 20 to 30 minutes. The current can conveniently be taken from electric-light wires and reduced by a transformer. Connections can readily be made to a faucet in the house and to a fire-hydrant outside, or to faucets in two houses. To regulate the current large sheet-iron terminals can be immersed in a bucket of salt water. Direct connection to house-lines should not be made on account of the danger of fire. A 6-inch main 320 feet long has been thawed in two hours with a current of 350 amperes at 100 volts.

Where the electrical method cannot be used steam may be employed, not only to warm the pipe, but to excavate through the frozen ground in a way similar to the operation of the water-jet. The pipe may thus be reached at points 4 to 5 feet apart and gradually thawed out. Service-pipes are often thawed by the use of a small steam-pipe inserted in the service-pipe through the house end, or from an opening at an excavation outside. Ground may be thawed by maintaining a fire on the surface for several hours, or more readily by the use of a gas-flame projected against the soil.

767. Valves and Hydrants.—Valves should be inspected occasionally to detect leakage and to ascertain if they are in working order and the boxes clean. Fire-hydrants require very careful attention, especially in cold climates, as it is of the greatest importance that they be at all times available. The chief trouble with fire-hydrants is from the freezing of the valves due to imperfect drainage, although a hydrant branch sometimes freezes up.

Hydrants should be carefully examined on the approach of cold weather and put in good condition. Valves should be tight and the hydrant thoroughly drained. If so located that the hydrant cannot be drained, it should be pumped out each time after being used. To ascertain if a hydrant is drained, a lead weight tied to a graduated cord can be let down through a nozzle. Hydrants should never be opened unnecessarily in cold weather, and never by others than those responsible for their condition. In very cold climates it is found desirable after using a hydrant to oil the packing and the nut at the top with kerosene in order to prevent sticking of the valve and nut.

To thaw frozen hydrants, a small portable steam-boiler is commonly employed, which is provided with a length of hose for conducting steam to the bottom of the hydrant. Hot water may also be used, and for mild cases a little salt may be effective. After thawing, the water should always be pumped out.

768. Detection and Prevention of Waste.—From the data given in Chapter II it was made evident that a very large percentage of the water supplied to American cities is wasted by the consumer and lost by leakage. In many cities the consumption of water is easily double the amount which can possibly be made use of, and in a very large proportion of them the wastage is fully one-third of the entire quantity supplied. This excessive use of water not only increases the cost of pumping unnecessarily, but adds to the expense in all parts of a water-works system. Its effect is noticed perhaps most of all in the reduction of pressure, since the frictional head is nearly proportional to the square of the discharge. For the same reason a moderate reduction of the consumption will result in a large increase of pressure. The problem of waste-prevention is thus seen to be one of great economic importance.

Unquestionably the easiest and most rational method of preventing the waste of water is by the use of meters, so that each consumer will pay for what he uses. It furnishes also the most equitable basis for charging up the cost of service, as by any other system the careful user is forced to pay for the water wasted by his careless neighbor. The use of meters is becoming much more general, and in most cities the larger consumers, at least, are now metered; but a very large part of the loss or waste is due to the small consumer, so that the full benefit of the system will not be felt until the use of meters becomes general. Usually much opposition is raised to the introduction of meters, but after they have been put into use the results are commonly such as to cause them to be greatly favored by the community. The effect of the use of meters has been generally discussed in Chapter II, and many individual cases could be cited showing the great economy consequent upon the introduction of the meter system. As a system of waste-prevention it is always in service, and for that reason is far superior to any system of inspection. In nearly all cases the decrease in cost of supplying water after the adoption of meters much more than balances the cost of the meters.

If meters are not used, some method of inspection is highly desirable whereby the most serious cases of waste can be detected and the consumption kept within reasonable limits. The most common method is a house-to-house inspection, carried out one or more times per year for the purpose of examining the plumbing fixtures. Any leaky or imperfect fixture is ordered repaired, and the premises reinspected shortly to make sure that the order has been complied with. Persistent refusal is followed by the shutting off of the supply. This method of inspection cannot be carried out frequently, and is of no value in preventing willful waste through open fixtures.

A comparatively effective method of house-to-house inspection was employed temporarily in St. Louis to avoid a water scarcity. A night inspection was first made at the curb-cock by means of a long key with the end flattened so that the ear could be held against it. By turning the water off and then turning on again, a flow as small as 5 gallons per hour could be detected by the hissing noise made. Any appreciable flow at night was inquired into next day, the house fixtures inspected, and notice given to avoid the waste of water. The same house was shortly reinspected, and after two or three notices to repair plumbing or stop waste the water would be shut off.

If meters are not used a system of inspection by districts will serve to determine and control the waste to a considerable extent. To accomplish this some method of measuring the water flowing into a given district must be employed. This system of district inspection was introduced in Liverpool in 1873 by Mr. G. F. Deacon, and a waste-water meter was devised by him to determine the flow.* His meter is in general use in many cities in England and has been employed to a limited extent in this country. A more convenient and economical method of determining the flow for inspection purposes is by the use of the Cole-Flad pitometer. This instrument consists of a pair of Pitot tubes, which can be inserted in a water main through an ordinary corporation cock. The pressure within these tubes is communicated to a glass "U" tube and recorded photographically by suitable apparatus. It can readily be carried from place to place and is found to be very satisfactory for inspection purposes.†

In the district system of inspection the city is divided into sections of a few blocks each and valves are closed controlling the section so that all water supplied to it will pass the meter or other measuring apparatus. If the records thus secured show the night consumption to be large, thus indicating much waste, this waste is then localized to certain streets as far as possible, by shutting off different streets in succession and noting the resulting curves. Finally the houses in the worst streets are inspected, or the excessive waste localized still further by closing off individual services, and noting the effect on the records, or by the same method as employed at St. Louis.

One of the weak points of the meter system is that it fails to detect leaks in the mains or in the services beyond the meters. The district system is advantageous in this respect, for by shutting off all services the leakage in the mains is at once known. It may thus be applied to good advantage even where the meter system is in operation, if a large amount of water is "unaccounted for." To localize a leak in a main, a waterphone may be used, which consists of a staff of wood or iron having at one end a diaphragm and ear-piece similar to a telephone-receiver. The staff is placed against the pavement over the pipe at various points, and the ear applied to the receiver, when any sound made by a leak is readily perceived.

Any method of waste-prevention except by the use of meters is of temporary value only and must constantly be repeated, and, to be

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* See illustration in Trans. Am. Soc. C. E., XXXIV. p. 199.

† See references at end of chapter.

effective at all, must be supported by strong laws and good plumbing ordinances.

769. **Meters.**—Water-meters may be divided into two general classes: the positive displacement meter, in which a definite quantity of water passes at each complete movement (neglecting the effect of the slight clearance necessary), and the inferential meter, in which the moving water actuates a screw or other similar mechanism and the amount of flow is inferred by the number of revolutions of the screw. The former type is in general use in this country, but the latter is a common form abroad. Both forms are sufficiently accurate at ordinary rates of flow, but the inferential type is the less accurate at low rates. Of the displacement meters there are the piston type, having either a reciprocating or a rotary piston, and the disk type, in which a disk has a sort of wobbling motion in a closed chamber. Most of the meters in use are of the rotary-piston or the disk type. Many different kinds of meters are on the market, most of which will give satisfactory service if properly treated, and many of them have been thoroughly tested by years of use. No new form of meter should be adopted without thorough and long-continued tests, and in all cases it is well to specify the desired requirements of a meter, and to test all new meters, in order to insure uniformly good workmanship.

The general requirements of a meter are: a fair degree of accuracy, ability to register approximately quite small rates of flow, suitable capacity for a given loss of head, durability, and low cost. All of these requirements except that of durability can readily be determined by a brief test. Some notion of the durability can also be had by a careful inspection of the parts, and by running a meter at a rapid rate for a considerable period and again determining its accuracy and sensitiveness. Maintained accuracy, accessibility, and ease of repairs are the most important qualities of a meter.

Great accuracy in the measurement of water is unnecessary. Most meters on the market will register within 1 or 2 per cent of the correct amount at ordinary rates of flow, which is abundantly accurate. To avoid dissatisfaction it is desirable that the error of registration be in favor of the consumer. A small error is otherwise of little consequence.

Sensitiveness, or accuracy at low rates of flow, is much more difficult to secure, and is a point in which meters differ more widely. Sensitiveness is desired in order that some account may be taken of small leaks, which in the aggregate may amount to a large proportion of the total flow. If the consumption of a family be 150 gallons per day, this will be at an average rate of about 6 gallons per hour. A flow of water at a uniform rate of one-half this amount would not move some meters at all. Great accuracy for small rates is not needed, but it is desirable that small flows be accounted for in part at least. A sensitiveness of about 90 per cent registration for a flow of 10 gallons per hour ought readily to be obtained.

In a test of fourteen different kinds of meters by J. W. Hill,* several of the $\frac{5}{8}$-inch meters tested would register a flow of 10 to 12 gallons per hour with less than 10 per cent of error. Tests of seven meters by J. W. Smith † gave a registration of 95 per cent of the flow with rates of 3 to 24 gallons per hour. His experiments also showed little reduction of accuracy or sensitiveness in the best meters, after registering an amount of water corresponding to thirty-five to forty years of service, and most were in good condition after a use corresponding to one hundred years of service. In general, the disk meters experimented upon showed a more uniform degree of accuracy at different rates and a better maintained accuracy than the piston type. The figures just mentioned cannot be taken as indicating the actual life of a meter, as many other things besides the actual wear affect the durability. The actual life of a good meter is probably not over twenty years, and in many cases will be less than this. The accuracy and durability depend much on whether the water contains suspended matter, and upon the character of the same.

Meters should be so designed that the various parts will be easily accessible and readily replaced, and the moving parts protected from serious injury by frost. The latter object is usually accomplished by frost-bottoms of cast iron, or cast-iron cases, made so as to be more easily broken than other and more costly parts of the meter.

The loss of head in a meter is a matter of some importance, as this virtually determines the size necessary for a given capacity, although meters are usually rated according to the size of connecting pipes. The ordinary sizes for domestic service are $\frac{5}{8}$ and $\frac{3}{4}$ inch. The loss of head in seven $\frac{5}{8}$-inch meters tested by J. W. Smith, varied from 3 to 12 pounds for a rate of flow of 10 gallons per minute, a rate which would consume about 70 feet of head per 100 feet of $\frac{5}{8}$-inch pipe. For 5 gallons per minute the loss of head ranged from 1 to 3 pounds. Mr. Hill found in disk meters, for 10 gallons per minute, a loss of 6 to 8 pounds, and in piston meters 7 to 13 pounds. For 5 gallons per minute the losses were, respectively, 2 to $2\frac{1}{2}$, and 2 to 3 pounds.

* Trans. Am. Soc. C. E., 1899, XLI. p. 326.
† *Ibid.*, p. 359.

The cost of $\frac{3}{4}$-inch meters is ordinarily from \$8 to \$12 each; and cost of setting \$1.50 to \$3.00. The cost of maintenance of meters at Providence, R. I., where careful accounts have been kept, is as follows: Interest on meters and setting, 50 cents; depreciation, assuming a life of twenty years, 75 cents; maintenance and repairs, testing, etc., 46 cents; reading and computing bills, 42 cents; total, \$2.13. The cost of repairs at several other places is variously reported at from 10 to 35 cents per year.

FINANCIAL.

770. General Considerations.—The financial management of a municipal water-works department is a matter of much importance to a community, inasmuch as upon this management depends largely the question of rates and, to some extent, of other forms of taxation. The total cost of the service must eventually be borne by the community, but much care is necessary in fixing the rates so that the expense will be equitably distributed, both with respect to various individuals at the present time, and with respect to future generations. To fix the rates equitably requires, first, a careful calculation of the expenses to be met; then a determination of how much should be met at the present time and what portion should be left to future generations; then what proportion of the total expense should be raised by water rates and what portion, if any, by general taxation; and finally, whether it be wise or expedient to so adjust the rates that the revenue will exceed the expenditure and so act to lower taxation in other ways. In the case of private companies the last element would represent the profit.

In many respects the question is largely a matter of bookkeeping, but it is highly desirable that a proper and businesslike method of accounting be adopted, both as an aid in equitably fixing the charges, and to enable the public to know the exact financial condition of the water department as a separate business.

771. Expenses and Charges to be Met.—The yearly expenses and charges will be included under some or all of the following heads:

1. Interest on bonded debt incurred for construction.

2. Yearly operating and maintenance expenses.

3. Yearly payment into a sinking fund for liquidating the bonded debt.

4. Yearly payment into a depreciation fund to provide for the renewal of various parts of the works when worn out or otherwise rendered valueless.

5. Yearly cost of extensions and improvements.

6. Profit.

Items (1) and (2) must evidently be fully met year by year by the annual income, and not by borrowing, if the department is to remain solvent. The only question is as to what should be included under the term maintenance. In some works it is customary to charge up some part of the cost of extensions to maintenance; also the replacing of small pipes with larger ones, and renewals of various other portions of a plant. But to keep the question clear it is usually considered better to include under maintenance only the regular operating expenses and the cost of minor repairs.

(3) and (4). In addition to the interest and maintenance expenses, a fund must be provided from the annual income, either for the payment of the borrowed money by the time the works are worn out, or for rebuilding the various parts when necessary; otherwise a city would, in the course of time, find itself with a worn-out plant on its hands and a bonded debt in addition. To provide both a sinking fund and a depreciation fund would be to tax the present generation for the entire first cost of the works, and for its renewal or its maintenance in perfect condition. This method of management is usually considered much too liberal towards the future generations, but may be adopted in part where the city finances are in good condition.

In actual practice the sinking fund usually receives the most and often the only consideration, and by some States such a fund must be provided for. If the sinking fund be adjusted to pay the bonds at the end of a period corresponding to the life of the plant as a whole, or for safety a little short of this time, then the sinking-fund provision is equivalent to a fund for depreciation, and the finances will be held in equilibrium. Renewals will then be paid for by a new issue of bonds, and the payments into the sinking fund will continue. To provide for contingencies and to relieve the future generations to some extent, it is considered good policy to make the sinking fund such as to pay off in time all the original debt, including that portion covering the permanent parts of the plant.

If a sinking fund is not provided, then a depreciation fund is necessary. This should be sufficient to furnish funds for the renewal or replacement of old parts, and, as a margin of safety in calculating the payments into this fund, the more permanent portions of the works should be assumed to have a limited life. A portion of the depreciation fund can then be used to gradually extinguish a part of the bonded debt.

(5) The cost of extensions may properly be met in the same way as the cost of new works, namely, by issuing bonds and at the same time providing a corresponding increase in the sinking or the depreciation fund. Such expenses are, however, as a matter of accounting, often paid in part from the annual receipts, or by general or special taxation, or by both methods.

(6) As a general proposition there can be no "profit" derived by a city from supplying itself with water. If more is paid into the treasury than sufficient to meet the expenses, it can only be considered as a sort of indirect tax levied for other purposes.

From these considerations it is evident that the annual charges upon the community must on the average cover at least the interest on the bonded debt, the operating expenses, including ordinary repairs, and a payment into a sinking or a depreciation fund. (For formulas for calculating sinking or depreciation funds see page 216.)

772. **Relative Cost of the Different Services Performed by a Water-works.**—The functions performed by a water-works are: (1) to furnish water for private use; (2) to furnish water for public use on the streets, and for sewers, fountains, public buildings, etc.; and (3) to furnish fire protection to property. In (1) and (2) the cost of service may be considered approximately proportional to the quantity of water supplied, but in (3) it is out of all proportion to the amount of water used, for while the cost of construction is greatly affected, the total amount of water consumed is slight. The extra cost involved in furnishing adequate fire protection is due to largely increased pumping capacity, increased size of mains, reservoirs, or stand-pipes, cost of hydrants, and increased cost of maintenance. Estimates of careful observers place the proportion of interest, depreciation, and maintenance expenses chargeable to fire protection at one-third or one-half the entire cost. Another very considerable part of the expense which is not directly chargeable to present consumers of water is the provision made for future growth. The expense of this in first cost may also easily be one-third the entire cost of construction.

773. **Sources of Revenue.**—The sources of revenue are the water rates and the funds received by general taxation. The former are paid by those who use the water, and more or less in proportion to the amount used. The latter are paid by assessment on all taxable property. If the revenue be so raised that each interest served be charged according to the cost of the service, it would appear from the

preceding article that the cost of furnishing water to private consumers should be paid by water rates; that the cost of supplying water for public purposes should be paid by taxation and according to the amount of water used; and that the cost of fire protection should also be met by taxation, since the individual is benefited by reason of the protection afforded to property. The expense of providing for the future should also properly be met by the city as a whole, and therefore by general taxation. It would therefore seem that ordinarily from 35 to 40 per cent of the total expense, plus the cost of the water used for public purposes, should be met by general taxation, and the remainder of the revenue obtained from the water rates. The exact proportion of the revenue which should be derived from each source depends much upon local conditions, such as size of town, character of supply, etc. In many small towns the works are primarily installed for fire-protection purposes, in which case nearly all the expense should be met by taxation. It is also good policy to begin with fairly low water rates, so as to encourage the use of water, but to enable this to be done a large proportion of the expense will have to be met for a few years by taxation.

In most water departments the general principle of distributing the cost as above outlined is recognized by making payments from the general fund into the water department, either yearly or at irregular intervals. In but few places, however, is the system fully carried out.

774. **Water Rates.**—The proportion of the revenue to be derived from private consumers requires careful consideration in its adjustment. The most equitable method of apportioning the cost is by the meter system. In fixing rates under this system, allowance should be made for the fact that quite a large percentage of the water recorded at the pumping-station cannot be accounted for (Chapter II), and rates per unit of volume registered by the meters must be correspondingly raised.

Meter rates are usually graduated, that is, a less rate is charged for large quantities than for small ones. This is partly on the ground that the cost of meter maintenance, keeping of accounts, etc., is proportionally greater for small quantities, and partly by reason of the policy of encouraging the operation of factories which contribute largely to the general prosperity of the community, and which may require large amounts of water. In establishing a graduated schedule, it should be so made that the lower rate shall apply only to the additional water used beyond the limit of the next higher rate. An example of such a schedule is that at Madison, Wis., which reads as follows:

For 1,500 cu. ft. or less per 6 months,		$2.00
" the next 5,000 cu. ft. " " "		13 cts. per 100 cu. ft.
" " " 15,000 " " " " "		10 " " " " "
" all over 21,500 " " " " "		5 " " " " "

An objection to the meter system which is often advanced is that it discourages the use of sufficient water for sanitary purposes, but this is entirely obviated by making a small minimum charge, such as given above, which will be enough to allow the use of an abundance of water for sanitary purposes, and at the same time will cover the expense of meter maintenance.

Most cities meter the larger consumers, but comparatively few have yet introduced the full meter system. In such cases private houses are charged mainly by the fixture. Usually a minimum family rate is charged for kitchen use, then an additional rate for each bath-tub, water-closet, wash-bowl, stable-hose, lawn-hose, etc., with often other variations depending upon the number of rooms, number of occupants of the house, etc. Little data exist as to the actual amount of water used by different fixtures, and the rates are largely arbitrary.*

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INDEX.

DIAGRAM FOR CALCULATING CAST-IRON PIPES.